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A. INFORMATION & THEORIES
This chapter elaborates on general theories used during the design process. In addition the EIA demands are specified.

A.1 EIA demands
An EIA is mandatory for the project if the requirements found below are met. The listed requirements have been cited from the EIA as obliged by the Department of Environmental Affairs (2010). The requirements below are considered to be relevant for any plausible solution. A proposed solution is obliged to meet the EIA requirements.

9. The construction of facilities or infrastructure exceeding 1000 meters in length for the bulk transportation of water, sewage or storm water -
   (i) with an internal diameter of 0.36 meters or more; or
   (ii) with a peak throughput of 120 litres per second or more,
   excluding where:
   a. such facilities or infrastructure are for bulk transportation of water, sewage or storm water or storm water drainage inside a road reserve; or
   b. where such construction will occur within urban areas but further than 32 meters from a watercourse, measured from the edge of the watercourse.

11. The construction of:
   (i) canals;
   (ii) channels;
   (iii) bridges;
   (iv) dams;
   (v) weirs;
   (vi) bulk storm water outlet structures;
   (vii) marinas;
   (viii) jetties exceeding 50 square meters in size;
   (ix) slipways exceeding 50 square meters in size;
   (x) buildings exceeding 50 square meters in size; or
   (xi) infrastructure or structures covering 50 square meters or more
   where such construction occurs within a watercourse or within 32 meters of a watercourse, measured from the edge of a watercourse, excluding where such construction will occur behind the development setback line.

12. The construction of facilities or infrastructure for the off-stream storage of water, including dams and reservoirs, with a combined capacity of 50000 cubic meters or more, unless such storage falls within the ambit of activity 19 of Notice 545 of 2010.

15. The construction of facilities for the desalination of sea water with a design capacity to produce more than 100 cubic meters of treated water per day.

18. The infilling or depositing of any material of more than 5 cubic meters into, or the dredging, excavation, removal or moving of soil, sand, shells, shell grit, pebbles or rock or more than 5 cubic meters from:
(i) a watercourse; 
(ii) the sea; 
(iii) the seashore; 
(iv) the littoral active zone, an estuary or a distance of 100 meters inland of the high water mark of the sea or an estuary, whichever distance is the greater but excluding where such infilling, depositing, dredging, excavation, removal or moving;
(a) is for maintenance purposes undertaken in accordance with a management plan agreed to by the relevant environmental authority; or 
(b) occurs behind the development setback line.

37.
The expansion of facilities or infrastructure for the bulk transportation of water, sewage or storm water where:
(a) the facility or infrastructure is expanded by more than 1000 meters in length; or 
(b) where the throughput capacity of the facility or infrastructure will be increased by 10% or more—excluding where such expansion:
(i) relates to transportation of water, sewage or storm water within a road reserve; or 
(ii) where such expansion will occur within urban areas but further than 32 meters from a watercourse, measured from the edge of the watercourse.

39.
The expansion of
(i) canals;
(ii) channels;
(iii) bridges;
(iv) weirs;
(v) bulk storm water outlet structures;
(vi) marinas;
within a watercourse or within 32 meters of a watercourse, measured from the edge of a watercourse, where such expansion will result in an increased development footprint but excluding where such expansion will occur behind the development setback line.

41.
The expansion of facilities or infrastructure for the off-stream storage of water, including dams and reservoirs, where the combined capacity will be increased by 50000 cubic meters or more.

45.
The expansion of facilities in the sea, an estuary, or within the littoral active zone or a distance of 100 meters inland of the high-water mark of the sea or an estuary, whichever is the greater, for—
(i) fixed or floating jetties and slipways; 
(ii) tidal pools; 
(iii) embankments; 
(iv) rock revetments or stabilizing structures including stabilizing walls; 
(v) buildings by more than 50 square meters; 
(vi) infrastructure by more than 50 square meters; 
(vii) facilities associated with the arrival and departure of vessels and the handling of cargo; 
(viii) piers; 
(ix) inter- and sub-tidal structures for entrapment of sand;
(x) breakwater structures;
(xi) coastal marinas;
(xii) coastal harbours or ports;
(xiii) structures for draining parts of the sea or estuary;
(xiv) tunnels; or
(xv) underwater channels – where such expansion will result in an increase in the development footprint of such facilities but excluding where such expansion occurs:
(a) behind a development setback line; or
(b) within existing ports or harbours where there will be no increase in the development footprint or throughput capacity of the port or harbour.

A.2 Tetra
Tetra is a multi-criteria decision making and evaluation tool. Tetra is a measurement and decision making tool that is based on sound mathematical foundations. The program constructs preference scales using user’s criteria weights and alternatives ratings. The program input consists of specifying the alternatives and the criteria and sub-criteria on which the alternatives are rated. The alternatives are rated on the criteria in a simple and intuitive manner and the criteria are weighted by their relative importance. Tetra then rates the alternatives on a scale that takes into account all these pieces of information. (Scientific Metrics, 2011) Tetra is used to execute different kind of trade-offs during the design synthesis.

A.3 Flow data
The most important input dataset used in modelling the system is flow data. The data listed in Appendix B.3 has been processed and converted into triangular flow peaks. US SCS method (U.S. Dept. of Agriculture. Soil Conservation Service, 1972) is used to compute triangular peaks. This method calculates the runoff volume and time to peak of runoff assuming a uniform rainfall over whole the catchment area.

A.4 Use of a 1D river model program
Modelling of rivers and channels can be accomplished with various 1D flow models. The most frequently used programs are MIKE 11 developed by the DHI group, HEC-RAS developed by the US Department of Defence, and SOBEK developed by Deltares. In this research SOBEK is used because of previous experience with this program. In addition SOBEK features a user friendly GIS interface which offers a quick and easy way to build a model. Furthermore SOBEK is able to cope with dry sections in a river system which is not possible in HEC-RAS.

A.5 Norms for structural design calculation
The Eurocodes guidelines have been followed during the design process of the wooden weir structure. The Eurocodes were preferred over the British Standards after a consultation with prof. Derek Stretch. According to Zingoni (2001) British Standard will replace the Eurocodes in the near future.
A.6 Game theory

The game theory is a method used in mathematical economics and business for modelling competing behaviour of interacting agents. (Wikipedia, http://en.wikipedia.org/wiki/Game_theory 2011) It is applied in many management literature and practice. It is related to decision-making and is applied in economics, sociology, and biology. The mutual interaction between decision-making is a important subject to understand the game.

“The metaphor of play is exploited in order to contribute our understanding of stakeholder management as a whole. By following Morgan’s (1980) considerations, the paper suggests that the playing metaphor may open us a new and creative way to examine and manage organizational stakeholders, though, of course, no one metaphor can capture all elements of organizations and inter-organizational behaviour.” (Stakeholder Management as a Play, Kalle Pajunen; Juha Näsi, Frontiers of E-Business research 2004)
B. DATA ACQUISITION

In this chapter different data is elaborated. The data has been retrieved and used as an input during the research. In paragraph B.1 the W2H032 gauge is elaborated. In B.2 the Cotcane gauge data is described. Furthermore, input data from the Mfolozi (B.3), fieldtrip results (B.4), flow velocity and erosion (B.5), the hypsometric curves (B.6), and the water balance error are elaborated in this chapter as well.

B.1 W2H032 Gauge

The W2H032 gauge is maintained by the hydrological services from the Department of Water Affairs of South Africa. The gauge is located 12km inland on the embankment of the Mfolozi as shown in Figure B-1 and logs the water level every 12 minutes. Both corrected and uncorrected data is freely available on the website of the department of water affairs.

The 12 minute runoff of the Mfolozi is computed based on the measured water level and a known cross section of the river. The accuracy of this computation is unknown as many factors can influence the results. It is unknown how often the cross section is measured in order to keep track of bed level changes or other changes of the riverbed. Any runoff above 783 m$^3$/s falls outside the operational range of the gauge.

The W2H032 gauge is assumed to be fairly accurate. One has to be cautious with the obtained gauge data because little is known about its accuracy.

Table B-1 till Table B-5 contain the daily discharge data from the W2H032 gauge. The 12 minute data is included in the data folder on the cd.
Table B-1 W2H032 daily discharge data; June 2011 (Department: Water Affairs, 2011)

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B.2 Cotcane gauge data

The Cotcane water level gauge Figure B-2 is a privately owned gauge maintained by UCOSP. The gauge is located close to the sugarcane fields upstream the Msunduze. Water levels are measured electronically every twelve minutes and downloaded manually once a day.

Figure B-2 Location of the Cotcane gage.

Figure B-3, Figure B-4 and Figure B-5 list data obtained from the Cotcane gauge in the period of the 31st of May 2011 until the 14th of September 2011. The daily values are taken
at a fixed point in time and do not represent the daily average flow. According to Gerrit de Jager (2011) values obtained before the 26th of September 2011 must be adjusted by +0.181m in order to obtain correct results. Data is assumed to be in GMSL.

The accuracy of the Cotcane gauge data is unknown. Comparison of the 12 minute data with field data measured on the 11th October 2011 (paragraph B.4.2) indicate there is an error of approximately -4cm with corrected values measured by the Trimble RTK station. Cotcane gauge has been surveyed with RTK equipment on the 26th of September 2011. Sukulu no37, a location with known height, longitude and latitude coordinates was used as base station. The distance between St Lucia and Sukulu is approximately 15 aerial kilometres. The accuracy of RTK GPS readings depends on the accuracy of the equipment. Equipment with a spatial decorrelation of 1ppm is often used in the industry these days. Its accuracy is up to 1.5 cm with the given spatial decorrelation and a distance to the base station of 15 kilometre. According to D. Stretch (2011) the Sukulu trig station is not well calibrated and does not return accurate heights.

In addition the “flow into lake” is given in Figure B-3, Figure B-4 and Figure B-5. The flow into lake is measured by Ricky Taylor with a stick. The obtained values are guestimates and can differ an estimated 25% from reality.
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Figure B-3 Cotcane measurements from the first of June 2011 till the 30th of June 2011.
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Figure B-4 Cotcane measurements from the first of June 2011 till the 30\textsuperscript{th} of June 2011.
Figure B-5 Cotcane measurements from the first of June 2011 till the 30th of June 2011.

**B.3 Input data of the Mfolozi River**

One of the most important input data for the water balance model as used in the variant study is the flow data of the Mfolozi. The flow rates and duration of flood peaks determine (1) the required capacity of the channel between the Mfolozi and St Lucia, (2) the rising velocity of the water behind the sandbank, and (3) the moment that the berm at the beach breaches during a flood. Two data sources are available: data from the UKZN and data from the UCOSP sugar cane farmers. In this chapter the data will be analysed and finally a decision is made about what data is used as input for the models.

**B.3.1 Data from UCOSP**

The Mfolozi enters the floodplains from the West. During normal conditions, water only flows through the Mfolozi. During floods (flows exceeding 800m³/s) water levels rise and water will start to flow over the spillway into the Msunduze. When the water levels rise even further the levees will overtop and the sugar cane fields will flood. Images of the spillway are shown in Figure B-6 and Figure B-7. The spillway overtops when the flow rate exceeds approximately 800m³/s. According to Table B-6, spilling happens on average once every two years. The duration of a flood is approximately 1 to 2 days. (De Jager, 2011)
Figure B-6 Spillway and the Mfolozi, arrow indicates the direction of the picture of Figure B-7 (Google; AfriGIS Ltd, 2011)

Figure B-7 The spillway on the Mfolozi floodplain.

Table B-6 Spillway operation (De Jager, 2011)

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<th>Year</th>
<th>Flood peak (m³/s)</th>
<th>Re-calculated peaks (m³/s)</th>
<th>Flood volumes (10⁶ m³)</th>
<th>Spill Time (hours)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2000 (Nov)</td>
<td>3.200</td>
<td>2.383</td>
<td>163</td>
<td>50</td>
</tr>
<tr>
<td>2000 (Dec)</td>
<td>2.850</td>
<td>2.076</td>
<td>35</td>
<td>14</td>
</tr>
<tr>
<td>2002 (Jul)</td>
<td>1.986</td>
<td>1.475</td>
<td>12</td>
<td></td>
</tr>
<tr>
<td>2004 (Jan)</td>
<td>3.157</td>
<td>2.383</td>
<td>61</td>
<td>16</td>
</tr>
<tr>
<td>2005 (Jan)</td>
<td>2.000</td>
<td>1.230</td>
<td>10</td>
<td>8</td>
</tr>
<tr>
<td>2006 (Dec)</td>
<td>1.971</td>
<td>42</td>
<td>12</td>
<td></td>
</tr>
<tr>
<td>2006 (Dec)</td>
<td>3.359</td>
<td>105</td>
<td>±24</td>
<td></td>
</tr>
<tr>
<td>2010 (Jan)</td>
<td>2.925</td>
<td>90</td>
<td>14</td>
<td></td>
</tr>
</tbody>
</table>
B.3.2 Data from the UKZN
The hydraulic models of the Mfolozi and the St Lucia estuary used in research are based on monthly and daily runoff data. The model used by Jugwanth (2011) uses monthly runoff data, which has been altered such that flood peaks have been averaged out over a month. These flood peaks transit the monthly runoff in a few days. Other models use daily flow data which show the occurring flood peaks over time as shown in Figure B-8.

Figure B-8 Flow data of gauge W2H032 based on daily flows (Department: Water Affairs, 2011)

Small floods between 100m$^3$/s and 300m$^3$/s occur frequently whilst floods higher than 500m$^3$/s are more seldom. Based on analysed data of the past ten years one can assume that the return period of a 300 m$^3$/s flood is in the order of once every 2 or 3 years. A flow higher than 800 m$^3$/s has only occurred once in the dataset as show in Figure B-8.

Some assumptions have been made about the duration of floods. For simplicity floods are modelled as a triangle with a base width of $t = 3 \cdot T_c$. The peak of the flood is located on $t = T_c$, as shown Figure B-9. $T_c$ is the time between the beginning of the flood and the peak of the flood. The value of $T_c$, used in this simplified hydrograph is 60 hours. This value has been calculated with the aid of the US SCS method (U.S. Dept. of Agriculture, Soil Conservation Service, 1972). This method calculates the runoff volume and time to peak whilst assuming a uniform rainfall over whole the catchment area. The formula of this method reads:

$$T_c = \frac{0.87 \times L^2}{1000 \times S^{0.385}}$$  \hspace{1cm} [1]$$

In which

- $L$ = length of the catchment area (388 km);
- $S$ = average slope of the river (0.0032).

Figure B-9 Triangular hydrograph
B.3.3 Comparison of the Data

The data of the UKZN and the UCOSP differ quite a lot. Only one flood peak listed in Figure B-8 exceeds 800 m$^3$/s. As a result it can be concluded that the threshold of the spillway has not been exceeded during the last 17 years. Data from Table B-6 shows that the spillway was used 8 times in the last 10 years. The gauge at the W2H032 site is not able to measure water levels higher than the level corresponding with a runoff of 783 m$^3$/s.

Some inconsistencies exist between the different datasets. This inconsistency has to be sorted in order to obtain reliable input data. A closer look into the data is required to find the discrepancies. Daily 12 minute data series were downloaded from the website of the department of water affairs (Department: Water Affairs, 2011). The result is a table that contains approximately 4,000,000 data points measured over 17 years. These data points are denser and more detailed than the data used by both the UKZN and UCOSP. The differences between Figure B-10 (which contains over 4,000,000 data points) and Figure B-8 are significant. The small flood peaks in Figure B-8 are large and very short flood events. Some are even that short that they were not registered in the daily data. It can be concluded that the results of calculations made with the daily data are an underestimation as compared to reality.
Figure B-10 Flow data based on 12 minutes flow data, gauge W2H032 (Department: Water Affairs, 2011)
B.3.4 Conclusion
A detailed data set is used to deliver more accurate results. Additionally a set of flood peaks must be compiled to model usable runoff data for a (preliminary) design of the channel. This has been done by extracting flood peaks from the dataset from 1995 to 2010. Some flood peaks show cut off trends like the flood peak in Figure B-11. This is because the gauge cannot measure flow rates higher than 783 m$^3$/s. Flood peaks that show this kind of trend were not included in the dataset. From the compiled dataset volumes of the flood peaks have been calculated and plotted against the maximum flow rate in Figure B-12. Based on this figure a rough estimation could be made on the floods that could be retained by the proposed variants without breaching of the berm. The correlation between the flow rate and the volume is quite high when a logarithmic curve fitting is applied (Figure B-12). If it is known which volume of water could be stored (derived from the DEM), a flood peak belonging to this volume can be found via this curve fitting. An example of an extracted flood peak is shown in Figure B-13.
In order to use the theory of the triangular hydrograph (which has been used in section 0) and to compare the compiled dataset with the data of the UKZN the curve fitting through the data points shown in Figure B-12 must be linear and should go through the origin. This curve fitting can be used to make an estimate of the base length and the $T_c$ of the triangular hydrograph. According to Figure B-14 the correlation between the two variables is low. Only a crude estimate of the $T_c$ can be made.

$$\text{Volume} = \frac{1}{2} \times \text{base length} \times \text{flood peak} = \text{linear gradient} \times x$$

$$\text{angle} = \frac{1}{2} \times \text{base length}$$

Based on Figure B-14 the average duration of a flood will be 63 hours. The base length is equal to 3 $T_c$ which results in 21 hours. This is much lower than assumed in section 0.

If the small floods below 500 m$^3$/s are examined the correlation improves drastically as can be seen in Figure B-15. The larger floods are not included in the analysis because the volumes involved with such high peak rates result in breaching anyways. Based on this graph the duration of a flood peak is 85 hours with a corresponding $T_c$ of 28 hours. It becomes evident that the US SCS method is not applicable in the catchment area of the
Mfolozi. The $T_c$ calculated with the US SCS method is too big compared with the real duration of the flood peaks and the calculated $T_c$ in this appendix.

![Graph showing linear fit](image)

*Figure B-15 linear fit of the flow rates lower than 500 m$^3$/s*

**B.3.5 Data for modelling**

Triangular hydrographs based on the range between 0m$^3$/s and 800 m$^3$/s will not be used because of the low correlation between the flow rate and the volume. A triangular hydrograph could be based on the range between 0m$^3$/s and 500m$^3$/s by leaving the higher flood peaks behind. In this research, flood peaks higher than 500m$^3$/s are of less interest because they will breach the berm anyhow. The correlation of the values plotted in Figure B-15 is moderate and one could use this fit to make an estimate for triangular hydrographs.

In order to get a better understanding of the physics and the flood peak details of the Mfolozi a more detailed investigation is required. Considering the scale and timeframe of this research project such a detailed investigation does not fall within the context of the project. However it can be recommended to perform more research on the flood peaks of the Mfolozi.

**B.3.6 Used data for the flood peak analysis**

In the analysis a lot of data has been used. Flood peaks are abstracted from the daily data set. From this flood peak dataset the volume is calculated using a Riemann sum. Figure B-16 till Figure B-36 show the flood peaks that used in the analysis.
Figure B-16 Floodpeak of 01-05-1995; total volume: 44 million m$^3$.

Figure B-17 Floodpeak 02-10-1998; total volume: 99 million m$^3$.

Figure B-18 Floodpeak 14-09-2001; total volume: 34 million m$^3$. 
Figure B-19 Floodpeak 13-02-2002; total volume: 44 million m³

Figure B-20 Floodpeak 19-07-2002; total volume: 112 million m³

Figure B-21 Floodpeak 06-02-2003; total volume: 30 million m³
Figure B-22 Floodpeak 28-11-2003; total volume: 18 million m$^3$

Figure B-23 Floodpeak 22-01-2004; total volume: 140 million m$^3$

Figure B-24 Floodpeak 04-01-2005; total volume: 110 million m$^3$
Figure B-25 Floodpeak 21-02-2005; total volume: 77 million m$^3$

Figure B-26 Floodpeak 25-04-2006; total volume: 19 million m$^3$

Figure B-27 Floodpeak 12-05-2005; total volume: 46 million m$^3$
Figure B-28 Floodpeak 07-11-2005; total volume: 47 million m$^3$

Figure B-29 Floodpeak 20-11-2005; total volume: 23 million m$^3$

Figure B-30 Floodpeak 06-01-2006; total volume: 20 million m$^3$
Figure B-31 Floodpeak 29-06-2007; total volume: 15 million m$^3$

Figure B-32 Floodpeak 10-10-2007; total volume: 35 million m$^3$

Figure B-33 Floodpeak 07-11-2007; total volume: 48 million m$^3$
Figure B-34 Floodpeak 12-02-2009; total volume: 40 million m$^3$

Figure B-35 Floodpeak 01-03-2009; total volume: 178 million m$^3$

Figure B-36 Floodpeak 18-02-2010; total volume: 64 million m$^3$
B.4 Fieldtrip data

Data listed in this paragraph has been obtained during a fieldtrip. The fieldtrip took place on the 10th and 11th of October 2011. Height profiles have been obtained by Real Time Kinetic GPS (TRIMBLE described in subsection B.4.2) and handheld differential GPS (Ashtech described in subsection B.4.3). Flow velocities and the depth of the channel have been measured with Doppler and measuring tape.

B.4.1 Acoustic Doppler

A Teledyne Streampro ADCP raft was used to measure flow velocities and channel depth. The maximum range of the Doppler is 2m and its resolution is 0.1cm/s. The accuracy of the data depends on the operators of the device whom influence the speed at which the raft traverses the channel. Measurements show the bottom is rough. Depth varies between 0.6 and 0.5 meter on the upstream end and between 0.7 and 0.5 metre at the downstream end. The uneven bottom profile is suspected to be created by the excavator that has been used to dig the channel. Depth measurements have been verified by hand. Flow velocities have been verified by the “stick method”.

The data measured with the Doppler device is post processed with WinRiver II (Teledyne RD Instruments, 2011). Figure B-38 and Figure B-45 summarize the cross sections. The cross sections of the old channel are shown in Figure B-39 till Figure B-44. The cross sections of the new channel are shown in Figure B-46 until Figure B-54. Measurements with an error larger than 5% have been excluded.

![Figure B-37 Measuring the waterdepth (Left: measuring tape, Right: Acoustic Doppler)](image)

The depth profiles measured with a measuring tape are shown in Table B-7 These values are used as a validation of the measured Doppler data.

<table>
<thead>
<tr>
<th></th>
<th>depth 1 (cm)</th>
<th>depth 2 (cm)</th>
<th>depth 3 (cm)</th>
<th>average (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inlet</td>
<td>60</td>
<td>67</td>
<td>64</td>
<td>64</td>
</tr>
<tr>
<td>1/4 channel</td>
<td>55</td>
<td>60</td>
<td>61</td>
<td>59</td>
</tr>
<tr>
<td>1/2 channel</td>
<td>70</td>
<td>71</td>
<td></td>
<td>71</td>
</tr>
<tr>
<td>3/4 channel</td>
<td>69</td>
<td>65</td>
<td>69</td>
<td>68</td>
</tr>
<tr>
<td>outlet</td>
<td>28</td>
<td>41</td>
<td></td>
<td>35</td>
</tr>
</tbody>
</table>
Figure B-38 WinRiver II data summary of the new channel
Figure B-39 WinRiver II data 3

Figure B-40 WinRiver II data 4

Figure B-41 WinRiver II data 7
Figure B-42 WinRiver II data 10

Figure B-43 WinRiver II data 11

Figure B-44 WinRiver II data 12
Figure B-45 WinRiver II data summary old channel
Figure B-49 WinRiver II data 17

Figure B-50 WinRiver II data 18

Figure B-51 WinRiver II data 20
Figure B-52 WinRiver II data 21

Figure B-53 WinRiver II data 23

Figure B-54 WinRiver II data 24
B.4.2 Trimble RTK station

Height profiles have been obtained by handheld differential GPS (Ashtech, 2011) and Real Time Kinetic GPS (Trimble). The base station was located at the KZN Ezemvelo office benchmark. Heights have been measured within a 2000 metre range around the trig station. The height of the trig station was set at +18.230m GMSL. The height and location of the trig station is derived from the Mtubatuba trig station. The RTK station was used to measure the water levels of the Mfolozi, St Lucia Estuary, and Back Channel. In addition the area around the New Back Channel was surveyed. The accuracy of the Trimble is 10mm +2ppm. Within 2000 metre from the trig station the accuracy of the device deviates up to 12mm, excluding errors made by the operator. The actual accuracy including operator faults should remain within 2cm. Table B-8 contains the uncorrected data.

Table B–8 Uncorrected Trimble data in WGS84 format

<table>
<thead>
<tr>
<th>Location</th>
<th>Hartbeesthoek 1994 (Lo 33)</th>
<th>WGS84</th>
</tr>
</thead>
<tbody>
<tr>
<td>Location</td>
<td>Easting</td>
<td>Northing</td>
</tr>
<tr>
<td>ROOF</td>
<td>57669.488</td>
<td>3141054.128</td>
</tr>
<tr>
<td>1</td>
<td>56403.261</td>
<td>3141652.476</td>
</tr>
<tr>
<td>CONTROL 1</td>
<td>56397.618</td>
<td>3141647.212</td>
</tr>
<tr>
<td>BERM 1</td>
<td>56358.114</td>
<td>3141660.977</td>
</tr>
<tr>
<td>BERM 2</td>
<td>56352.781</td>
<td>3141661.704</td>
</tr>
<tr>
<td>BERM 3</td>
<td>56340.679</td>
<td>3141663.682</td>
</tr>
<tr>
<td>BERM 4</td>
<td>56330.194</td>
<td>3141666.304</td>
</tr>
<tr>
<td>BERM 5</td>
<td>56318.491</td>
<td>3141662.758</td>
</tr>
<tr>
<td>BERM 6</td>
<td>56288.963</td>
<td>3141673.593</td>
</tr>
<tr>
<td>BERM 7</td>
<td>56273.891</td>
<td>3141683.773</td>
</tr>
<tr>
<td>STLUCIA1</td>
<td>56558.499</td>
<td>3141274.762</td>
</tr>
<tr>
<td>STLUCIA2</td>
<td>56560.891</td>
<td>3141272.416</td>
</tr>
<tr>
<td>PIN</td>
<td>56846.381</td>
<td>3141506.620</td>
</tr>
<tr>
<td>CHANNELNEW1</td>
<td>57907.432</td>
<td>3141978.150</td>
</tr>
<tr>
<td>CHANNELNEW2</td>
<td>57909.924</td>
<td>3141973.435</td>
</tr>
<tr>
<td>CHANNELNEW3</td>
<td>57924.551</td>
<td>3141952.582</td>
</tr>
<tr>
<td>CHANNELBOTTONEW1</td>
<td>57926.359</td>
<td>3141955.811</td>
</tr>
<tr>
<td>CHANNELBOTTONEW2</td>
<td>57923.772</td>
<td>3141958.886</td>
</tr>
<tr>
<td>CHANNELBOTTONEW3</td>
<td>57932.703</td>
<td>3141952.110</td>
</tr>
<tr>
<td>CHANNELBOTTONEW4</td>
<td>57937.835</td>
<td>3141947.275</td>
</tr>
<tr>
<td>CHANNELBOTTONEW5</td>
<td>57937.877</td>
<td>3141947.312</td>
</tr>
</tbody>
</table>
Figure B-55 Uncorrected water levels

Figure B-56 Uncorrected heights near the Back Channel (Google maps overlay)

Figure B-57 Uncorrected heights measured with the Trimble (Google maps overlay)
According to Stretch (2011) measurements by Chrystal indicate the level of the roof should be +18.03m GMSL as opposed to +18.23m GMSL. The reason of this +0.2m error is not clear. The Mtubatuba trig station might be wrong. A gravitational abnormality in the area might affect GPS results. The +0.2m was derived from a test using Ashtech devices and a canoe. Using the canoe with GPS devices the tidal motion of the ocean was measured. Comparing this tidal data with the tidal data from Richards Bay returned the Mean Sea Level (MSL). After transforming MSL into GMSL a +0.2m discrepancy was found with respect to the roof. Various other trig stations and benchmarks in the area also confirm a discrepancy exists. The size of the error varies between the stations. The height derived from MSL is the most accurate height available in the area, given the unknown degree of inaccuracy of the height at the other locations. GPS data has been corrected by -0.2m. In addition corrected GPS data is more consistent with the Cotcane values found in Appendix B.2. Due to the large inconsistencies in the area it is unknown whether this is just “luck” or reality. Figure B-59 shows the corrected water levels.

Figure B-58 Trimble base station setup on two locations.

Figure B-59 Corrected waterlevels
B.4.3 Ashtech
Ashtech differential GSP devices have been used as backup for the Trimble GPS. Ashtech handheld devices require post processing and are not real time kinetic. The purpose of the Ashtech devices was offering a way to check and verify data obtained by the Trimble station. The base station was located at the KZN Ezemvelo office benchmark. Obtained data proved to be highly unreliable. Large inconsistencies were found in height levels. Inconsistencies are blamed on faulty equipment.

Figure B-60 Measuring with DGPS Ashtech during the fieldtrip.

B.4.4 Soil parameters
Data has been collected in different ways. The following section treats the shear vane tests performed. An indication of the undrained shear strength is derived from these tests. In addition the tests performed on the taken soil samples are discussed.

Shear vane test
Shear vane tests have been performed to get an idea of the undrained shear strength of the soil. A Humboldt H-4227 vane inspection kit was used with the largest shear vane. Results had to be divided by two and multiplied by ten to get the undrained shear strength of the soil (in kPa). This test is only effective in cohesive soil.

The soil at the project location is a combination of riverine settlements and dredge spoil from the St Lucia estuary. The riverine settlements are cohesive in nature and consist mainly of clay and silt. The dredge spoil is granular in nature and consist mainly of fine sand. The soil was suspected to be cohesive enough for the shear vane tests.

Tests have been performed at eight locations. Six tests have been performed at a depth of 10 cm and two at a depth of 1 metre. Of the six shallow tests two were performed at the water level of the newly excavated channel and four were performed at ground level next to the newly excavated channel. Of the deeper tests one was performed at the water level and one was performed at the bottom of the channel.

Every test consists of four evenly spread readings over 1 m² from which the average is taken. The maximum reading obtained was 65 kPa. Null readings are not taken into account for the calculation of the average. The results are listed in Table B-9.

The results obtained at the water level show a lot of null readings. Possible causes of the null readings at a depth of 0.1 m are:

- Sand or silt in the subsoil;
- Weak behaviour of the subsoil submerged;
- Numerous crab holes, and
- Recently stirred up sediment during the excavation of the new channel.
Table B-9 Results shear vane test

<table>
<thead>
<tr>
<th>Locations</th>
<th>Depth [m]</th>
<th>Reading 1 [kPa]</th>
<th>Reading 2 [kPa]</th>
<th>Reading 3 [kPa]</th>
<th>Reading 4 [kPa]</th>
<th>Average [kPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water level halfway the newly excavated channel</td>
<td>0.1</td>
<td>20.0</td>
<td>12.0</td>
<td>0</td>
<td>5</td>
<td>12.3</td>
</tr>
<tr>
<td>Water level, at outlet newly excavated channel</td>
<td>0.1</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>6.0</td>
<td>6.0</td>
</tr>
<tr>
<td>Ground level, directly next to newly excavated channel</td>
<td>0.1</td>
<td>18.0</td>
<td>20.0</td>
<td>13.0</td>
<td>34.0</td>
<td>21.3</td>
</tr>
<tr>
<td>Ground level, near vegetation 1</td>
<td>0.1</td>
<td>31.0</td>
<td>44.0</td>
<td>33.0</td>
<td>35.0</td>
<td>35.8</td>
</tr>
<tr>
<td>Ground level, near vegetation 2</td>
<td>0.1</td>
<td>37.0</td>
<td>53.0</td>
<td>31.0</td>
<td>43.0</td>
<td>41.0</td>
</tr>
<tr>
<td>Ground level, near vegetation 3</td>
<td>0.1</td>
<td>&gt;65.0</td>
<td>&gt;65.0</td>
<td>&gt;65.0</td>
<td>&gt;65.0</td>
<td>&gt;65.0</td>
</tr>
<tr>
<td>Water level halfway the newly excavated channel</td>
<td>1.0</td>
<td>10.0</td>
<td>1.0</td>
<td>0</td>
<td>0</td>
<td>5.5</td>
</tr>
<tr>
<td>Bottom of channel halfway the newly excavated channel</td>
<td>1.0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

Only a few crab holes may occur at a depth of 1 metre below ground level. It is assumed the readings at 1 metre depth are little influenced by the recent excavation. Based on the results of the shear vane tests it is assumed the soil consist of a combination of sand/ silt and clay. The soil behaves very weak when submerged. The crab holes will not have a large influence on long term settlement. The holes will most probably collapse under the load of the weir.

**Soil samples**

Two tests have been performed on the taken soil samples. The first test resulted in the specific density of soil. The second test was a sieve test performed on the granular part of one of the two soil samples. The soil samples have been taken at the slope of the newly excavated channel near the water level. One soil sample consisted of clayey material with some sandy areas. The second sample consisted of a combination of clayey and sandy material. The distribution of both materials was much coarser in the second sample than in the first sample. Figure B-61 shows the dried leftovers of both samples. The left picture shows clumps of clay. The right picture shows the coarse distribution clay and sand.

Figure B-61 Dried leftovers of soil samples. Left: clayey sample. Right: sandy sample.
**Specific density**
From both soil samples a test sample was taken for the determination of the specific density. The volume of the test samples was measured before they were dried. The weight of the test samples has been measured before and after they were dried. The test samples were dried overnight at a temperature of 108°C. The results are given in Table B-10.

*Table B-10 Results specific density measurement*

<table>
<thead>
<tr>
<th>Test ring number</th>
<th>Volume $[10^{-6} \text{ m}^3]$</th>
<th>Weight, wet $[10^{-3} \text{ kN}]$</th>
<th>Specific density, wet $[\text{kN/m}^3]$</th>
<th>Weight, dry $[10^{-3} \text{ kN}]$</th>
<th>Specific density, dry $[\text{kN/m}^3]$</th>
</tr>
</thead>
<tbody>
<tr>
<td>6 (Clayey)</td>
<td>125.66</td>
<td>2.14</td>
<td>17.03</td>
<td>1.50</td>
<td>11.94</td>
</tr>
<tr>
<td>8 (Sandy)</td>
<td>129.38</td>
<td>2.32</td>
<td>17.93</td>
<td>1.57</td>
<td>12.10</td>
</tr>
</tbody>
</table>

*Figure B-62 Dried crushed test samples. Left: clayey test sample. Right: sandy test sample.*

**Sieving granular material**
To get an idea of the composition the sandy part of the sandy soil sample a sieve test has been performed. First the test sample has been crushed by hand. Second the coarse clay clumps have been removed with a coarse sieve. After this the remaining sand was led through sieves with a mesh size of 425, 300, 150, 75 and 63 µm respectively. Figure B-63 shows most of the sand falls within the range of 150 to 300 µm followed by the range of 75 to 150 µm. The particles smaller than 63 µm are a mix of clay and silt particles according to the soil classification system of the United States Department of Agriculture (USDA).
Figure B-63 Sieved sample, from left to right and top to bottom: 425, 300, 150, 75, 63 and <63 µm.

B.5 Flow velocity and erosion

The allowed flow velocity with respect to erosion follows from empirical graphs. Values are empirical which means they are somewhat accurate given the conditions match the conditions of the test on which they are based. On average results are fairly consistent with reality however it is important to stay cautious.

Figure B-64 Allowable flow velocities. On the left the soil properties. From top to bottom: sand, peat, sanded clay, weak clay, reasonable stiff clay and stiff clay. On the right: allowable flow velocities. “Bij kortdurende belastingen” means: during short term events.
Many research has been performed on the storage capacity of the Mfolozi and Msunduze Floodplains. First the hypsometric curve as proposed by Kelbe & Taylor (2011) will be discussed. Secondly the unpublished hypsometric curve as proposed by Chrystal and Stretch will be elaborated.

B.6.1 Hypsometric curve as proposed by Kelbe & Taylor (2011)
The following has been used to compute the bathymetry:
- 5 and 10m elevation contour data of the National Department of Survey
  - Covers whole the floodplain area.
- Raster elevation data made by the Shuttle Radar Topography Mission (SRTM) by NASA
  - Vertical resolution is 6 metre
  - Grid size is 90x90 metre
  - Radar bounces back on the tops of the threes
  - Covers the entire floodplain area.
- Point source data from Google Earth
  - Finer spatial resolution than SRTM
- Covers the entire floodplain area
- High resolution data from Light Detection and Ranging (LiDAR) technology
  - Vertical resolution of 24cm under pine land cover
  - Some radiation penetrates the leaves.
  - Only coverage on the dunes.

The data obtained from the National Department of Survey, SRTM, Google Earth and LiDAR have been combined to cover the entire floodplain area. The result of this survey is shown in Figure B-70.

![Figure B-66 DEM based on SRTM, Google Earth and LiDAR data.](image)

The DEM data has been used to make the graphs as shown in Figure B-67 and Figure B-68. The volume of the stored water increases exponentially as the water level rises. During rising water the flooded area increases and above 0.5 MSL the gradient becomes more gentle as shown in Figure B-68. A gentler gradient means the area of flooding compared to water level rise is lower than with a steeper gradient.

![Figure B-67 Plot of the depth above MSL (stage) and the volume of the swamp area. The yellow line shows the difference between the linear regression and the volume derived from the DEM. (Kelbe & Taylor, 2011)](image)
B.6.2 Unpublished hypsometric curve as proposed by Chrystal & Stretch

The hypsometric graph as proposed by Chrystal & Stretch (unpublished) has been deduced from field measurements with D-GPS devices. The area that was not covered by LiDAR has been mapped using a triangular grid system. A GPS base station located at the office of KZN wildlife was used to increase the accuracy of the measurements. The line in Figure B-69 shows the distinction between the LiDAR (right) and the triangular DGPS data (left). The DEM obtained has been used to deduce the volume and area graphs of Figure B-70.

Figure B-68 Stage-Area function of the floodplain area (Kelbe & Taylor, 2011).

Figure B-69 Triangular DEM
Figure B-70 shows that at first the submerged area remains quite constant during rising water. As soon as the Mfolozi breaks its banks the floodplains become submerged. The floodplains are quite flat compared to the Mfolozi resulting in a rapid increase of the flooded area. As soon as the river breaks its banks at approximately 1.2 metre GMSL both storage and volume will increase substantial.

B.6.3 Discussion

Only one hypsometric curve can be used in the model. Both hypsometric curves do have advantages and disadvantages. The data of Kelbe & Taylor (2011) is based on a lot of data from different sources which might improve accuracy if joined and checked correctly. The resulting hypsometric curve is doubtful given the steep increase in area during the first few decimeters of flooding. This would mean the Mfolozi immediately breaks its banks and starts flooding the floodplains, as opposed to an initial filling of the river channel until the levees are overtopped.

If one compares the graphs of Figure B-67 and Figure B-68 (Kelbe & Taylor, 2011) with the graph of Figure B-70 (Chrystal & Stretch, 2010) a large difference becomes obvious. The flooded area in Figure B-68 increases a lot in the first phase and flattens in the second phase. The flooded area in Figure B-70 stays quite constant in the first phase, increases substantially during the second phase and flattens a bit in the third phase. Also the differences between Figure B-67 and Figure B-70 are quite large. The volume plotted in Figure B-67 increases almost linearly with the water level. This would imply the floodplain is almost flat. The volume plotted in Figure B-70 remains quite constant until the levees of the Mfolozi are engulfed. After overtopping the volume grows exponential with the water level.

Following the physics behind flooding the hypsometric curve proposed by Chrystal & Stretch is more plausible than the hypsometric curve proposed Kelbe & Taylor (2011). The hypsometric curve as proposed by Chrystal & Stretch will be used in the model.
B.7 Water balance error

When combining the data from the W2H032 gauge, fieldtrip and Cotcane water levels a water balance error is found. According to the Cotcane water levels behind the berm were fairly stable in the period of 11th till the 20th of October 2011. During this period the average discharge of the Mfolozi amounted to 7m$^3$/s (Department: Water Affairs, 2011), whilst the outflow of the Back Channel was 2 m$^3$/s. Even when taking factors like seepage through the berm and evaporation into account the difference between the inflow and outflow remains significant. In a stable situation, as measured by the Cotcane gauge the inflow and outflow should be equal. The cause of this water balance error is unknown. Possible reasons are:
- Incorrect water level measurement at W2H032 gauge near Monzi.
- Incorrect cross section used in discharge computations of W2H032 gauge near Monzi.
- Poorly calibrated or malfunctioning Cotcane gauge.
- Unexpected high seepage through the Mfolozi berm.
- Evapotranspiration by vegetation.
- Extraction of water by sugarcane farmers.

It is important to note that this inconsistency is present. It goes beyond the scope and power of this project to determine the real cause.
C. STAKEHOLDER ANALYSIS

The stakeholder analysis is based on document analysis, interviews, and common sense. In this chapter the reasoning of the stakeholders will be explained. For this project sixteen of the most relevant stakeholders have been distinguished. Of course there are many more stakeholders involved, but these are not described below. There has been attempted to take these other stakeholders’ interests into account through one of the sixteen described stakeholders below.

C.1 Government of South Africa

The government of South Africa has an indirect decision-making role in the process. It can be seen as the top layer management of the iSimangaliso Wetland Park Authority. In 1971 the South African government signed the Ramsar Convention to protect and conserve a number of wetlands of international importance because of their large number of species and unique ecology. The government controls the overall management of the UNESCO World Heritage sites in South Africa. In 1999 the Greater St Lucia Wetland Park (nowadays known as iSimangaliso Wetland Park) was declared as an UNESCO World Heritage Site. The government designated the Greater St Lucia Wetland Park Authority with the legislative management part. The executive part was designated to Ezemvelo KZN Wildlife. Together with the UNESCO declaration, the South African government adopted the World Heritage Convention Act, 1999 to provide the Park authority with the necessary legal protection. The government reports directly to UNESCO. Once every five year a large “check” on the requirements of the UNESCO World Heritage Site takes place.

In addition, next to the interests in UNESCO/Ramsar World Heritage sites the government attaches value to the wellbeing of the inhabitants in KwaZulu-Natal (KZN). This area is also known as the 2nd poorest province and comprises about 20% of the total population of South Africa (James & Govender, 2011). Currently, ecology tourism is more and more important for the functioning of the economy of KwaZulu-Natal. Therefore the World UNESCO/Ramsar status is an important attractor for tourism. Deterioration of tourism attractions and losing the UNESCO/Ramsar status would increase the risk of an economic decline.

The department of water affairs is the main related department op the South African government. Their goal is to ensure that all South Africans gain access to clean water and safe sanitation, the water sector also promotes effective and efficient water resources management to ensure sustainable economic and social development. (Departement of Water Affairs, 2011)

The governments’ power is related to financial resources, legislation power, and decision-making power. They are considered as a strong indirect related party that supports project developments to protect the values of the iSimangaliso Wetland Park through the iSimangaliso Wetland Park Authority.

C.2 iSimangaliso Wetland Park Authority

The iSimangaliso Wetland Park Authority is the legislative management part of the iSimangaliso Wetland Park. The iSimangaliso Authority was set up to manage the Park, created from 16 different parcels of land. These lands were divided in state-owned land, commercial forests and former military sites. (iSimangaliso Wetland Park Authority, 2011)

Their interests are divided in a couple aspects. Overall park management is the most important aspect. In the end they are responsible for the current status and activities of the iSimangaliso Wetland Park. The iSimangaliso Authority reports to the national Minister of Environmental Affairs and Tourism. Also, the iSimangaliso Authority reports directly to the...
government for the evaluation of the UNESCO requirements. This happens about two times a year. Small things that are relevant are reported to UNESCO directly. Secondly, they exploit the park operations activities, the commerce unit, the research unit, and training and capacity building unit in an integral way. These are organised in a matrix like organisation. Thirdly, another interest is the wellbeing of inhabitants of the area and the local economies. Finally the last mentioned important interest is the nature conservation and development. Park establishment programs have seen the removal of some 12 000 ha of alien plants and commercial forests. Wetland and dune rehabilitation programs, the introduction of game, runway upgrades, the building of new roads, game fences, new water supply and bulk electricity supply systems and substations have all contributed to building the new Park. (iSimangaliso Wetland Park Authority, 2011)

The main problem perception is the social, environmental, and economic impacts through large desiccation and human interventions in the iSimangaliso Wetlands Park and finding a common interest between commercial activities and nature conservation. As a result of the large amount of stakeholders it is hard to manage the park such a way that there is a win-win situation for all involved parties. Human interventions at the Mfolozi floodplains and earlier "mismanagement" of the mouth dynamics resulted in major impacts on the ecological situation of the iSimangaliso Wetland Park.

Therefore the goal of the iSimangaliso Wetland Park Authority is to improve biodiversity conservation, hydrology and ecosystem functioning of the iSimangaliso Wetland Park by promoting conservation-compatible local economic and cultural developments. Biodiversity conservation could be stimulated through institutional capacity building through effective and efficient governance, and the strongest possible legislative, regulatory and institutional arrangements (James & Govender, 2011).

The Park authorities’ power is related to financial resources from the government and the GEF project fund, decision-making power, knowledge, and landownership. They have the overall management control and have the power to force a decision. A lot of knowledge has been regained along the years and the Park Authority “owns” a lot of land. On the western shores there has been no land buy, but only agreements. Buying up lands is expensive. However, a large part of the Mfolozi floodplains lie below the 100 year flood line. This “100 year flood line” area is part of the government. However, due to political problems and complex “land ownerships” these lands cannot be considered as properties of the state.

The Park Authorities are considered as a strong direct related party (principal) that supports project developments to protect the values of the iSimangaliso Wetland Park.

C.3 Ezemvelo KZN Wildlife

The Ezemvelo KZN wildlife is the executive part of the iSimangaliso Wetlands Park and falls under the authority of the iSimangaliso Wetland Park Authority. They are mandated to manage nature/biodiversity conservation within the province of KwaZulu-Natal. In addition they are the successor to two agencies with fifty years of experience in managing nature conservation in the combined province (iSimangaliso Wetland Park Authority, 2009). Their main goal is the conservation of the iSimangaliso Wetland Park through day-to-day management of the wildlife and natural systems of the Park. In addition their interests lie in ecotourism operations and “To be a world renowned leader in the field of biodiversity conservation” (Ezemvelo KZN Wildlife, 2011).

The Ezemvelo KZN Wildlife is a public entity that is primarily accountable to the KwaZulu-Natal Provincial Government and ultimately to the communities of KwaZulu-Natal. The
Climate Action Partnership (CAP) describes such accountability as a two-fold responsibility, namely:

- "EKZNW business must be discharged within the legislative framework.
- Legislative compliance must not be an end in itself, but a means to sustainable biodiversity conservation which provides a social, economic, and environmental development platform."

Like the Park Authorities, their problem perception focuses on KwaZulu Natal's biodiversity through desiccation of the iSimangaliso Wetlands Park and through human interventions in the mouth dynamics. However, there are some conflicts with local communities and the park authorities. In earlier days, there have been many conflicts with local communities about land reclamation. These activities did not always happen without violence. These historical events obstructed the current process. Especially the trust relationship between local communities and KZN could be harmed. In addition, there are also some discussions with the park authorities on the impacts of a combined Mfolozi – St Lucia estuary mouth.

The main goal of the Ezemvelo KZN Wildlife is the biological conservation. Conservation stands for "the rational and prudent management of biological resources to achieve the greatest sustainable current benefit while maintaining the potential of the resources to meet the needs of future generations." (Ezemvelo KZN Wildlife, 2011). In addition they want to exploit eco-tourism operations.

Along the years there has been built up an incredible amount of knowledge about the ecological system. A lot of research has been done. In addition they have a great amount of supporting/blocking power in relation to interventions of the iSimangaliso Wetland Park authority. However, the latter has the final word. They are considered to be a directly involved stakeholder with a lot of important resources. Currently they are working in close interaction to find a most optimal solution for the long and short term considering a sufficient amount of fresh water inflow without too much sediment transport into the St Lucia system.

C.4 Municipality of Mtubatuba

St Lucia falls under the district of the municipality of Mtubatuba. Their vision as described on their website is: "Mtubatuba – with its people, for its people – the vibrant, prosperous eco-tourism and development Heartland of Umkhanyakude, KwaZulu-Natal." (Municipality of Mtubatuba, 2011). The ecotourism branch is of great importance to them and extends beyond the municipal area into adjoining municipalities, formal game and nature reserves and private game lodges. Besides that, the largest employment sector is the agricultural sector. The focus lies mainly on timber and sugarcane production. In order to keep the tourism sector running the well-functioning of the Wetland Park is crucial.

The problem perception of the municipality of Mtubatuba is twofold. It can be described through social, environmental and economic impacts through desiccation and human interventions on the iSimangaliso Wetland Park. On the other hand, the industries on the floodplains could suffer from solutions to stimulate the ecology of the Park. These conflicting interests make it a complex problem for the municipality of Mtubatuba.

Their goal is on the one hand to protect the iSimangaliso Wetland Park from deterioration and on the other hand to stimulate the important sugarcane industry. Next to that they want to protect St Lucia from flooding as well. Mtubatuba would probably support a win-win solution and is considered to be a supportive player in the process.
C.5 Umfolozi Sugar Mill (Pty) Ltd (USM)

Umfolozi Sugar Mill (Pty) Ltd (USM) is representing the agro-processor. One of their main interests is making profit. In addition they provide a lot of work for the local communities, but profit is concerned to be their main objective. This latter is based on the fact that there is a lot of money involved in the sugarcane industry.

In order to keep the sugar mill running as best as possible the supply of sugarcane has to be guaranteed. Their problem perception concerns mainly the low availability of sugarcane supply as a result of interventions in the Mfolozi floodplains’ water system.

Their goal is focussed on the creation of appropriate win/win sustainable mitigation measures. Due to the pressure that something has to happen they probably are standing open for win-win solutions. In the end their main goal is to sustain/improve their profits. The sugarcane industry (sugar mill) is regarded to be a powerful stakeholder as a result of their economic relevance. However, depending on the stakeholder “game” they can support or block the process. At the moment they are indirectly involved through the UCOSP that represents them. In addition they are considered not to be supportive to implement changes in the area that have a negative impact on the sugarcane industry.

C.6 UCOSP (Pty) Ltd

The UCOSP, better known as Umfolozi Sugar Planters Ltd, are considered to be an important party. Their main interests are providing infrastructures and flood protection infrastructures on behalf of the sugar cane farmers. The sugarcane farmers on the floodplains provide almost 70% of the sugar mill supply. They also provide a lot of work in the area. However, many sugarcane farms are lying below the ‘100 year flood line’. This means that these lands can be reclaimed by the government. However, since 2008 the government provided farm lands as part of a poverty relief initiative. In addition, the landownership is quite complex due to a lot of political reasons. Also local historically disadvantaged farmers received lands on the lower flood plains. (Collings, 2009)

Indirectly they are interested in making profits from plantations and sugarcane industry. However, their main concerns are indirectly about the height of the Mfolozi sand berm near sea. When the water level behind this berm reaches a level of 1,50m above GMSL, low lying farm lands are exposed to flooding. Even at a level of 0,9m above GMSL the farmers on the lower parts want to breach the mouth to drain their fields. Along the years the Mfolozi mouth has been kept open for quite some time and stimulated farmers to farm on the lower parts of the fertile floodplains as well. However, the risk of flooding is much higher over there. These floods are a result of both rainfall and eventually the backup of water in the Mfolozi. When the mouth is closed the water cannot flush away from the fields. So eventually there main interest is probably to find a way to prevent the Mfolozi from backing up too much. In addition, they do belief that something has to happen to prevent deterioration in the St Lucia estuary, Narrows, and Lakes. (De Jager, 2011) However, the consideration between a fresh inflow of water versus lower water levels in the Mfolozi remains very complex.

Summed up, their main goal is to ensure the exploiting activities of farming. Next to that they participate in the process to find a win-win solution. They are considered to have a limited amount of supporting/blocking power. On the one hand they contribute a lot to the local economy and could hamper the process. The UCOSP’s attitude is considered to be a supportive one, but could change if they are not satisfied with the outcome of the process. Their support is important to foster the process.
C.7 Inhabitants of St Lucia

The inhabitants in St Lucia are indirectly involved in the process. They do not have much power to influence decisions or to hamper the process. However, they are directly affected by changes in the environment. Deterioration would hamper tourism incomes and fishery. On other hand too much water in the Estuary would lead to an increasing flood risk.

The problem perception varies per citizen. Some say that the long droughts and desiccation are the results of the low water levels in the St Lucia lakes. Others think that the connection with sea has to be established, and some say that the connection with the Mfolozi has to be established.

Their goal is to make profit out of tourism and consolidate their living circumstances by protecting the well-functioning of the iSimangaliso Wetland Park for tourism and fishery. They are considered as a neutral party with limited influence on the process.

C.8 Surrounding municipalities

The surrounding municipalities are indirectly involved in this project. They are affected by the impacts of the functioning of the St Lucia estuary, narrows, and lakes. Desiccation and salinization in the lakes have a negative impact on the surrounding environments as well. Fertile grounds and fresh water could become scarce and the iSimangaliso Wetland Park boundaries increase. The main interest of the surrounding municipalities is probably providing “better” living circumstances and future opportunities for their inhabitants.

On the one hand by supporting the developments of the iSimangaliso Wetland Park the chance of increasing developments regarding health, education, and living becomes higher. Therefore their problem perception is related to the functioning of the Wetland Park. Desiccation has a negative impact on the available fertile grounds, the amount of fresh water, and the well-functioning of the Wetland Parks’ habitat and wildlife. However, an increase of tourism would lead to a change in making money. When the latter is not preferred by the inhabitants the increasing boundaries on fertile grounds of the iSimangaliso Wetland Park could conflict with the interests of the inhabitants.

Their goal is to provide job opportunities for local inhabitants to make some money and survive. In addition they strive for better living circumstances, health conditions, and educational opportunities. They are not considered as a powerful stakeholder, but are assumed to be to some extent supportive.

C.9 Local communities

Local communities are not considered to be part of the surrounding municipalities. The local communities are very poor, living very primitive, and support any way of making money/food. Their main interest is surviving.

Many of the local communities do not have much information regarding the main problems and there causes. All they are considered with is surviving in their surroundings, whether or not it is in a fenced park.

Their main goal is to make some money or food to survive. They are not considered as important stakeholders, but they can obstruct the process and cause damage on the boundaries of the park. By demolishing fences they let their animals graze on the lands of the iSimangaliso Wetland Park. A lack of knowledge could lead to the assumption that they are not easily willing to change their customs.
C.10 Universities

The Universities work as independent organizations for many involved stakeholders, like the iSimangaliso Wetland Park authority, WRC, CERM, UCOSP, and Ezemvelo KZN Wildlife. Their main interest is gathering knowledge through research to find out what the different impacts of human and natural interventions on habitats and wildlife in the iSimangaliso Wetland Park are. These interventions are related to the dynamic mouth conditions of the Mfolozi and the St Lucia estuary, the desiccation and salinization, and sugarcane activities on the Mfolozi floodplain.

Their problem perception is that there is insufficient knowledge about the impacts of human and/or natural changes in the iSimangaliso Wetland Park. As a result some of the management decisions that have been taken are not all or not enough based on scientific grounds.

Their goal is to foster awareness and commitment from governments, the public, and decision makers at large to take action, and provide adequate knowledge/research resources to do so in a scientific correct way. They are considered as a neutral advising party with a lot of knowledge. Through scientific research they could support and/or influence the decision-making process of the project. Awareness should be on place because they could be influenced by their client.

C.11 Global Environment Facility

The Global Environment Facility (GEF) granted funds for the sustainable development of the iSimangaliso wetlands and its surroundings. Their interest are concerned with the investments in projects for improvements of biodiversity, land degradation, and international waters and reduction of negative impacts regarding climate change, ozone layer, and persistent organic pollutants. (Global Environment Facility, 2011)

Their problem perception is the lack of available funds to conserve the iSimangaliso Wetland Park ecosystem functioning.

Their goal is to improve the ecosystems functioning that address global environmental issues. By providing money they want to maintain the availability of fresh water of adequate quality to the Lake St Lucia System. In addition they want to increase access among local communities to conservation compatible economic opportunities. (Barbut, 2009)

The GEF is considered as an indirect supportive party with a lot of financial resources. They are represented by the iSimangaliso Wetland Park Authority, which also manage the GEF project.

C.12 UNESCO

United, Nations, Educational, Scientific, and Cultural Organization (UNESCO) strive for global visions of sustainable development encompassing observance of human rights, mutual respect and the alleviation of poverty. Their interests lie in encouraging the identification, protection and preservation of cultural and natural heritages around the world that are considered to be of outstanding value to humanity. (UNESCO World Heritage, 2011)

Since 1999 the iSimangaliso Wetland Park is known as a UNESCO World Heritage site. Their problem perception regarding this project could be the degradation of the ecosystems in the iSimangaliso Wetland Park due to human influences.
UNESCO’s mission is “to contribute to the building of peace, the eradication of poverty, sustainable development and intercultural dialogue through education, the sciences, culture, communication and information” (UNESCO World Heritage, 2011).

They are considered as a neutral, indirectly involved, party with administrative power and some limited financial resources. As administrative resource the UNESCO is providing technical assistance and professional training. In addition they support States Parties’ public awareness-building activities for World Heritage conservation. The limited financial resource provides emergency assistance for World Heritage sites in immediate danger.

In certain extent they support the developments to prevent the iSimangaliso Wetland Park from deterioration and salinization. In addition, if the iSimangaliso Wetland Park does not comply with the UNESCO requirements it could lose its World Heritage status.

C.13 RAMSAR

“The Convention on Wetlands (Ramsar, Iran, 1971) -- called the "Ramsar Convention" -- is an intergovernmental treaty that embodies the commitments of its member countries to maintain the ecological character of their Wetlands of International Importance and to plan for the "wise use", or sustainable use, of all of the wetlands in their territories.” (Ramsar, 2011)

Their problem perception is the degradation of the iSimangaliso wetlands due to the impacts of human interventions.

Their goal is to support the maintenance of the ecological character of the wetland by stimulating the conservation and wise use of all Wetland. Local and national actions and international cooperation is needed to achieve sustainable development of Wetland throughout the world. (Ramsar, 2011)

Like the UNESCO they are considered as a neutral, indirectly involved, party with administrative power. If the iSimangaliso Wetland Park does not comply with the Ramsar requirements it could lose its status of Wetland of international importance.

C.14 Mining companies

The mining companies are not directly involved in the project, but are considered as an opposite party. Their main interest is making profit out of mining activities. At the moment mining activities are forbidden along the coast line of the iSimangaliso Wetland Park. One of the main problems in the viewpoint of these mining companies is the international UNESCO and Ramsar site status that make mining activities impossible in the area.

Their goal is to make profit, but at the moment they are not considered as an important and influential party. However, if deterioration of the Wetland Park continues and other forms of investment would be unstoppable, the mining companies could come into play.

C.15 Water Research Commission

The Water Research Commission develops and supports a water-related knowledge base in South Africa. Their main interests lie within the current understanding of the system, with particular emphasis on information required to reconnect the Mfolozi to the St Lucia system.

Their problem perception is the threat of a lack of sufficient water, while water quality and availability issues are becoming more acute in the country.
By generating new knowledge and by promoting the countries water research the WRC tries to combine and order all available knowledge of the water systems.

They are considered to be indirectly involved in the project as a supportive stakeholder. They stimulate research in the iSimangaliso Wetland Park through funding.

C.16 **Consortium for Estuarine Research and Management**

The Consortium for Estuarine Research and Management (CERM) brought researchers together to combine information regarding the Mfolozi – Msunduze estuarine system. They are interested in its process and dynamics, both biotic and physical components.

Their problem perception is concerned with the knowledge gaps in the complex estuarine processes at ecosystem levels. Until now, decisions regarding this estuarine system have not proven their worth.

Therefore the CERM promotes joint participation in directed research, training, and technology transfer to foster the sustainability of the St Lucia system and to produce a WRC report summarizing our current understanding of the system. They are regarded as a supportive stakeholder with a big network and knowledge.
### C.17 Stakeholder analysis table

<table>
<thead>
<tr>
<th>Actor</th>
<th>Interests</th>
<th>Problem perception</th>
<th>Goals</th>
<th>Important resources</th>
<th>Critical Actor</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Government of South Africa (Department of water affairs)</strong></td>
<td>Welbeing of the inhabitants in KwaZulu Natal and the protection of the World Heritage and RAMSAR site</td>
<td>Loosing the UNESCO world heritage and Ramsar site status in the iSimangaliso Wetland Park</td>
<td>To provide resources nessecary to protect the natural UNESCO and RAMSAR sites; Ensure that all South Africans gain access to clean water and safe sanitation</td>
<td>Financial resources, legalisation power, decision power</td>
<td>YES</td>
</tr>
<tr>
<td><strong>iSimangaliso Wetland Park Authority</strong></td>
<td>Overall park management; Exploitation of the activities of park operations, the commerce unit, the research unit, and training and capacity building in an integral way; Welbeing of inhabitants of the area; Economic growth; Nature conservation; Stimulating local activities and economies</td>
<td>Social, environmental, and economic impacts through large desiccation and human interventions in the iSimangaliso Wetland Park; Finding a common interest between commercial activities and nature conservation</td>
<td>To improve biodiversity Conservation, Hydrology and Ecosystem Functioning of the iSimangaliso Wetland Park; Promoting conservation-compatible local economic and cultural development; Institutional capacity building for biodiversity conservation</td>
<td>Decision power, financial resources, knowledge, landownership</td>
<td>YES</td>
</tr>
<tr>
<td><strong>Ezemvelo KZN Wildlife</strong></td>
<td>“To be a world renowned leader in the field of biodiversity conservation”; Eco-tourism operations; Maintenance of the iSimangaliso Wetland park.</td>
<td>Problems with KwaZulu Natal’s biodiversity through desiccation of the iSimangaliso Wetland Park and through human interventions in the mouth dynamics.</td>
<td>The rational and prudent management of biological resources to achieve the greatest sustainable current benefit while maintaining the potential of the resources to meet the needs of future generations. To exploit Eco-tourism operations.</td>
<td>Knowledge, supporting power</td>
<td>YES</td>
</tr>
<tr>
<td><strong>Municipality of Mtubatuba (Including St Lucia district)</strong></td>
<td>An attractive image of tourist towns for visitors in the iSimangaliso Wetland park; The well functioning of the St Lucia estuary and lakes and of the sugarcane and timber industries; Floodprotection of the inhabitants of St Lucia;</td>
<td>Social, environmental and economic impacts through desiccation of the iSimangaliso Wetland Park</td>
<td>Finding a balance between the protection of the St Lucia estuary and lakes and the protection of the sugarcane industry; Protecting the inhabitants of St Lucia against floods.</td>
<td>Diffuse power</td>
<td>YES</td>
</tr>
<tr>
<td><strong>Umfolozi Sugar Mill (Pty) Ltd (USM) (representing the agro-processor)</strong></td>
<td>Profit</td>
<td>Low availability of sugar cane supply</td>
<td>Implementing appropriate win/win sustainable mitigation measures and sustain profits</td>
<td>Diffuse power</td>
<td>NO</td>
</tr>
<tr>
<td><strong>UCOSP (Pty) Ltd (representing the farmers)</strong></td>
<td>Providing infrastructures and flood protection infrastructures on behalf of the sugar cane farmers;</td>
<td>Flooding of lands by backing up the water of the Mfolozi and above a level of 1,50 meter above MSL</td>
<td>To ensure the exploiting activities of farming by protecting the farmers against floods</td>
<td>Diffuse power; Blocking power through demonstrations and institutional complaints, limited land ownership</td>
<td>YES</td>
</tr>
<tr>
<td>Inhabitants of St Lucia</td>
<td>Living conditions in St Lucia, profit from tourism and fishery in the St Lucia Wetland Park, protection against floods</td>
<td>Desiccation by a shortage of rain which can lead to a degradation of the Wetland Park; Missing of a open mouth with the sea leads to a shortage of marine species</td>
<td>Making profit out of tourism and consolidate their living circumstances by protecting the well functioning of the St Lucia Wetland park for tourism and fishery</td>
<td>Limited blocking power through demonstrations and institutional complaints</td>
<td>NO</td>
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<tr>
<td>Surrounding municipalities</td>
<td>Well functioning of the local economy; Good living circumstances of inhabitants concerning health, education, and living.</td>
<td>Desiccation through a shortage of rain that leads to the movement of humans towards more fertile grounds near the Wetland Parks; Poverty; Increasing the boundaries of the iSimangaliso Wetland Park could decrease the opportunities for plantations and agricultural activities</td>
<td>Providing job opportunities for local inhabitants in and around the area; Providing better living circumstances, health conditions and educational opportunities through additional incomes from tourism.</td>
<td>Diffuse power</td>
<td>NO</td>
</tr>
<tr>
<td>Local communities</td>
<td>Survive</td>
<td>Boundaries/fences of the game reserves around fertile grounds; poverty; lack of ecological problem understanding</td>
<td>Surviving by farming, poaching, and/or tourism</td>
<td>Diffuse power</td>
<td>NO</td>
</tr>
<tr>
<td>Universities</td>
<td>Gaining knowledge through research and getting attention of the public, governments, and decision makers at large, on the importance of the environmental impacts in the iSimangaliso Wetland park through desiccation and human developments.</td>
<td>Insufficient knowledge about the impacts of human/natural changes in the iSimangaliso Wetland Park</td>
<td>To foster awareness and commitment from governments to take action, and provide adequate knowledge/research resources</td>
<td>Knowledge, supporting power</td>
<td>NO</td>
</tr>
<tr>
<td>GEF</td>
<td>Investments in projects for improvements of biodiversity, land degradation, and international waters and reduction of negative impacts regarding climate change, ozone layer, and persistent organic pollutants.</td>
<td>Lack of funds to conserve the iSimangaliso Wetland Park ecosystem functioning</td>
<td>Improving the ecosystem functioning of the Lake St Lucia System to conserve wetland habitats of global importance</td>
<td>Financial power</td>
<td>YES</td>
</tr>
<tr>
<td>UNESCO</td>
<td>Cultural and natural heritage around the world considered to be of outstanding value to humanity</td>
<td>Degradation of the ecosystem in the iSimangaliso Wetland Park due to human influence.</td>
<td>Seeking to encourage the identification, protection and preservation of cultural and natural heritage around the world considered to be of outstanding value to humanity.</td>
<td>Administrative power, very limited financial resources</td>
<td>NO</td>
</tr>
<tr>
<td>RAMSAR</td>
<td>Conservation and wise use of Wetland and their resources.</td>
<td>Degradation of the ecosystem in the iSimangaliso Wetland Park due to human influence.</td>
<td>the conservation and wise use of all Wetland through local and national actions and international cooperation, as a contribution towards achieving sustainable development throughout the world</td>
<td>Administrative power</td>
<td>NO</td>
</tr>
<tr>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>Mining companies</td>
<td>Profit</td>
<td>The UNESCO and RAMSAR status of the iSimangaliso Wetland Park</td>
<td>To make profit</td>
<td>-</td>
<td>NO</td>
</tr>
<tr>
<td>Water Research Commission (WRC)</td>
<td>Develop and support a water-related knowledge base in South Africa; Understanding of the St Lucia system to reconnect the Mfolozi to St Lucia</td>
<td>Threat of a lack of sufficient water, while water quality and availability issues are becoming more acute in the country</td>
<td>Generating new knowledge and to promote the country’s water research purposefully</td>
<td>Knowledge, limited financial resources</td>
<td>NO</td>
</tr>
<tr>
<td>Consortium for Estuarine Research and Management (CERM)</td>
<td>Bring together researchers to develop a description of the Mfolozi/Msunduzi estuarine system, its process and dynamics (biotic and physical components)</td>
<td>Knowledge gaps about complex processes within estuaries at the ecosystem level</td>
<td>Promoting the wise management of estuarine systems through joint participation in directed research, training, and technology transfer to foster the sustainability of the system</td>
<td>Knowledge</td>
<td>NO</td>
</tr>
</tbody>
</table>

C.18 Analysing the stakeholders’ organization

C.18.1 Stakeholder mapping

The power versus interest grid shows that there are many stakeholders involved that have a lot of interest in the project. However, it is hard to identify whether or not these stakeholders are a supporter or part of the opposition. Therefore the stakeholders have been represented in the stakeholder problem frame in Figure C-1. An interesting notion to make is that there are a lot of stakeholders represented as (weak or strong) supporters. These “attitudes” are based information gained from interviews and articles with and from different actors.

The universities are also an important party. They can have a lot of impact on the perceptions of the different stakeholders. Due to a lot of scientific research the working of the different ecological and biological systems is described. As a result adequate knowledge to make a well found decision is acquired. Therefore the Universities can influence the project process a lot.

Sensitive actors are the UCOSP and the Umfolozi Sugar Mill. The UCOSP is currently considered as a supporter. According to Gerrit (De Jager, 2011) and Knox (Knox, 2011) they are willing to help to solve the ecological problems in the iSimangaliso Wetland Park. However, it is assumed that if they are sensitive for the outcome of the process. If they are not satisfied or have the feeling they are not involved they probably could turn into opponents of the process. Therefor the UCOSP and the Umfolozi Sugar Mill should be taken into account with care.

In addition, the local communities’ perception could differ per group of people. They have been considered as a neutral party, but should not be underestimated. A lack of knowledge could have some negative impacts on their willingness to adapt. Some could be supportive and other could be highly against changes. On the other hand they do not have much power, but could hamper the process in a legal and illegal way. Especially the latter is the biggest concern for the authorities.

![Figure C-1 Problem frame stakeholder map (for explanation of numbers see Figure C-1) (Source used: (Nutt & Backoff, 1992) pp. 198)](image)
C.18.2 Network diagram

The perceptions of every stakeholder have resulted in a figure, which indicates the relationships between the stakeholders (Figure C-2). This is called a network analysis. Most important acknowledgement that should be made is the distinction between directly involved stakeholders and indirectly involved stakeholders. The directly involved stakeholders are situated within the large rectangle. The indirectly involved stakeholders are not. The thickness of the lines represents the strength of the relationships and the size of the blocks represents the degree of influence. The colors represent their position in terms of supporters, opponents, and neutrals. Red stands for opponent, yellow for neutral, and green for supporters. In addition four types of relations will be distinguished in the network analysis. These are a representation relationship, a hierarchical relationship, an influence relationship, and a control relationship.

A representation relationship is understood as actor A represents the interests of actor A1. For example the municipality of Mtubatuba represents the inhabitants of St Lucia. A hierarchical relationship is understood to be a relation where “the superior is entitled to exert influence on the subordinate and the subordinate is obligated to accept the superior’s influence. These rights and duties, however, are not unlimited.” (Kelman, Herbert C. in (Darley, Messick, & Tyler, 2001).

When a stakeholder could influence its directly connected stakeholder it is called an influence relationship. The perception of a particular stakeholder could change perceptions of other stakeholders. Finally a control relationship is related to the power to influence people’s behaviour or the way they undertake actions. The controller “controls” the other party due to requirements and demands. The most important different relations are being described briefly below.

A representation relationship is the case for a couple of relations. The iSimangaliso Wetland Park authority represents the governmental in the iSimangaliso Wetland Park. The latter is responsible for the whole functioning of the Park and reports to the government. They also have received some additional legislative powers to control the project. However, next to the park authority the Ezemvelo KZN Wildlife is also representing the government, but only for the executive/maintenance part of the project. The main difference between KZN and the iSimangaliso park authority is that KZN falls under the control of the iSimangaliso Wetland Park authority. Two other representation relationships are the UCOSP that represent the farmers and indirectly the Umfolozi Sugar Mill and the municipality Mtubatuba should also represent the interest the inhabitants of St Lucia.

The relations between the government, the iSimangaliso Wetland Park authority, and Ezemvelo KZN Wildlife are also a hierarchical relationship. The government is the superior where the iSimangaliso Wetland Park authority and Ezemvelo KZN Wildlife are the subordinates. They are obliged to accept the superiors’ influence and demands. In this case the iSimangaliso Wetland Park authority also stands above Ezemvelo KZN Wildlife. However the distinction in responsibilities in history makes this is a complex relation.

Influential relationships are the most common ones in this project. Many stakeholders can influence each other by for example taking use of their networks or their power, giving biased information, or even through bribing. The iSimangaliso Wetland Park authority has many influential relationships. They provide a lot of information for different stakeholders and organize the process. In addition, the Universities do also have influential relationships with the KZN, UCOSP, and the iSimangaliso Wetland Park authority. Through scientific research they provide many inputs for the decision making processes. Even when they have to take
assumptions others could use that to structure their perception. On the other hand, clients could also influence the Universities by providing them the input that they seem to understand important.

A control relationship is also the case between the government and the Ezemvelo KZN Wildlife and the iSimangaliso Wetland Park authority. The two latter are responsible for the whole project. Particularly the iSimangaliso Wetland Park authority as they are ultimately responsible. Also the relation between the UNESCO and Ramsar with the government is a control relationship. The UNESCO and Ramsar are checking the governments if their “exceptional” sites still meeting the required demands. In addition the GEF is checking the iSimangaliso Wetland Park authority if they are meeting the required demands concerning the GEF.

![Network relation diagram](image)

**Figure C-2 Network relation diagram**

C.18.3 Management strategies

Next, we will briefly discuss some strategies that might be most appropriate to implement for specific scenarios. In the power versus interest grid in Figure C-4 the quadrants of “key players” and “Keep informed” are overrepresented.

According to the figure there are several powerful stakeholders and no really powerful opponents. Scholes mentions that the main danger is one of complacency (Ambosini, Johnson, & Scholes, 1998). He argues that a strategy of keeping the stakeholders satisfied is a priority.

However, if some stakeholders like the UCOSP and municipalities are not happy with the outcome of the process they could change their attitude from supporting to opposing. Even
when the process would take too long and people get the feeling they are not involved enough their perceptions could change together with their attitude. In a situation like that there conflicting interest between the key stakeholders could become an issue. In such situation it could be wise to facilitate the dominance of the supporters or even break down the resistance and prevent smaller power groups from linking up. However, this latter “divide and rule” principle could hamper the situation even more and could raise a lot of external resistance.

There are also a couple opponents with some small power. However, they should not be underestimated. When they form alliances their power could rise sufficiently. They even could step out of the process and start their own power base. To keep them informed is therefore very important. Again complacency could lead to the change of perceptions of weak or strong supporters towards opponents that could join the new alliances. For example the locals have to be informed quite well about the causes of the current problems and about the new opportunities for them in future. In this way they could become supporters of the project.

C.18.4 SWOT analysis

In the SWOT analysis, the Strengths and Weaknesses as well as the Opportunities and the Threats will be exposed from the viewpoint of the project. This is in the best interests of the implementation an additional connection from the Mfolozi towards the St Lucia estuary. Unlike the stakeholder analysis the SWOT has more opportunities to offer. It is assumed to be used in more situations. Individuals could use it to evaluate their developmental needs in terms of their strengths, weaknesses, opportunities and threats for development. (The Open University, 2011)

The SWOT is divided in four aspects, which are two internal and two external quadrants. The internal quadrant consists of strength and weaknesses. Opportunities and threats are subject to the external point of view. The reasoning of the SWOT model is based on to build on strengths, to eliminate weaknesses, to exploit opportunities, and to mitigate threats. The internal aspects are directly related to the project and the external are not. For example, that an opportunity is not yet included in the project, but could be in the future, or that the relation between the municipalities would improve during the project (see Figure C-3).

<table>
<thead>
<tr>
<th>Table 2 Common questions used in a SWOT analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Internal factors</strong></td>
</tr>
<tr>
<td><strong>Strengths</strong></td>
</tr>
<tr>
<td>What are we best at?</td>
</tr>
<tr>
<td>What intellectual property do we own?</td>
</tr>
<tr>
<td>What specific skills does the workforce have?</td>
</tr>
<tr>
<td>What financial resources do we have?</td>
</tr>
<tr>
<td>What connections and alliances do we have?</td>
</tr>
<tr>
<td>What is our bargaining power with suppliers and intermediaries?</td>
</tr>
<tr>
<td><strong>Weaknesses</strong></td>
</tr>
<tr>
<td>What are we poor at doing?</td>
</tr>
<tr>
<td>Is our intellectual property outdated?</td>
</tr>
<tr>
<td>What training does our workforce lack?</td>
</tr>
<tr>
<td>What financial constraints do we have?</td>
</tr>
<tr>
<td>What connections and alliances should we have, but don’t?</td>
</tr>
</tbody>
</table>

(Source: Based on Blythe, 2001, p. 17)

Figure C-3 Common questions that are relevant to the SWOT (source: (The Open University, 2011))
An important value of SWOT is that it could be used as a self-assessment for management. It can also stress some covered data that otherwise would have been forgotten. However, the SWOT remains still a relative simple tool. Assessing and describing what the strengths and weaknesses of an organization and the impact and probability of opportunities and threats are is very complex. The quality of the SWOT outcome is dependent on the amount of effort in internal and external research.

As described by Morrison (Morrison & Daniels, 2010) there are a number of advantages and disadvantages of using the SWOT approach to analysis:

Advantages include:
- It is a simple four box framework.
- It facilitates an understanding of the strengths and weaknesses of the organization.
- It encourages the development of strategic thinking.
- It enables a management team to focus on strengths and build opportunities.
- It can enable an organization to anticipate future business threats and take action to avoid or minimize their impact.
- It can enable an organization to spot business opportunities and exploit them fully.
- It is flexible.

Disadvantages include:
- Some users over-simplify the amount of data used for decisions – it is easy to use scant data.
- To be effective this process needs to be undertaken on a regular basis.
- The best reviews require different people being involved, each having a different perspective.
- Access to quality internal data sources can be time consuming and politically difficult (especially in more complex organizations – parent company, etc).
- The pace of change makes it increasingly difficult to anticipate developments that may affect an organization in the future.
- The risk of capturing too much data is that it may make it difficult to see the wood for the trees and lead to “paralysis by analysis”.
- The data used in the analysis may be based on assumptions that subsequently prove to be unfounded (good and bad).
- It lacks detailed structure, so key elements may get missed.

In the upcoming tables the SWOT’s of the four most important stakeholders will be described. The others are not mentioned in a SWOT analysis and are described in the stakeholder analysis in chapter C.
### Table C-1 SWOT Government of South Africa

<table>
<thead>
<tr>
<th>Internal</th>
<th>External</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Strengths</strong></td>
<td><strong>Opportunities</strong></td>
</tr>
<tr>
<td>* Power to overrule other stakeholders;</td>
<td>* Take over land control within 100 year flood line</td>
</tr>
<tr>
<td>* Landownership</td>
<td>* After a big flood the environment could be much more willing to adopt changes</td>
</tr>
<tr>
<td>* Financial resources</td>
<td>* Demands from UNESCO and Ramsar could put extra pressure on the process to adopt changes</td>
</tr>
<tr>
<td>* They have legislation power</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Weaknesses</th>
<th>Threats</th>
</tr>
</thead>
<tbody>
<tr>
<td>* Conflicting interests between sugarcane industry and ecotourism</td>
<td>* Mining companies could put pressure on the government when tourism incomes stay behind</td>
</tr>
<tr>
<td>* Lot of political influences</td>
<td>* Poverty relief initiative could work against effective measures</td>
</tr>
</tbody>
</table>

### Table C-2 SWOT iSimangaliso Wetland Park authority

<table>
<thead>
<tr>
<th>Internal</th>
<th>External</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Strengths</strong></td>
<td><strong>Opportunities</strong></td>
</tr>
<tr>
<td>* Power to overrule most of the other stakeholders;</td>
<td>* Tourism could stimulate local economies and the ecologic environment</td>
</tr>
<tr>
<td>* Representative of the government</td>
<td>* After a big flood the environment could be much more willing to adopt changes</td>
</tr>
<tr>
<td>* Flooding problem at the lower floodplains is not the problem of the park authority</td>
<td>* To change opponents into supporters by informative sessions and future expected prospects</td>
</tr>
<tr>
<td>* They have limited legislation power</td>
<td>* Higher water levels in St Lucia could stimulate the combining of the mouths.</td>
</tr>
<tr>
<td>* Big network</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Weaknesses</th>
<th>Threats</th>
</tr>
</thead>
<tbody>
<tr>
<td>* Limited financial resources</td>
<td>* Internal conflict between the different stakeholders concerning required measures to improve the ecological situation of St Lucia</td>
</tr>
<tr>
<td>* Lot of different stakeholder interests</td>
<td>* Uncertain weather prospects for future</td>
</tr>
<tr>
<td>* &quot;Limited&quot; rights of land ownership</td>
<td>* Illegal activities could damage electrical fences and reputation of the Park</td>
</tr>
<tr>
<td></td>
<td>* Local communities could not all be willing to adopt changes</td>
</tr>
</tbody>
</table>

### Table C-3 SWOT Ezemvelo KZN Wildlife

<table>
<thead>
<tr>
<th>Internal</th>
<th>External</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Strengths</strong></td>
<td><strong>Opportunities</strong></td>
</tr>
<tr>
<td>* Power to overrule most of the other stakeholders;</td>
<td></td>
</tr>
<tr>
<td>* Representative of the government</td>
<td></td>
</tr>
<tr>
<td>* Limited financial resources</td>
<td></td>
</tr>
<tr>
<td>* Lot of different stakeholder interests</td>
<td></td>
</tr>
<tr>
<td>* &quot;Limited&quot; rights of land ownership</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Weaknesses</th>
<th>Threats</th>
</tr>
</thead>
<tbody>
<tr>
<td>* Limited financial resources</td>
<td></td>
</tr>
<tr>
<td>* Lot of different stakeholder interests</td>
<td></td>
</tr>
<tr>
<td>* &quot;Limited&quot; rights of land ownership</td>
<td></td>
</tr>
<tr>
<td><strong>Internal</strong></td>
<td><strong>External</strong></td>
</tr>
<tr>
<td>----------------</td>
<td>----------------</td>
</tr>
<tr>
<td><strong>Strengths</strong></td>
<td><strong>Opportunities</strong></td>
</tr>
<tr>
<td>* Represent executive part of the government in the park</td>
<td>* Restoring the relations with local communities to increase effectiveness of the project and to decrease negative impacts on the project</td>
</tr>
<tr>
<td>* Knowledge acquired along the years required ecologic and biologic environment</td>
<td></td>
</tr>
<tr>
<td>* Important stakeholder</td>
<td></td>
</tr>
<tr>
<td><strong>Weaknesses</strong></td>
<td><strong>Threats</strong></td>
</tr>
<tr>
<td>* Conflicts between local communities and KZN in history still have their consequences</td>
<td>* When not complying with iSimangaliso Authority they can lose their power</td>
</tr>
<tr>
<td>* No financial resources</td>
<td>* A combined Mfolozi-St Lucia system increases sediment transport</td>
</tr>
<tr>
<td>* Dependent on governmental bodies regarding decision making</td>
<td></td>
</tr>
</tbody>
</table>

**Table C-4 SWOT UCOSP**

<table>
<thead>
<tr>
<th><strong>Internal</strong></th>
<th><strong>External</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Strengths</strong></td>
<td><strong>Opportunities</strong></td>
</tr>
<tr>
<td>* Strong stakeholder due to high economic interest</td>
<td>* Try to influence the decision making process to create a win-win situation</td>
</tr>
<tr>
<td>* High valued properties</td>
<td>* Shifting the problem of flooding the lower floodplains towards the Park authority</td>
</tr>
<tr>
<td></td>
<td>* Flooding could stimulate the decision to breach the Mfolozi.</td>
</tr>
<tr>
<td></td>
<td>* Combining the mouths would reduce the problem of back flooding</td>
</tr>
<tr>
<td><strong>Weaknesses</strong></td>
<td><strong>Threats</strong></td>
</tr>
<tr>
<td>* Dependent on the decisions of the governmental bodies</td>
<td>* Floods could flush away parts of the flood plain and could stimulate changes of land-use</td>
</tr>
<tr>
<td>* Deterioration of the St Lucia system works against the decision to breach the Mfolozi</td>
<td>* Measures regarding stimulating the ecologic situation could have negative impacts on the sugarcane farmlands</td>
</tr>
</tbody>
</table>
C.19 Stakeholder game

C.19.1 Analysing the stakeholders’ power position

In Figure C-4 the power versus interest matrix is represented. Price (Price, 2009) has identified four different kinds of stakeholders, which are “Monitors”, “Intruders”, “Onlookers”, and “Outsiders”. The different actors have been listed and can be found in Figure C-4.

1. Government of South Africa (Department of water affairs)
2. iSimangaliso Wetland Park Authority
3. Ezemvelo KZN Wildlife
4. Municipality of Mtubatuba (Incl. St Lucia district)
5. Umfolozi Sugar Mill (Pty) Ltd (USM) (representing the agro-processor)
6. UCOSP (Pty) Ltd (representing the farmers)
7. Inhabitants of St Lucia
8. Surrounding municipalities
9. Local communities
10. Universities
11. GEF
12. UNESCO
13. RAMSAR
14. Mining companies
15. Water Research Commission (WRC)
16. Consortium for Estuarine Research and Management (CERM)

Figure C-4 Power versus interest grid (source used: (Price, 2009))

Price (2009) identifies four categories of stakeholder as follows:

- Monitors are individuals or groups who have a lot of power, but a low interest in the project. They could support or undermine the project process. They could also be unique stakeholders that are hard to replace. Price (Price, 2009) remarks the possible position they can take in the process. They could work with you, but also against you. They are “monitoring” the process and could step in whenever they like. However, when satisfied they would probably not put much effort in the process.

- Intruders are similar to monitors regarding their power play. However, they do have a lot of interest. They are sufficiently interested to take action if they want to and they have the power to do so. They are an important group of stakeholders.

- Onlookers may be very interested in the changes taking place but they are depended on the powerful stakeholders. Through their networks or through an “alliance” they could influence the attitudes of more powerful stakeholders. It is important to inform them about the process.

- Outsiders are individuals or groups that have both a low power and interest. On their own they do not have much to say. They could try to join or block the process, but are most of the time replaceable. However, they should not be forgotten.

The power versus interest grid shows that there are many stakeholders involved that have a lot of interest in the project. However, it is hard to identify whether or not these stakeholders are a supporter or part of the opposition. Therefore the stakeholders have been represented in the stakeholder problem frame in Figure C-1. An interesting notion to make is that there are a lot of stakeholders represented as (weak or strong) supporters. These “attitudes” are based information gained from interviews and articles with and from different actors. On explanation of the classification of the stakeholders is been given in paragraph C.
C.19.2 Management approach

The different kind of management approach is analyzed to get a better understanding of the design process. The used theory is related to the philosophy of the course book “Stakeholder-oriented Project Management (2011)”. According to Van Gunsteren (2011) the management approach could be divided in PI and PII management. In short, the distinction between PI and PII management is that the “fast” and “easy” PI approach could be in favor in a project that is straightforward and predictable. The problem is known, goals are fixed, and the project is pretty predictable. In addition, the open minded PII approach is in favor in a complex project with a lot of uncertainty, unpredictability, and different stakeholder’s interests.

Generally, in project management literature the focus is mainly on PI traditional management practices. The PII approach is “a stakeholder-oriented approach to all issues that (may) arise”. According to Van Gunsteren (2011) this PII approach could conflict with many of the best recommended construction management practices in literature. For example fulfilling the contract and its specifications, hold on to the earlier made planning, and concentrating on the critical path. They describe that the contract should not be the basis for decision making, that the planning is a dynamic object, and that there is no such a thing as a Critical Path.

Looking at the size of the Back Channel project, it is considered as a small scale project in a very big complex overall project, which is called the GEF project. However, the impacts of the small scale project are immense in relation to the overall GEF project. According to the theory described by Van Gunsteren (2011) the overall project could be managed with a PII approach in combination with a PI approach. The project should be addressed with PII management and the less complex activities (sub parts) should be addressed with PI management. For a successful project, it is important to mix both approaches appropriately. In this section eight aspects regarding the literature are assessed against the practices used.
in the overall project. The eight aspects are categorized in three issues: process related, information handling related, and structure related. Most of the distinctions between PI and PII are based on the interviews with Bronwyn James and Nicky Forbes (James & Govender, 2011) (Forbes, 2011) and on literature in “A review of studies on the Mfolozi estuary and associated flood plain, with emphasis on information required by management for future reconnection of the river to the St Lucia system” (Bate, Whitfield, & Forbes, 2011). The described management approach is focusing at the iSimangaliso Wetland Park Authority in relation to the Back Channel project.

**Process related issues**
The process related issues are the issues that determine the quality and adaptability of the process. The issue is subdivided into the following aspects:

- Goal setting
- Leadership
- Conflict resolution
- Design process

**Goals setting: PI with aspects of PII**
The goal setting is mostly done with a PI approach. The overall goal of the iSimangaliso Wetland Park Authorities organization is fixed and considered as PI. The latter is related to the consolidation of parks / land, biological and legal. "Managing the iSimangaliso through effective and efficient governance and the strongest possible legislative, regulatory and institutional arrangements" (James & Govender, 2011). Also, as a consequence of the GEF fund there are some additional contractual requirements that steer the goals of the park authority.

The project is also focusing on PII management. The project is re-evaluated on its objectives and nothing is fixed in advance. Changing circumstances and new insights could bring up new solutions. However, the board makes the final decisions.

**Leadership: PII**
The iSimangaliso Wetland Park authority is the overall manager of the GEF project. They have the final responsibility of the project. However, they are aiming at leadership that is focusing on defending the relevant stakeholders' interest. Many stakeholders are involved in the overall process to stimulate developments and local improvements. Many decisions are also made in collaboration with the Ezemvelo KZN Wildlife, which is responsible for the nature conservation part of the park.

**Conflict resolution: mainly PII**
Conflict resolution was solved open with attention to everyone's interests. However, there were a number of contract items as a result of getting a GEF fund. This puts extra pressure on the goals of the iSimangaliso authority to comply with their demands. On the other hand the park authority is not only focusing at the strong stakeholders. Their aim is that valid information rather than power structure determines the outcome. However, when the latter fails the park authority could use its power to force a solution. At the moment, a conflict about breaching the Mfolozi mouth is still unsolved and currently lawyers are involved in the process. Until a solution is found that fits both the interest of the farmers and the park authorities conflicts will probably be handled in a PI approach.

**Design process: PII**
In the past, the design process was mainly focused on a PI approach. The focus was on getting a solution through a trial and error process starting from an arbitrarily chosen first
design. Examples are mouth separation and the Link channel. Since the iSimangaliso park authority is in control they tried to change the design process into a more solution space oriented process. Many stakeholders are involved as well. Even in the Back Channel project many stakeholders are involved because of the large importance of the project in relation to the whole St Lucia Lake system.

**Information handling related issues**
The issue is subdivided into the following aspects:

- Communication
- Persuasion of players
- Progress control

Communication: PI
Communication of the park authority is assumed to be mainly focused on informing everyone about the design status, changes, and planning. Communication is information oriented. Not all relevant information is provided, for example about the problems in the St Lucia estuary. Many of the inhabitants are still less informed about the actual problem causes of the low water levels in the system. However, whether or not this is a strategic behaviour from one or both of the parties is hard to say.

Persuasion of players: PI & PII
The persuasion of players is firstly assumed to be based on a PI approach. For example convincing parties through audio-visual aids has been used. Improving the PR of the authority and image building was necessary to counteract the bad reputation that has been gained in the past. Additionally, there is a lot at stake. It is doubtful if all valid and relevant information is provided. There is a lot at stake and strategic behaviour of parties cannot be avoided. However, by involving many stakeholders in the process the idea is suggested that there is no such thing as a hidden agenda.

Progress control: PI & PII
The progress control is assumed to be a mixture of PI and PII management. The whole project focus is divided into small steps with identifiable milestones and probably planned deadlines. On the other hand, initiatives from other players are appreciated as well. New research outcomes and scientific results and insights are used to solve the problem. Communities are involved and have the opportunity to provide input as well.

**Structure related issues**
For the structure related issues only the division of tasks is considered as relevant and has been described briefly.

Division of tasks: PII
The organization of the iSimangaliso authority is mainly a horizontal organization. Park operations, commerce unit, research unit, and training and capacity building are the four cornerstones of the park authority. It is assumed that groups are responsible rather than the individual and mutual adjustment between the tasks is considered as important.

**Summary of the management styles**
The project is a combination of PI and PII approaches with a main focus on the PII part. From Table C-5 there can be seen that PI has the overhand by the information handling. The process related issues are mostly PII approach. A reason for this could be that the park authority wants to create an open design, but still has some of the responsibilities to steer the process in a preferred direction. Especially because of the large impacts in the project the park authority could respond and act very cautious.
Table C-5 summarized management approaches

<table>
<thead>
<tr>
<th>ASPECTS</th>
<th>PI/PII</th>
</tr>
</thead>
<tbody>
<tr>
<td>Process related issues:</td>
<td>Mostly PII</td>
</tr>
<tr>
<td>- Goals setting</td>
<td>PI with aspects PII</td>
</tr>
<tr>
<td>- Leadership</td>
<td>PII</td>
</tr>
<tr>
<td>- Conflict resolution</td>
<td>PII with aspects of PI</td>
</tr>
<tr>
<td>- Design process:</td>
<td>PII</td>
</tr>
<tr>
<td>Information handling related issues:</td>
<td>PI &amp; PII</td>
</tr>
<tr>
<td>- Communication</td>
<td>PI</td>
</tr>
<tr>
<td>- Persuasion of players</td>
<td>PI &amp; PII</td>
</tr>
<tr>
<td>- Progress control</td>
<td>PI &amp; PII</td>
</tr>
<tr>
<td>Structure related issues:</td>
<td>PII</td>
</tr>
<tr>
<td>- Division of tasks</td>
<td>PII</td>
</tr>
</tbody>
</table>

C.19.3 Core elements of a process design

An open process is characterized by an open attitude. Transparency is an important subject. Other actors are involved in decision making and trust is an important issue. The involved actors range from opponents to supporters and sometimes even actors that do not have any power. The feeling of being heard and being involved is considered as an important value for actors.

However, next to the fact that they want to be heard they also want to protect their core values. An open process could result in situations where an actor’s core value is at stake. The core values have to be protected regardless of the outcome. It is also important to commit to the process instead of the result. However, an actor should have an exit option if his core values are at stake. Even earlier made sub-decisions should sometimes being revoked. Preventing a feeling of no return on sub-decisions could foster the process.

The third core element is speed. The latter focuses on the speed of the design process. A threat of an open process where everybody’s values are protected could result in a never ending process. Therefore sometimes decisions have to be made in order to keep the process running. Prospects of gain and incentives are important to keep the actors motivated. In addition, it is important that the gain is not being paid out to earlier for various actors. The latter could work as a disincentive for the cooperative behaviour and could lead to opportunistic behaviour.

The final core element is the substance of the process. The process should be sufficiently substantive. Insights of experts could be implemented to facilitate the decision making process.

A sense of urgency is a sine qua non for the success of a process. A common problem perception is really important. The stakeholders should be convinced that the problems have to be solved through some sort of cooperation. A sense of urgency is not created when the process starts too early. It has to be developed along the dynamic process. According to De Bruijn this process starts with an initiative, followed by a process of pushing and pulling (stakeholder game) between the actors. This latter would lead to results or stagnates. If the process stagnates parties may develop a sense of urgency among some of the parties. The sense that something has to happen could be a breeding ground for process agreements between parties. (De Bruijn & Heuvelhof, 2008)
C.19.4 Network organization

Table C-6 comparing decision making in hierarchies and networks (De Bruijn & Heuvelhof, 2008)

<table>
<thead>
<tr>
<th>Hierarchy</th>
<th>Network</th>
</tr>
</thead>
<tbody>
<tr>
<td>Regular and sequential</td>
<td>Irregular and no clear sequence of activities</td>
</tr>
<tr>
<td>Phases</td>
<td>Rounds</td>
</tr>
<tr>
<td>Actors are stable</td>
<td>Actors join and withdraw and behave</td>
</tr>
<tr>
<td></td>
<td>strategically</td>
</tr>
<tr>
<td>One arena, process has a clear</td>
<td>Several arenas, no isolated starting</td>
</tr>
<tr>
<td>point and end point</td>
<td>point and end point</td>
</tr>
<tr>
<td>Content of the problem is stable</td>
<td>Content of the problem shifts</td>
</tr>
<tr>
<td>Incentive to regard problems as</td>
<td>Incentive to regard problems as unstructured</td>
</tr>
<tr>
<td>structured</td>
<td></td>
</tr>
<tr>
<td>Consistency and predictability</td>
<td>Flexibility and unpredictability</td>
</tr>
</tbody>
</table>

The network-like structure shows a lot of similarities when analyzing the kind of organization in the project. Different actors may perceive different interpretations of the same intervention for example about the kind of construction or way of combining the systems. Actors could block further decision making that could harm their interests. Many problems could remain unsolved. Furthermore, the design cycle goes in rounds. A winner of an earlier round could be the loser in the next round. For example the Link Channel in the 1980s. It was considered to be the best alternative, but after the Demonia flood a hard concrete structure was out of the picture.

In the past many new players joined the “arena” while the process progressed. For example, biologists, UNESCO inspectors, scientists, researchers etc. Next to that there are several arenas. The Back Channel area is not the only decision that has to be made concerning the different actors. The GEF project involves many more “arenas”. In addition the process does not have a clear end point. The dynamic character is one of the causes of the latter.

In addition, the content of the problem is shifting in time. Parties could reformulate their issues or put forward new ones. In the past the problem focus varied between siltation, sediment transport, fresh water inflow, marine distribution, salinization, etc. There are also not only one or a few solutions for the problem. Effects on biologic and ecologic system are hard to predict. Normative questions pop up as well. Therefor the problems are considered as unstructured. Finally the organization is characterized by flexibility and unpredictability. A major issue of today could become a detail tomorrow and vice versa. For example the problem of siltation that resulted in a separated mouth. Now the fresh inflow of water is the major issue and siltation a detail. Additionally, the decision making process is unpredictable and events could take their own course.

Networks also consist of mutual dependencies. Interdependency between actors could lead to poor decision making (grey compromises). For example, in the past they kept the mouths separated in the interests of the farmers in relation to flooding, but they did not realize the impacts on nature of such a separation. Yet, there are also advantages through dependencies in a network-like organization. Parties are forced to behave moderate towards each other due to interdependencies. Instead of grey comprises substantive enrichment could be realized. Substantive enrichment could lead to solution where the sum of the parts may be greater than the whole. “Variety makes an organization smart” (De Bruijn & Heuvelhof, 2008). There are more possibilities for exchange when interdependency is complex.
Decisions are made in a process of interaction. As a consequence an actors’ strategically behaviour could focus on how to influence the process of interaction. This means that the behaviour of an actor is not determined by his opinions. Consolidating his power position in the network is then more important. For example, the farmers interpret the problem of flooding on the floodplains as a problem of the iSimangaliso Wetland Authority. However, by farming on the lower parts of the floodplains the farmers created their own problem. The open Mfolozi mouth was one of the reasons for the possibilities of farming on the lower parts. In addition, they did not take into account the impacts on the St Lucia system. Giving too much room available for decision making could have an impact on their power position. Therefore it is really important to take strategic behaviour into account in a stakeholder analysis. De Bruijn states that the attention of an actor analyses has to shift from an ex ante, analytic activity towards an ongoing learning process with a focus on the right actions instead of the right analysis.

To end with, the dynamic behaviour of a network could lead to capricious decision making. Threats could become opportunities and opportunities could become threats. The content of the problem shifts in the course of time and therefore it is really important that the design process adapts to the outcomes of the network in order to keep searching for a win-win situation. Finally, the process does not reach an end. Implementation creates new rounds and therefore new opportunities and furthermore evaluation remains an ongoing activity.
**D. REQUIREMENTS ANALYSIS**

The requirement analysis starts with an input of objectives, requirements, and stakeholder needs. First the input values are analysed. Secondly functional and performance requirements are derived and project constrains are defined. In order to make the requirements analysis the 15 steps as prescribed in the Department of Defence handbook (Department of Defence, 2011) are followed (see Figure D-1). The stakeholder analysis is described in appendix C and has been used as basic input for the requirements analysis.

![Figure D-1 Requirements analysis steps, source: (Department of Defence, 2011)](image)

In this chapter the 15 steps of system engineering according to the DAU will be elaborated. Some steps will be elaborated in detail and other less important steps will be mentioned briefly.

**D.1 Customer Expectation**

Defining and quantifying customer expectations is an important step in determining the functions incorporated in the design. The purpose of this task is to determine what the customer wants and how well each required function must be incorporated. The iSimangaliso Wetland Park Authority is the main customer in this project.

**iSimangaliso Wetland Park Authority**

1. Introducing sufficient fresh and clear water into the estuary to keep salinity levels in the lakes down to acceptable levels.
2. To restore the original ecology of the iSimangaliso wetlands parks.
3. Restoring the biological link between the sea and the iSimangaliso wetlands.
4. A solution which does not increase sediment load in the wetlands.
5. Allowing local inhabitants to participate in the project to increase local support.
6. A solution which does not require artificial breaching of the Mfolozi mouth.
7. Required maintenance should be as low as possible.
10. If possible environmental impact assessments should be avoided.
11. Solution should comply with environmental regulations.

**Ezemvelo KZN Wildlife**

1. To restore the iSimangaliso wetlands to a healthy ecological state in order to increase tourism and biological diversity in the estuary.
2. A simple construction without complex subsystems which are difficult to construct or maintain.
3. Accessibility of weir.
4. No harmful disturbance of local wildlife.
D.2 Project and enterprise constrains

In this subchapter constrains that have an impact on the design solutions are defined.

- Proposed solution should not result in excessive flooding on the cultivated floodplains.
- In times of drought, sufficient fresh water must flow from the Mfolozi into the St Lucia Estuary.
- Water entering the Estuary must be as sediment free as possible.
- A short term drought reducing solution is sought after.
- Solution should comply with the Euro codes.
- Manual labour is preferred over machinery.
- A maintenance friendly solution is preferred.
- Environmental impact studies should be taken into account.

D.3 External Constrains

External constrains can alter design solutions or implementation of the systems engineering process activities. Here some relevant external constrains will be described briefly:

- It is not allowed to combine water sources from different catchment areas.
- The construction is situated in a wildlife zone and can be exposed to wildlife.
- An open connection between the St Lucia Estuary is necessary and should be available as often as possible in order to increase migration of fish.
- Fishery conditions may not decrease.
- Solution must comply with the guidelines and criteria of the UNESCO World Heritage Convention. These latter consist of three main criteria which are in short:
  - “The park contains exceptional aesthetic qualities. Three natural phenomena are judged outstanding. One is the shifting salinity states within Lake St Lucia which are linked to wet and dry climatic cycles, with the lake responding accordingly with shifts from low to hyper-saline states. A second is the spectacle of large numbers of nesting turtles on the beaches and the abundance of dolphins and migration of whales and whale sharks off-shore. Finally, the huge numbers of waterfowl and large breeding colonies of pelicans, storks, herons and terns are impressive and add life to the wild natural landscape of the area.” (UNESCO, 1999)
  - “The combination of fluvial, marine and aeolian processes initiated in the early Pleistocene in iSimangaliso has resulted in a variety of landforms and continues to the present day. The Park’s transitional geographic location between sub-tropical and tropical Africa as well as the coastal setting have resulted in exceptional species diversity.” (UNESCO, 1999)
  - “The five interlinked ecosystems found in iSimangaliso provide habitat for a significant diversity of African biota, including a large number of threatened and/or endemic species. The outstanding diversity of habitats (terrestrial, wetland, coastal and aquatic) supports a wide variety of animal species, some at the northern and many at the southern limit of their range.” (UNESCO, 1999)
- Solution must comply with the guidelines of Ramsar as described by the “Strategic Framework and guidelines for the future development of the List of Wetlands of International Importance of the Convention on Wetlands (Ramsar, Iran, 1971)” (Ramsar, 2008)
- An Environment impact assessment (EIA) should be prevented. Requirements regarding the relevant requirements of the EIA are represented in Appendix A.
D.4 Operational scenarios

Scenario planning is used to describe plausible stories about the future and to extend our thinking about the opportunities and threats that the future might hold. Scenarios can involve the evolvement of relevant issues like political environment, social attitudes, demographical change, regulation, and economy. An important notion relevant for scenarios is that scenarios are not predictions. They are hypotheses that describe a range of future possibilities, which can be good and bad, expected and surprising, growth or decline. Scenarios are a medium through which great change can be envisioned and actualized when done properly. Exploring the future could lead to solutions that are better prepared for different future outcome scenarios. In a multi-stakeholder environment it could stimulate useful conversations about future possibilities that consist of a common ground of interest.

Scenarios have been made for the short-term (0-5 years) and mid-term (5-15 years). In addition to these scenarios there is also the zero-scenario. This is the situation that is the most realistic when nothing happens.

Two scenarios have been described in earlier researches. (Bate, Whitfield, & Forbes, 2011) One is the scenario with on the axis’s Mfolozi mouth open versus Mfolozi mouth closed and St Lucia mouth open versus St Lucia mouth closed as used by Kelbe and Taylor (Kelbe & Taylor, 2011). The other one is the scenario of Lawrie et al. (Lawrie, Chrystal, & Stretch, On the role of the Mfolozi in the functioning of St Lucia: water balance and hydrodynamics, 2011) were they combined the scenarios of ‘Separate inlets with mouth manipulation’, ‘separated inlets, no artificial mouth manipulation’, ‘combined mouth, no artificial mouth manipulation’, and ‘separate inlets, but with fresh water transferred into St Lucia via a link canal (Back Channel)’. These scenarios have been used as a basis for the scenario as defined in this report.

Important aspects to take into account are the following scale ranges:

1. Mfolozi mouth open – Mfolozi mouth closed;
2. St Lucia estuary mouth open – St Lucia estuary mouth closed;
3. Combined open mouth Mfolozi-St Lucia; Combined closed mouth Mfolozi-St Lucia;
4. Separated open mouth Mfolozi-St Lucia – Separated closed mouth Mfolozi-St Lucia;
5. Shorter dry periods – Longer dry periods;
6. Economic growth – Economic decline;
7. Increasing tourism – Decreasing tourism;
8. Less rainfall – more rainfall;

One operational scenario has been designed in order to take all of these aspects into account. This scenario consists of the ecology improvement versus deterioration on the x-axis and the economic growth versus economic deterioration on the y-axis (see Figure D-2). This scenario focuses on almost all of the interested stakeholders. Longer dry periods (droughts) and chance/decline of species are covered under ecology deterioration. Longer wet periods, more rainfall, and an increasing amount of species are covered by the ecology improvement. The aspects of tourism are covered by the economy axes.

An operational scenario is defined between the two extremes of economic growth versus decline and ecological deterioration versus improvement. The four scenarios are abstractly named as “Kruger 2”, “Save the Hippo”, “Alternative goldmine”, and the “Hard battle” and are elaborated further below.
The zero scenario is the scenario that is the most obvious situation when nothing will change in the area. This means that the current Back Channel is the only link between the Mfolozi and the St Lucia estuary. A model about the St Lucia mouth open versus St Lucia mouth closed related to the Mfolozi mouth open versus Mfolozi mouth closed has been used by Kelbe & Taylor (Kelbe & Taylor, 2011). This latter scenario has been used to describe the current zero scenario of the system.

D.4.1 Zero scenario
Currently the mouth of the Mfolozi and St Lucia estuaries are closed during dry periods. However, the mouth of the Mfolozi is being breached by the farmers to protect their lands from flooding when the water levels in the lower floodplains reach up to 1.5 meter above GMSL. The farmers prefer the berm height even lower. As a result, water levels in the Mfolozi are becoming too low and reduce the functioning of the current Back Channel. The movement of water from the Mfolozi into St Lucia is of greatest value for the functioning of the St Lucia lakes (Kelbe & Taylor, 2011). The Mfolozi water would normally partially replace water evaporated from the lakes, but is too low to compensate the total evaporation. The water levels in both systems are mostly dependent on river inflows. Flow duration analysis of Mfolozi flows indicates that besides the winter period, the likelihood of the Mfolozi remaining closed for extended periods is very small. (Lawrie, Chrystal, & Stretch, On the role of the Mfolozi in the functioning of St Lucia: water balance and hydrodynamics, 2011)

An open Mfolozi mouth is influenced by tidal flows. The backup of water in the lower floodplains is less due to the open Mfolozi mouth. As a result the risk of flooding on the lower floodplains is lower. Farming on the lower parts of the floodplains is risky, but not impossible. As a result of the lower water level in the Mfolozi the inflow into the Back Channel will decrease rapidly.

During wet periods both mouths could be possible that both systems would naturally breach. Currently this is much more often the case for the Mfolozi then it is for the St Lucia estuary.
This would only happen once in 10 years for the St Lucia mouth (Lawrie, Chrystal, & Stretch, On the role of the Mfolozi in the functioning of St Lucia: water balance and hydrodynamics, 2011).

Currently, it is improbable that only the mouth of the St Lucia estuary will open since the Mfolozi mouth remains open for most of the time due to the high run-off of flows or through human interventions. The inflow from other catchments into the St Lucia system is too low to provide enough outflows to open the St Lucia mouth.

D.4.2 “Kruger 2”

The scenario takes into account longer wet periods and more chances of floods. More floods would keep the Mfolozi mouth open for sure. The St Lucia system could breach more often as a result of a bigger outflow. However the water levels and inflow are such low that they cannot meet the required amount of water for the short term to breach naturally. A high outflow of water decreases the chances of extensive siltation in the Narrows and salinity in the lakes would be avoided. On the short term this could result in an increase of wildlife in the iSimangaliso Wetlands Park. In addition, the growing economy stimulates the development of the iSimangaliso Wetlands Park. Tourism facilitations are becoming more popular and more people will visit the park. Increasing tourism could stimulate the local economies and creates more opportunities for work. As tourism becomes more important the pressure on the sugar cane farmers to comply with required changes in the system increases.

In the midterm, if the mouth breaches this could increase the exchange of species between the systems. According to Sandy van der Waal (2011), an open mouth near St Lucia would re-establish vital nursery grounds for estuary dependent marine fish. In addition, sediment would flush away into sea instead of flowing up into the narrows and the St Lucia lakes. Fishery could also be stimulated by an open St Lucia mouth. However, it is not assumed that even in wet periods the St Lucia mouth could remain open for a long time.

When the developments continue the iSimangaliso Wetlands Park could compete with bigger game reserves like the Kruger Park. In order to stay competitive, more land can be bought/required from the local farmers to increase the boundaries of the park. For the local communities this would mean a shift from an agricultural business towards a more tourism business. Sugar cane farmers could be forced to move to other lands in order to keep the Back Channel optimal functioning or even to restore a combined mouth estuarine system. On the other side, as a result of more rain and longer wet periods, the risk of floods in St Lucia becomes higher. Erosion along the rivers could result in more sediment in the lakes and The Narrows, but would probably flush away quickly.

D.4.3 “Save the Hippo”

Less water inflow and more and longer droughts are assumed in the case of a growing economy and ecological deterioration. Distribution of water between the Mfolozi and St Lucia estuary is minimal and the rain in the other catchments areas around the St Lucia lakes could compensate the evaporation partly. A closed mouth of the St Lucia estuary would restrict the exchange of marine species with sea and salinity in the lakes would increase, especially during the dryer winter months. Due to a closed mouth and drought the Lakes could become separated. Occasionally, during a few floods the Mfolozi mouth could breach naturally. Probably then the Mfolozi would stay open for quite some time. This would have a negative impact on the St Lucia system because the Back Channel functioning decreases. The natural breaching of the St Lucia mouth would occur only once in 10 years or may take even longer (Lawrie, Chrystal, & Stretch, On the role of the Mfolozi in the functioning of St Lucia: water balance and hydrodynamics, 2011). As a result the lake dries up to fast and salinity levels will become higher. For the short term this would not result in direct impacts in
the functioning of the wetlands. The wetlands are resilient and could survive for a couple of years.

On the other hand, a growing economy and thereby the increase of tourism could put extra pressure on the government to invest in solutions against the drought. The government has to invest in solutions to keep the iSimangaliso Wetlands Park liveable and attractive. Tourism is one of the main driving forces of the poor local economies.

For the midterm, a closed mouth and longer droughts would lead to higher salinity levels in the lake. The inflow from other rivers into the St Lucia lakes is too low to compensate the evaporation in time. The current Back Channel could not provide sufficient inflow (the mean annual fresh water contribution was about 150Mm$^3$ per year (Lawrie, Chrystal, & Stretch, On the role of the Mfolozi in the functioning of St Lucia: water balance and hydrodynamics, 2011)) to compensate the evaporation and water becomes scarce and salty. When the water level drops below estuary mean water level (EMWL) the sand berm of the St Lucia estuary could be breached for an inflow of sea water. Water levels would rise, but salinity levels would increase even further. Current wildlife and vegetation could disappear. This would work destructive for the current habitat. In worse cases the wildlife has to be transferred to other park reserves in order to survive. New forms of salt water species and vegetation could show up in the park. The park probably cannot meet the criteria of shifting salinity states, huge numbers of waterfowl and large breeding colonies, and diversity of habitats of the UNESCO World Heritage Convention. As a result they could lose their UNESCO status.

In order to save the park the government could intervene in the process and could impose a solution. As a result of the economic interest of the park extra funds could becoming available. However, if nothing changes to keep the park functioning, tourism incomes could decrease. To keep the economy running new ideas of investment could pop up instead of the tourism branch. Mining could become an interesting alternative to further stimulate the economy if the Wetland Park becomes doomed to failure.

D.4.4 “Alternative goldmine”

As a result of an ecological deterioration and dryer periods the water level in the Mfolozi will be too low for a good functioning of the current Back Channel. When the Mfolozi water level would rise up to 1.5 meter above GMSL it would probably breach naturally. As a result the functioning of the Back Channel decreases even further. Especially during winter the St Lucia lakes will face a lot of salinization. This would have negative impacts on the marine life and animals in and around the lakes. Due to a closed St Lucia mouth and the drought the Lakes could become separated. The lakes will dry further up and a sufficient distribution of water and species becomes almost impossible.

As a result of economic decline, tourism profits decrease and there are no or minimal funds available to make a difference. Local economies need to be stimulated. Illegal poaching and criminality becomes a bigger problem. In addition, labour at the sugar cane farms and plantations on the lower floodplains become more interesting as well. An increasing interest of the sugar cane production could allow artificial breaches (even for water levels below the 1.5 meters above GMSL) if the water level in the Mfolozi backs up to far inlands. Lower water levels in the Mfolozi will not stimulate the functioning of the Back Channel.

For the midterm, if water levels remain low and the drought persists, breaching the St Lucia mouth would become inevitable. This latter would lead to an increased inflow of sea water into the lakes. On the other hand, this would also increase higher salinity levels in the lakes. Wildlife will face dead or have to travel to another place to survive.
As a consequence of the economic decline funds are not available to sustain the functioning of the park. Sugar cane farmers become more powerful due to their contribution to local economies. Necessary changes to provide a fresh water inflow from the Mfolozi to St Lucia become almost impossible because raising the water levels of the Mfolozi evokes a lot of resistance from farmers and local workers. The park could not provide sufficient forms of income through tourism. The park probably cannot meet the criteria of shifting salinity states, huge numbers of waterfowl and large breeding colonies, and diversity of habitats of the UNESCO World Heritage Convention. Eventually, the iSimangaliso wetlands park will be degraded and could lose their UNESCO status. As a result of all above, the area would not be interesting enough for tourism. Local economies have to be stimulated even more. If the tourism branch is not sufficient enough to provide enough incomes, other ways of making money become more interesting and even inevitable. Mining, farming, and wood plantations are such alternatives. Even poaching would be inevitable. The boundaries of the park could be reduced and the iSimangaliso Wetlands Park would probably be split up in smaller reserves.

D.4.5 “Hard battle”
The scenario is based on longer wet periods with ecological improvement together with an economic decline. The Mfolozi system would breach more often as a result of a bigger outflow and the mouth would stay open for a longer period. This would have a negative impact on the current Back Channel. Only during very long wet periods the St Lucia lake system would have enough outflows to breach the St Lucia mouth without any inflows of the Mfolozi. During these long wet periods the Lake system could function properly. Fishery would not be stimulated a lot because the mouth probably would not stay open for a long time.

The iSimangaliso Wetlands Park could grow further in size and in wildlife, and finally becomes more interesting for tourism. However, due to the economic decline, profits remain behind. Tourism facilities become neglected, developments stay behind, park boundaries become unprotected, criminality becomes an issue, and the tourism will even drop further down. Other ways of making money are becoming more important than the development of the iSimangaliso Wetlands Park.

On the other hand, sugar cane farming and plantations become more and more important to stimulate the economy. Farming activities provide work for the local communities. Lower floodplains could be used for sugar cane farming as well. The mouth of the Mfolozi would be breached more often or would even have to open at all times. The impact of the current Back Channel would become inadequate and the St Lucia estuary and lakes are dependent on the rainfall in and around the other catchment areas.

For the short term this situation could be sustained, but when profits remain behind for a longer period of time people can become impatient. Local communities would like to use the land, owned by iSimangaliso, for more profitable activities like farming, poaching, or plantations. Local communities will use the fertile grounds for agriculture in order to make some money. Even in worse case the government could stimulate sugar cane farming or forces the operation of mining activities in the dunes to stimulate the local economy. This would have again a negative impact on the ecological state of the park.

D.5 Measures of effectiveness and suitability
Measures of effectiveness and suitability define quantitatively how well the system has to perform to the expectations. It is a more detailed quantification of the expectations mentioned in paragraph D.1. Only measurable elements have been described.
iSimangaliso Wetland Park Authority

1. The total amount of fresh and clear water delivered to the iSimangaliso wetlands park should be minimal in the order of 30 to 60 million $m^3/\text{s}$ a year. (Lawrie & Stretch, Evaluation of the short-term link between the Mfolozi Estuary and St Lucia Lake, 2008)

2. The ecologic state of the wetlands can be measured by counting populations of different species in the park. One should also look at original vegetation and wildlife with respect to the 1900's. The presence of alien species and to what extend could be used as a measurement.

3. Measuring the amount of fish, larvae and zooplankton passing the construction.

4. Measuring and surveying the turbidity of the water passing the construction.

5. Amount of local support and general opinion about the project. Also the amount of required local labour is a measure of effectiveness.


7. The lifespan of the materials compared to the desired lifespan of the solution. The amount of dredging and removal of vegetation during lifetime.

8. Predicted lifespan and amount of work required to restore project site in its original state.

9. The total construction costs and components involved are a measure of costs and complexity.

10. The number of EIA's performed.

11. Check solution with regulations.

Ezemvelo KZN Wildlife

1. See paragraph D.5.1 Additional measuring is annual tourism income.

2. To what extend can local people maintain the Back Channel and weir.

3. Can the required equipment reach the project site?

4. Deterioration of habitat.

D.6 System boundaries

A project can be expressed in geographical boundaries. Often these boundaries are not sufficient enough. A systems’ boundary goes beyond geographical boundaries. System boundaries define what part of the elements is controllable by the engineer, and what part is not. In addition the expected interactions between the different elements are mentioned. The system boundaries are described in chapter 2.1 in the main report.

D.7 Interfaces

Interfaces can be considered from an internal or external point of view. The internal point of view address elements inside the system boundaries and external interfaces address the relationships outside the system boundaries. Internal interfaces are controlled by the client/contractor and external interfaces are controlled by the government/principal.

Internal interfaces within the project are:

- Flow of water between Mfolozi and The Narrows;
- Interaction of marine species between Mfolozi and The Narrows;
- Sediment exchange between Mfolozi and The Narrows;
- Water levels at the floodplains near the sugar cane farmers;
- Commercial profit and conservation of iSimangaliso wetland park;
- Interaction between the old Back Channel and the new “Waterdraer”;
- Interaction between the buffer area of the Mfolozi and the Back Channel;
Interaction of the subsoil and the superstructure.

External interfaces within the project are:
- Special South African legislation on UNESCO World Heritage sites;
- Ramsar wetlands regulations;
- Public opinion amongst sugar cane farmers.

**D.8 Utilization environments**

The utilization environments concern the environments for each operational scenario. Eight distinct scenarios will be elaborated briefly.

**Scenario 1: Kruger 2**
- Wet periods compared to the mean situation;
- More floods;
- Reduced soil strength;
- Big outflow of water;
- Increased chances of naturally breaching the mouth;

**Scenario 2: Save the hippo**
- Dryer periods compared to the mean situation;
- The muddy soil turns into a hard, cracked groundcover;
- High content of chlorides in the environment;
- Closed mouth;
- Salinization of the lakes;
- Area turns into a dessert like area;
- Change of vegetation.

**Scenario 3: Alternative goldmine**
- Dryer periods compared to the mean situation;
- The muddy soil turns into a hard, cracked groundcover;
- High content of chlorides in the environment;
- Park lands are used for agriculture in order to survive;
- Salinization of the lakes;
- Change of vegetation;
- Area turns into a dessert like area;
- Mining in the dunes.

**Scenario 4: Hard battle**
- Disserted (tourism) villages;
- Wet periods compared to the mean situation;
- More floods;
- Reduced soil strength;
- More sugar cane activities;
- Mining activities in the dunes.

**D.9 Life cycle process concepts**

The life cycle process requirements require the use of integrated teams representing the eight primary functions. This latter are (1) Development, (2) Construction, (3) Deployment, (4) Operation, (5) Support, (6) Disposal, (7) Training, and (8) Verification. The purpose of this step is to define key life cycle process requirements. Emphasize should lie on cost drivers.
and higher risk elements that are anticipated to impact supportability and affordability over the useful life of the system.

1. Development;
   Stakeholders must be involved as much as possible. Proper stakeholder involvement results in a win-win situation for all parties.
2. Construction;  
During construction as little damage as possible should be done to the environment. Construction works should comply with guidelines as prescribed by the EIA and UNESCO.

3. Deployment;  
After deployment, focus must be put on training local wildlife rangers on how to maintain and operate the structure.

4. Operation;  
Guidelines must be made stating how to operate the proposed solution. These rules must be adhered to at all time to assure a proper functioning of the system.

5. Support;  
A maintenance plan including a maintenance schedule should be included. Maintenance plan should be easy to read and should include schedules stating when times visual checks should be done, when certain materials should be ordered or when maintenance should be performed.

6. Disposal;  
Proposed solution should be completely reversible within a limited timeframe.

7. Training;  
Training schedules should be made in order to achieve and maintain the knowledge and skill levels necessary to efficiently and effectively perform operations and support functions.

8. Verification.  
During design, construction, operation and disposal of the proposed solution several regular checks should be done in order to assure a problem free course through the several steps of the life cycle.

**D.10 Functional requirements**

Functions describe what the system must accomplish or must be able to do. The functions are being described for the subsystems. By decomposing the subsystems into sub-subsystems the level of detail for the functions can be defined as well.

- **Floodplains**  
  - Sugar cane fields  
  - Nature  
  - Flood protection  
  - Spillway  
  - Irrigation drains  
- **Mfolozi**  
  - Waterway  
  - Barrier between sea and Mfolozi  
- **Msunduze**  
  - Waterway  
  - Main drainage channel during flood  
- **St Lucia Estuary**  
  - Living habitat for animals  

<table>
<thead>
<tr>
<th>Function</th>
<th>Category</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sugar cane fields</td>
<td>Agricultural</td>
</tr>
<tr>
<td>Nature</td>
<td>Ecological</td>
</tr>
<tr>
<td>Flood protection</td>
<td>Safety</td>
</tr>
<tr>
<td>Spillway</td>
<td>Overflow</td>
</tr>
<tr>
<td>Irrigation drains</td>
<td>Irrigation</td>
</tr>
<tr>
<td>Waterway</td>
<td>Runoff</td>
</tr>
<tr>
<td>Barrier between sea and Mfolozi</td>
<td>Retaining water</td>
</tr>
<tr>
<td>Waterway</td>
<td>Runoff</td>
</tr>
<tr>
<td>Main drainage channel during flood</td>
<td>Runoff</td>
</tr>
<tr>
<td>Living habitat for animals</td>
<td>Ecological</td>
</tr>
</tbody>
</table>
- Link between narrows and sea  
  - Barrier between sea and Estuary  
  - “Waterdraer” between the Mfolozi Estuary and St Lucia Estuary  
    - Link between Estuaries  
    - Inlet  
    - Outlet  
    - Waterway  
- Existing Back Channel(s)  
  - Link between Mfolozi and St Lucia  
- Existing link-channel  
  - Old solution  

D.11 Performance requirements
In this step emphasis lies on the performance requirements for each higher-level function performed by the system. Focus should be placed on the measures of effectiveness and key performance parameters.

- Floodplains located by the sugar cane farmers  
  - Maximum level before flooding is 1.3 meters above GMSL (Stretch, 2011).  
  - Hypsometric curve of the floodplains: Figure D-3

Figure D-3 Hypsometric Curve (Stretch, 2011)

- Mfolozi  
  - Cross section; not used in the design
- Surface area: 1.2km² (Lawrie & Stretch, Evaluation of the short-term link between the Mfolozi Estuary and St Lucia Lake, 2008).
- Maximum flow rate of the Mfolozi is 1100m³/s (De Jager, 2011)
- Max 800m³/s before overtopping the spillway (Kelbe & Taylor, 2011)
- Mean annual sediment load 0.68*10^6tons (Whitfield & Taylor, 2009)
- Grain sizes; not used in the design
- Flow rate: readings of the W2H032 gauge (DWAf, 2011)
- Flood return Periods: Figure D-4
- Overflows sandbank at water level of 3.2 till 3.7 m above GMSL (Taylor, The Mfolozi Floodplain: water and sediment processes, 2011a) (Huizinga & Van Niekerk, 2005)

![Flood Return Periods - Umfolozi River](image)

**Figure D-4 Frequency of floods of various magnitudes in the Mfolozi River (BateG.C., WhitfieldA.K., ForbesA.T., 2011)**

- Msunduze
  - Cross section; not used in the design.
  - Roughness; not used in the design.
- St Lucia Estuary
  - Evaporation rate of 5mm per day (Bate, Whitfield, & Forbes, 2011)
  - Natural overflows sandbank at water level of 2.45m above GMSL (Lawrie & Stretch, Evaluation of the short-term link between the Mfolozi Estuary and St Lucia Lake, 2008)
- “Waterdraer” between the Mfolozi Estuary and St Lucia Estuary
  - Soil Sample:
    - Angle of internal friction Φ= 27.5°
    - Cohesion c= 2 kPa
    - Specific dry density γ_dry=17.5kN/m³
    - Specific wet density γ_wet=17.5kN/m³
    - Compression index range: 0.027≤Cₜ≤0.19
- Coefficient of secondary compression $C_\alpha = 0.004$
- Swelling index range: $0.025 \leq C_s \leq 0.063$
- Undrained shear strength $0 \leq f_{undr} \leq 10$
- Vertical permeability range $10^{-6} \leq f_{undr} \leq 10^{-6}$
  - Standard Cone Penetration: not able to execute.
- Existing Back Channel(s)
  - Cross section. Variable, modelled in Hec-ras model of Clint Chrystal
  - Roughness; modelled in Hec-ras model of Clint Chrystal
  - Water depth: 60cm; measured during field trip
  - Total discharge: $2.6m^3/s$ measured during field trip
- Existing link-channel
  - Cross section is 30m$^2$ (Lawrie & Stretch, 1D model Mfolozi (Microsoft Excel), 2008)
  - Hydraulic radius 1m (Lawrie & Stretch, 1D model Mfolozi (Microsoft Excel), 2008)
  - Manning n=0.05 (Lawrie & Stretch, 1D model Mfolozi (Microsoft Excel), 2008)

![Figure D-5 Schematic showing key levels required for the functioning of the Back Channel (BateG.C., WhitfieldA.K., ForbesA.T., 2011)](image)

### D.12 Modes of operation

Environmental, configuration, and operational conditions determine the modes of operation for the system products under development. These are not relevant for the draft design. The only modes of operation used for this project depend on the state of the Mfolozi and St Lucia mouths. The mouths are separate and could independently close and open. In Table D-1 different states are represented and these have been described below.

#### Table D-1 Possible combination of mouth states

<table>
<thead>
<tr>
<th>St Lucia</th>
<th>Mfolozi</th>
<th>Open</th>
<th>Closed</th>
</tr>
</thead>
<tbody>
<tr>
<td>Open</td>
<td>A</td>
<td>B</td>
<td></td>
</tr>
<tr>
<td>Closed</td>
<td>C</td>
<td>D</td>
<td></td>
</tr>
</tbody>
</table>

**Scenario A**

Both mouths are open. This situation will only occur during or shortly after wet periods. (Kelbe & Taylor, 2011). Tidal flow can enter both estuary systems and dictates the water level in the tidal region. Tidal range and duration depends on the dimensions of both estuary...
Due to variable water levels caused by tidal differences and wind action limited exchange of water is possible between the two systems.

**Scenario B**
An open Mfolozi mouth whilst the St Lucia mouth is closed occurs during periods in which there is not enough flow to prevent the St Lucia mouth from silting up. The Mfolozi mouth only remains open due to sufficient outflow, a breach after a flood or anthropogenic actions. Only during high tide water levels in the Mfolozi the system might be high enough to meet the threshold value of a proposed solution. This would result in brackish or salt water flowing into the St Lucia estuary.

**Scenario C**
A closed Mfolozi mouth and open St Lucia mouth will only occur during periods of low runoff from the Mfolozi and high runoff from the St Lucia estuary. During these conditions the governing northerly directed long shore transport could cause the Mfolozi to close sooner than the St Lucia mouth. Water will rise on the Mfolozi side and once the threshold value has been reached water will flow through the Back Channel into The Narrows. Right after closing off the Mfolozi and during high tide a reversed flow pattern might occur: water flows from The Narrows into the Mfolozi.

**Scenario D**
The most frequently occurring behaviour of the system is the situation in which both mouths are closed. (Lawrie & Stretch, Evaluation of the short-term link between the Mfolozi Estuary and St Lucia Lake, 2008), (Kelbe & Taylor, 2011). Closure of both mouths results in a build-up of water on the Mfolozi side whilst the water level in the St Lucia estuary will drop due to low inflow rates and high evaporation rates. As soon as the threshold value has been reached water will flow through the Back Channel from the Mfolozi into The Narrows.

**D.13 Technical Performance Measures**
Technical performance measures are measurements that are made during the design and manufacturing process to evaluate the likelihood of satisfying the system requirements. Technical performance measures are usually linked to high risk requirements. Technical performance measures can be expressed in values like “the strength of concrete”. Due to the nature of this task technical performance measures are not required.

**D.14 Physical characteristics**
Focus must lie on the use of soft materials as opposed to hard materials like concrete and steel. The construction must blend with its surroundings to prevent “sight view pollution” Furthermore it must be easy to restore or reshape the structure after a big flood. The construction also needs to be able to remain stable during normal conditions.

**D.15 Human Factors**
The final step of the systems engineering process as described by the Department of Defence (U.S.) is the definition of human factor considerations. Human factor considerations are physical space limits, climatic limits, ergonomics, etc. that will affect the operation of the system products under development.

In the case of the St Lucia estuary system one can consider the young mangrove forest in the Back Channel to be a side effect of the Back Channel or as highly valuable nature which ought not to be damaged during construction works. It depends on peoples opinion whether parts of this mangrove forest can be sacrificed for a solution to the problems in St Lucia.
Another factor that has to be considered is the illegal breaching of the Mfolozi mouth by the sugarcane farmers. If their lands are flooding and the berm remains closed they could take the right into their own hands.
E. FUNCTIONAL ANALYSIS

Table E-1 shows the different interfaces between the different objects. The objects follow from the functional analysis in the main report. The interfaces could be used as a supportive tool during the design synthesis. In addition, the interfaces have to be verified whether or not they comply with the requirements.

Table E-1 Object interfaces

<table>
<thead>
<tr>
<th>Code</th>
<th>Interface between</th>
<th>Interface demand</th>
<th>control measure</th>
</tr>
</thead>
<tbody>
<tr>
<td>i100</td>
<td>Inlet - Outlet</td>
<td>The connection between the in- and outlet has to take into account piping.</td>
<td>Verification on requirements</td>
</tr>
<tr>
<td>i101</td>
<td>Inlet - Outlet</td>
<td>The form of the outlet has to be adjusted to the dimensions of the inlet.</td>
<td>Verification on decided parameters</td>
</tr>
<tr>
<td>i102</td>
<td>Inlet - Outlet</td>
<td>The inlet/outlet has to be adjusted in such a way that the functioning of both in/outlet can be reversed.</td>
<td>Verification on requirements</td>
</tr>
<tr>
<td>i103</td>
<td>Inlet - Outlet</td>
<td>The foundation of the in- and outlet is depended on soil properties</td>
<td>Verification on decided parameters</td>
</tr>
<tr>
<td>i104</td>
<td>Inlet - Overflow</td>
<td>The dilatation joint has to be adjusted to the dimensions of the inlet</td>
<td>Verification on decided parameters</td>
</tr>
<tr>
<td>i105</td>
<td>Inlet - Overflow</td>
<td>The dimensions of the inlet have to be adjusted to the capacity of the overflow.</td>
<td>Verification on requirements</td>
</tr>
<tr>
<td>i106</td>
<td>Inlet - Channel</td>
<td>The dimensions of the channel have to be adjusted on the dimensions of the inlet</td>
<td>Verification on decided parameters</td>
</tr>
<tr>
<td>i200</td>
<td>Outlet - Overflow</td>
<td>The dilatation joint has to be adjusted to the dimensions of the inlet</td>
<td>Verification on decided parameters</td>
</tr>
<tr>
<td>i201</td>
<td>Outlet - Overflow</td>
<td>The dimensions of the outlet have to be adjusted on the capacity of the overflow</td>
<td>Verification on requirements</td>
</tr>
<tr>
<td>i202</td>
<td>Outlet - Overflow</td>
<td>The scour and flood protection of the outlet is depended on the overflow</td>
<td>Verification on decided parameters</td>
</tr>
<tr>
<td>i203</td>
<td>Outlet - Channel</td>
<td>The dimensions of the channel have to be adjusted on the dimensions of the outlet</td>
<td>Verification on decided parameters</td>
</tr>
<tr>
<td>i300</td>
<td>Overflow - Channel</td>
<td>The capacity of the channel has to be adjusted on the capacity of the overflow</td>
<td>Verification on requirements</td>
</tr>
<tr>
<td>i301</td>
<td>Overflow - Retention basin</td>
<td>The actual throughput of the overflow is depended on the dimensions of the retention basin</td>
<td>Verification on requirements</td>
</tr>
<tr>
<td>i302</td>
<td>Overflow - Monitoring system</td>
<td>The construction of the overflow has to be adjusted in such a way that the flows and turbidity can be monitored</td>
<td>Verification on requirements</td>
</tr>
<tr>
<td>i400</td>
<td>Channel - Retention basin</td>
<td>The waterlevels in channels could be dependent on the waterlevels in the retention basin</td>
<td>Verification on requirements</td>
</tr>
<tr>
<td>i401</td>
<td>Channel - Monitoring system</td>
<td>The channel has to be adjusted in such a way that the flows can be monitored</td>
<td>Verification on requirements</td>
</tr>
<tr>
<td>i500</td>
<td>Monitoring - Retention basin</td>
<td>The retention basin has to be adjusted in such way that the waterflows can be monitored</td>
<td></td>
</tr>
</tbody>
</table>

The functional flow diagram of Figure E-1 shows what steps are being walked through by the product. It starts with an inflow of water into a retention basin/channel. Sediment has to settle down in this basin/channel. The inflow of water has to be monitored in order to understand the hydraulic dynamics of the system. Further up the water reaches the overflow construction through the inlet. Marine species has to be protected and a distribution of species is preferred. The overflow has to be monitored as well and through the outlet the water reaches the St Lucia estuary.
Figure E-1 Functional flow diagram

Figure E-2 relates the different requirements from the requirements analysis with the different objects from the functional analysis. The figure could be used as a verification tool. It shows which requirements are related to the different objects.
F. VARIANT STUDY

F.1 Options/alternatives and variants

In this step the identification and selection of alternatives has been done. On basis of different options viable variants have been worked out. Reconnecting the Mfolozi with St Lucia estuary can be done in several ways. The additional options elaborated below in F.2 making use of the alternatives of the Back Channel, the existing Link Channel and other alternatives. They will be elaborated briefly in this chapter.

![Diagram of connection alternatives](Google; AfriGIS Ltd, 2011)

**Figure F-1 different connection alternatives**

### F.1.1 Old Main Channel

The first option makes use of the Old Main Channel between the Mfolozi and St Lucia estuary. The new channel will be excavated in the old river contours of the Mfolozi, (yellow line in Figure F-1). This river branch has been cut off in the past by blocking it with an earth dam. An advantage of this option is the already present shallow and dammed low area, which makes the excavation operation easier. A connection with the existing Back Channel can be made in order to reduce the amount of work. The inlet is quite far upstream resulting in higher sediment loads. It is furthermore hard to construct a proper bifurcation there because of the scouring power of the river during high runoff conditions. The length of the channel is quite long, which requires a large head difference to induce flow when vegetation starts to grow in the channel.

<table>
<thead>
<tr>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
<tbody>
<tr>
<td>No mangrove forest present</td>
<td>Labour intensive</td>
</tr>
<tr>
<td>Enough room for wide channel</td>
<td>Expensive</td>
</tr>
<tr>
<td></td>
<td>Turbidity of inflow</td>
</tr>
<tr>
<td></td>
<td>Length of channel</td>
</tr>
</tbody>
</table>

### F.1.2 Improving existing Back Channel

Instead of only using the last part of the existing Back Channel (Green in Figure F-1) one could improve the entire existing Back Channel. In order to increase the rate of flow one should consider widening and/or deepening of the channel. In addition the outlets of the channel have to be examined carefully (black and green in Figure F-1). The presence of
mangrove forests might pose a problem. The inlet of the channel is quite far downstream resulting in a fairly low sediment stream.

Table F-2 Advantages and disadvantages of improving the existing Back Channel

<table>
<thead>
<tr>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
<tbody>
<tr>
<td>Turbidity of inflow</td>
<td>Mangrove forest present</td>
</tr>
<tr>
<td>Length of channel</td>
<td></td>
</tr>
</tbody>
</table>

F.1.3 Beach Channel
Excavating a new channel along the beach is another option (Orange in Figure F-1). Nowadays, the mouth of the Mfolozi is blocked from St Lucia estuary by a sand plug on the beach, which is shown in Figure F-1. One could excavate a channel through this sand plug, and insert a weir to regulate the rate of flow. A big advantage in this option is the reconnection of the Mfolozi and St Lucia in a ‘natural way’. The same kind of linkage existed before man entered this area. The disadvantage of this option is the stability of the sand berm and the soil around the weir. Erosion of the sand berm could result in an unstable construction and widening of the channel. The inlet of the channel is quite far downstream resulting in a fairly low turbidity.

Table F-3 Advantages and disadvantages of the Beach Channel

<table>
<thead>
<tr>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
<tbody>
<tr>
<td>Turbidity of inflow</td>
<td>Stability of soil</td>
</tr>
<tr>
<td>Length of channel</td>
<td>Low lifetime</td>
</tr>
</tbody>
</table>

F.1.4 Restoring the old Link Channel
In the past, a Link Channel was excavated to provide St Lucia estuary with fresh water. Its purpose was to transfer water from the Mfolozi directly into the St Lucia lakes. The channel was abandoned after hurricane Demoina destroyed the inlet structure.

To restore the old Link Channel, several options exist. In addition to clearing the channel from vegetation and removal of the levees used to shut down the channel one must decide on the location of the inlet and outlet of the channel. A few options are possible: Water can flow in the outer bend located at B in Figure F-1 or at the original inflow point at A. The outflow could be located at the original location C or could be joined with the existing Back Channel at D.

The inlet of the Link Channel is located far upstream. During high runoff turbidity of the water will be higher than in the lower floodplains. Besides that, the Link Channel is fairly long resulting in a high hydraulic resistance when vegetation is allowed to grow in and along the channel.

Table F-4 Advantages and disadvantages of restoring the old Link Channel

<table>
<thead>
<tr>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
<tbody>
<tr>
<td>Channel already present</td>
<td>Turbidity near inlet</td>
</tr>
<tr>
<td>Lots of space</td>
<td>Length of channel</td>
</tr>
<tr>
<td></td>
<td>Far upstream</td>
</tr>
</tbody>
</table>

F.1.5 Pipeline
Instead of excavating a channel a pipeline could be used to connect the Mfolozi with the St Lucia estuary. Water will flow from the estuary part of the Mfolozi towards the lower lying St Lucia estuary. Sediment could be trapped in the Mfolozi floodplains. If this is not sufficient one could install a sediment filter in the pipeline. During periods of drought it is hard to get
water from Mfolozi into the St Lucia estuary. In order to increase the rate of flow a pumping station could be installed to pump enough water from the Mfolozi into the St Lucia estuary even in periods of drought. Also an extra pipeline could be installed.

<table>
<thead>
<tr>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
<tbody>
<tr>
<td>Easy to construct</td>
<td>Siltation of pipeline</td>
</tr>
<tr>
<td>Easy to expand</td>
<td>Hard to regulate and to maintain</td>
</tr>
<tr>
<td></td>
<td>Can become expensive with certain options</td>
</tr>
</tbody>
</table>

**F.2 Other alternatives**

Besides channels various other alternatives or options can be denoted. The described options can fulfill the same task as the channels as described in section F.1, or different tasks as explained.

**F.2.1 Flood protection on the floodplains**

Flood protection on the floodplains might be required if a solution requires a large retaining basin. To do so without flooding farms levees or heightening of the land could be required. During flood farmers could use pumping stations to get rid of the water. This would turn the lower floodplains into a polder during periods of high water.

**F.2.2 Relocating the farmers**

When the lower farmers are relocated a swampy area could be created. In this area more water can be stored during times of high runoff. Another way to increase the water basin is submerging the area between the Mfolozi and St Lucia. A disadvantage of this option is the amount of labour and the impact on the area because of the size of the formed basin.

**F.2.3 Desalination plant**

If not enough water can be brought to the St Lucia estuary during drought conditions, seawater could be used to counteract the drought and low water levels. Desalination is required to prevent possible hyper saline conditions. This option involves a pipeline, a pumping station and a large desalination plant.

**F.3 Sediment retaining alternatives**

One of the largest fears of the local management agencies is siltation of the St Lucia estuary. Several alternatives could be used to trap sediment and prevent it from flowing into St Lucia. Some solutions match with certain options and other solutions are universal applicable.

**F.3.1 Floodplain silting basin**

During floods water backs up behind the berm of the Mfolozi. The velocity of the water will drop and sediment will settle. The basin is only present when the mouth of the Mfolozi is closed. When the mouth is breached, water will stir up the sediment from the bottom and flush it out to the sea. This system cleans itself during a breaching event.

**F.3.2 Silting basin / sediment trap**

Instead of a natural basin an artificial basin could be used. Somewhere in the channel a basin which is larger and deeper than the normal river cross section will be dredged. The velocity of the water which enters the basin will drop causing sediment to settle. When too much sediment settles down in the basin the sediment should be dredged away.
F.3.3 Sediment filter
Especially in a solution with pipelines, a sediment filter could be applied. This filter should be installed in the pumping station. Sediment filters have a small throughput making large flows difficult.

F.4 Weirs
Weirs can be applied in some variants; some alternatives are elaborated.

F.4.1 Broad crested weir
In a broad crested weir there are no movable parts, which is in favor in this design because blocking by trees or other vegetation is nearly impossible. This weir can be applied for water level regulation and proportional discharge regulation. In order to regulate the discharge a staged weir could be constructed. During low water levels little water will flow over the weir and during higher water levels more water will be able to flow into St Lucia estuary.

F.4.2 Sharp crested weir
A variant of the broad crested weir is the sharp crested weir. The building costs of this weir are smaller because the structure is quite smaller. When the water flows over the weir, it will fall down into the stilling basin. This will cause a lot of turbulence in the water and also decreases the energy head. Also this weir could be constructed as a staged weir.

F.4.3 V-notch weir
The amount of flow through a V-notch weir increases nonlinear with a rising water level. During low water levels little water will enter St Lucia, when enough water has backed up behind the berm water levels will be higher and more water will flow over the weir.

F.5 Variants
The five different connection options which are described in section F.1 are further elaborated in this section. In addition a null variant is elaborated. The variants have been dimensioned coarsely. Some information is given about the environment and roughness of the variants. Furthermore some calculations have been made for each variant. Calculations are done to be able to make a good comparison between the proposed variants.
F.5.1 Description of the variants
In this paragraph the six variants are characterized. Cross-sections are given for all six variants. The cross-sections are assumed to be constant over the length of the channels.

**Null variant**
The null variant is the situation as it was prior to this research: a mangrove overgrown channel with a natural weir between the channel and the St Lucia estuary. Dimensions have been estimated by means of measuring in Google Earth and via estimations received by verbal transmission (Jugwanth, 2011). The cross-section is displayed in Figure F-4. The water, flowing through this variant, experiences friction caused by the mangroves in the channel. This variant has only one natural outflow connection with the St Lucia estuary. This outflow is constricting the flow area of the channel. The length of the channel is approximately 1900 meters (Figure F-5).

![Figure F-4 Cross-section existing Back Channel [mm]](image1)

**Old Main Channel**
The rough dimensions have been derived from the original dimensions of the Old Main Channel (Figure F-6) (Google; AfriGIS Ltd, 2011). The channel is assumed to be prismatic. The side slopes have an inclination of 1:2 because the channel is located in a clayey area and has been formed by scouring water. The slope has degraded over time and became less steep. The channel is assumed to have some vegetation in it resulting in a larger roughness. The length of the channel is approximately 2800 meters. A weir is planned at the end of the channel near the outflow location of the old Back Channel. The remainder of the existing Back Channel will be closed. In order to prevent a bifurcation point the first few hundred meter of this channel follow a different path as compared to the old riverbed. The location of this variant is depicted in Figure F-7.

![Figure F-5 Location of null variant](image2)
Improving the existing Back Channel

This variant is almost the same as the null variant, though there are some differences. First of all a weir is planned at the end of the current Back Channel instead of the "natural" current weir. In addition an extra connection with a weir will be excavated towards the St Lucia Estuary (Figure F-1, and right outlet in Figure F-9). This new connection will have a cross-section as shown in Figure F-8. The weir will be situated in this connection. The side slopes of this channel are assumed to be 1:1, because it is situated in clayey soil. When required, it is possible to deepen or widen the original Back Channel to allow a larger flow of water. The large advantage of this variant over the null variant is the absence of the dimensional constrictions near the current outflow.
Beach Channel

Due to the lower soil stability the prismatic cross-section is expected to have fairly gentle slopes because of possible scouring during flow conditions. It is assumed that the slopes have an inclination of 1:5 (Figure F-10). The hydraulic roughness is low because the channel will not have a lot of vegetation in it due to the low soil stability. The length of this channel is estimated to be 1000 meters. The channel starts near the inflow point of the existing Back Channel at the sea side, and flows into the estuary near the sand plug, see Figure F-11. For this variant a weir and a stilling basin directly behind the weir are situated at the inflow point of the channel. It is expected that the flow through the channel will bring sediment back into suspension. The existing Back Channel will be closed.

Link Channel

Considering the costs and the hydrology of this variant, the best option is to let the revived Link Channel start at B and end in D (see Figure F-1 and Figure F-13). This is the shortest option of the possible combinations at a length of approximately 2800 meters. The cross-section for this variant (Figure F-12) has been estimated at the narrowest part of the old Link Channel with the aid of Google Earth. The channel is assumed to be lined with grass; therefor removal of vegetation from the original Link Channel will be needed. The water will flow into the St Lucia estuary via a weir at the end of the channel. The remaining part of the old Back Channel will be closed off. In order to lower the rate of sediment being transported into the estuary, a stilling basin could be constructed halfway the revived channel. This basin will
lower the TSS somewhat but it also means that regular dredging will be needed to remove the settled soil.

![Cross-section revived link channel](image)

**Figure F-12 Cross-section revived link channel [mm]**

![Location revived link channel](image)

**Figure F-13 Location revived link channel**

**Pipeline**

For this variant the easiest option of connecting the Mfolozi with the St Lucia estuary will be laying a pipeline from the Mfolozi estuary via the beach towards the end part of the St Lucia estuary (see Figure F-14). The hydraulic head needed for this variant can be calculated with formula of Darcy-Weisbach for pipeline losses [1].

\[
\Delta H = f \frac{L}{D} \frac{v^2}{2g} + \sum \xi \frac{v^2}{2g} \tag{1}
\]

In which:
- \(\Delta H\) = hydraulic head loss [m];
- \(f\) = friction factor [-];
- \(L\) = length of the pipeline [m];
- \(D\) = inner diameter of the pipeline [m];
- \(\xi\) = loss coefficient [-];
- \(v\) = fluid flow velocity [m/s];
- \(g\) = gravitational acceleration (9.81 m²/s).

The first part of the formula calculates the head loss caused by friction over the length of the pipeline and the second part calculates the head loss caused by additional losses, like inflow losses, outflow losses and bends. The inflow loss coefficient is set at 0.5 which means that the inflow end of the pipeline has no special inflow construction and just sticks into the water. The outflow loss coefficient has been set at 1.0, which means that the end of the pipeline just sticks in the water, but keeps submerged. The bend in the pipeline has such a large radius that the loss coefficient for the bend has been set at 0. For a first estimation the pipeline length has been set at 1200 meters and the inner diameter at 1 meter. Calculation shows that the flow through the pipeline will be completely turbulent. This has the advantage that
the friction factor for the concrete pipeline is independent of the Reynolds number and can be easily read from the Moody diagram. The friction coefficient is set at 0.026.

F.6 Water balance model set up

In order to compare the flow capacity and hydraulic resistance of the different variants a simple model is required. The system as described in paragraph D.6 is simplified to a water balance model with an inflow and outflow Figure F-15. The inflow is equal to the runoff of the Mfolozi/Msunduze as defined in appendix I. In addition a flood condition will be modelled. The outflow is equal to the outflow over the (natural) weir and evaporation in the retention basin. In the model the outflow over the weir does not limit the flow through the channels. The maximum rate of flow is estimated in paragraph F.6.2 with the discharge formula of Manning. The area and volume of the retention area, also known as basin or floodplain, is estimated by means of a Digital Elevation Model (DEM) in appendix B.6.

F.6.1 Channel simplifications

The flow in the channels is assumed to be steady, this means the properties of the fluid at a single point in the system do not vary with time. This simplification can be made because the timescale of variation is fairly large compared to the time step of the simulation.

The flow in the channels is also assumed to be uniform. Uniform means the flow velocity is has the same magnitude and direction at every single point in a fluid. If this would not be assumed, backwater curves would have to be calculated and thereby assuming a certain weir height. At this stage in the design process not much is known yet about the weir. Therefor uniform flow is assumed.

F.6.2 Flow

The amount of flow between the Mfolozi and the weir site depends on the hydraulic radius, length and roughness of the channel. In order to estimate the flow Q, one can choose between the formula of Chézy, Strickler and Manning. The only difference between these
three equations is the roughness coefficient which depends on the roughness of the channel. The formula of Manning ([2]) is widely used in English Speaking countries. Therefore, Manning will be used.

\[
Q = \frac{1}{n} \cdot A \cdot R^{2/3} \cdot S^{1/2}
\]

In which:
- \(Q\) = Discharge \(\text{m}^3/\text{s}\);
- \(A\) = Area of the channel \(\text{m}^2\);
- \(R\) = Hydraulic radius \(\text{m}\);
- \(S\) = Bed slope [-];
- \(n\) = Manning coefficient \(\text{s/m}^{1/3}\).

The roughness coefficient \(n\) as proposed by Manning is dependent of the general characteristics of the channel being vegetation, soil and the spatial form of the channel. It must be noted that the coefficient is independent of the depth of the channel (Ankum, 2002).

**Friction coefficients in mangrove forests**

Measurements by Wolanski et al (2007) show that Manning friction coefficients vary between 0.2 and 0.6 in mangrove swamps. Measurements were taken at two stations at a distance of respectively 90 and 135 meter from a tidal creek. Taking into account the large water level gradient and low water flow velocity as observed by Wolanski et al (2007) (Figure F-16) one can conclude flow in mangrove swamps is highly frictional. The friction coefficient depends on the type of mangrove forest. In general Pneumatophores forests encounter less resistance than Rhizophora forests. The average resistance coefficient in mangrove swamps is 0.4 \(\text{s/m}^{1/3}\) (Mazda, Wolanski, & Ridd, 2007).

![Figure F-16 Time series plot of water velocity, gradient and Manning coefficient in the mangrove of Nakawa Gawa. (Mazda, Wolanski, & Ridd, 2007).](image-url)
Due to the presence of small channel in the existing Back Channel the average Manning coefficient will be lower than the Manning coefficient used in mangrove swamps. According to Clint & Stretch (2010) a good estimate for the Manning roughness of the mangrove filled Back Channel would be of the order of 0.025 to 0.1 s/m$^{1/3}$. Initial calculations use a Manning coefficient of 0.05 s/m$^{1/3}$ in the existing Back Channel. Manning coefficients of other channels are 0.03 for straight, weedy maintained channels and 0.022 for straight channels without much vegetation (Chow, 1959).

\[\text{Table F-6 Manning coefficients used in model}\]

<table>
<thead>
<tr>
<th>Channel</th>
<th>n value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Null variant</td>
<td>0.05</td>
</tr>
<tr>
<td>Improving the old Back Channel</td>
<td>0.05</td>
</tr>
<tr>
<td>Old Main Channel</td>
<td>0.03</td>
</tr>
<tr>
<td>Link Channel</td>
<td>0.03</td>
</tr>
<tr>
<td>Beach Channel</td>
<td>0.022</td>
</tr>
</tbody>
</table>

F.6.3 Slope

One of the implications of assuming uniform flow is that the energy slope is equal to the bed slope. According to Grenfell et al (2007) the bottom slope in the lower floodplains is 0.02% or 0.0002. To compensate for the assumption of uniform flow a slightly steeper slope $S$ of 0.0005 or 0.5% has been assumed for all variants. Differentiation between the variants can be made by altering the Manning coefficient for friction.

F.6.4 Optimal throughput

The maximum rate of flow in the channels at +1.5m GMSL and +2m GSML can be computed with formula 2 from section F.6.2. The parameters obtained in paragraph F.5 and F.6.2 are used as input values. The bottom of the channels has been fixed at 0 meters GMSL. Additionally the time required until the 60 million m$^3$ threshold has been reached is given. It must be stressed that this is the minimum time required at maximum capacity of the channel and furthermore assuming that the capacity of the weir is sufficient. The rate of flow through the pipeline has been calculated with a head difference of 1 meter. Because of the large diameter of the pipeline and the shallowness of the Mfolozi estuary area the head difference will not be much larger. The rate of flow through the pipeline is 0.61 m$^3$/s, which is fairly low compared to the other variants. The throughput can be heightened by installing pumps or by using more pipelines.

\[\text{Table F-7 Optimal throughput of channels at +2 meter GMSL}\]

<table>
<thead>
<tr>
<th>Channel</th>
<th>Maximum rate of flow [m$^3$/s]</th>
<th>Time till 60*10$^6$ m$^3$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Null variant</td>
<td>15.3</td>
<td>46 days</td>
</tr>
<tr>
<td>Improving the old Back Channel</td>
<td>15.3</td>
<td>46 days</td>
</tr>
<tr>
<td>Old Main Channel</td>
<td>66.5</td>
<td>11 days</td>
</tr>
<tr>
<td>Link Channel</td>
<td>37.6</td>
<td>19 days</td>
</tr>
<tr>
<td>Beach Channel</td>
<td>48.8</td>
<td>15 days</td>
</tr>
</tbody>
</table>

\[\text{Table F-8 Optimal throughput of channels at +1.5 meter GMSL}\]

<table>
<thead>
<tr>
<th>Channel</th>
<th>Maximum rate of flow [m$^3$/s]</th>
<th>Time till 60*10$^6$ m$^3$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Null variant</td>
<td>5.3</td>
<td>132 days</td>
</tr>
<tr>
<td>Improving the old Back Channel</td>
<td>5.3</td>
<td>132 days</td>
</tr>
<tr>
<td>Old Main Channel</td>
<td>40.7</td>
<td>17 days</td>
</tr>
<tr>
<td>Link Channel</td>
<td>23.3</td>
<td>30 days</td>
</tr>
<tr>
<td>Beach Channel</td>
<td>27.3</td>
<td>26 days</td>
</tr>
</tbody>
</table>
F.6.5 Computation
Based on the data of the foregoing sections a water balance model has been set up to make some preliminary calculations on the performance of the different channel variants. The derivation of the input data for flow is described in appendix B.3. First some remarks will be made on the used input data. Secondly the results of the calculations will be described.

In order to do some preliminary calculations on the performance of the different channel variants six runoff peaks have been chosen from the compiled dataset of appendix B.3.5. These peaks have been chosen such that the spread is relatively large and that smaller peaks are not taken into account. These smaller peaks \( (Q < 80 \text{ m}^3/\text{s}) \) are considered as normal runoff. The six peaks lie within a range of 80 till 560 \text{ m}^3/\text{s}, which have a 1-3 year return period (De Jager, 2011).

The water level at the start of the modelled periods is set at 1.20 meter + GMSL, which is the same as the level of high water spring without additional run-up at the beach. The first set of calculations was done with the berm level at 1.50 m + GMSL, which is the preferred berm level according to the sugar cane farmers. These led to failure of all variants before the end of the smallest peak (99 \text{ m}^3/\text{s}). The storage behind the berm for this berm level is too small to contain the volume of a small peak. From these first results it can already be concluded that the berm should not be artificially breached at a water level of 1.5 m + GMSL for the channels to work properly. Therefore the berm level has been reset to 2.00 m + GMSL in consultation with professor Stretch for the second set of calculations. The results of this second set can be found in the next section.

F.6.6 Results and discussion
The results of the model are displayed in Table F-9. The result figures to calculate the performance of the channels can be found in appendix I.1 Green shading means the variant is able to cope with the run-off peak, red shading means the berm breaches before the end of the run-off peak and orange shading means the berm just breaches before the end of the run-off peak. From the results it can be concluded that the Null variant and the Improved Back Channel do not perform well. The capacity of the Back Channel without removal of the mangrove forest is too low to cope with the smallest assessed peak. The other variants can easily cope with the 99 \text{ m}^3/\text{s} peak and the 121 \text{ m}^3/\text{s} peak.

Another remark is that the Old Main Channel and the Beach Channel only just breach at a peak of 223\text{ m}^3/\text{s}, which is a runoff peak with a narrow double peak. The peaks of 138\text{ m}^3/\text{s} and 183\text{ m}^3/\text{s} also have double peaks, but are a lot broader, which means that high flow rates occur over a longer period. Because these variants fail only just during a large but narrow peak and fail during smaller and broader peaks, it is believed that these variants are able to cope with single peak run-off with a maximum between 120 and 220\text{ m}^3/\text{s}. Such data was not available from the compiled dataset and therefore has not been tested. Also the Link Channel variant is believed to be able to cope with a single peak in this range.
Table F-9 Performance of channels during historical peaks (green: berm does not breach, orange: berm only just breaches, red: berm breaches)

<table>
<thead>
<tr>
<th>Peaks</th>
<th>99 m³/s (single peak) Apr-2006</th>
<th>121 m³/s (single peak) Nov-2005</th>
<th>138 m³/s (double peak) Feb-2009</th>
<th>183 m³/s (double peak) Okt-2007</th>
<th>223 m³/s (double peak) Feb-2002</th>
<th>565 m³/s (single peak) May-1995</th>
</tr>
</thead>
<tbody>
<tr>
<td>Null variant</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Old Main Channel</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Improved Back Channel</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Beach Channel</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Link Channel</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

F.7 Evaluation of variants

F.7.1 Value

The trade-off matrix is used as a tool to evaluate the different variants. The criteria of the trade-off matrix for value are defined by the functional values. These have been separated in “experience” values, “use” values, and “technical” values.

Experience as a category of functional values can be described as an impression, feeling, appearance, emotion, and many more. In dictionary people could find the definition of experience as “The apprehension of an object, thought, or emotion through the senses or mind”. (The American Heritage®, 2009) Secondly, the functional values of the category “use” define all possible functional values of the users of the end product. Finally, any civil engineering project has functions on a technical level. These values describe the things that a system has to do on a technical basis. These three categories will ultimately create the criteria of the trade-off matrix.

Table F-10 functional criteria

<table>
<thead>
<tr>
<th>Experience:</th>
<th>Use:</th>
<th>Technique:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Ecological linkage: Retaining:</td>
<td>Habitat Protection Flood sediment transport</td>
</tr>
<tr>
<td></td>
<td>Species Distribution Retaining sediment</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Control:</td>
<td>Water transport</td>
</tr>
<tr>
<td></td>
<td>Maintainance Monitoring</td>
<td></td>
</tr>
</tbody>
</table>

In Table F-10 the used functional criteria are represented. The column of experience remains empty due to the fact that there are too many different expectations to take into account for the relative small size of this project. In Table F-11 the weights of the different functional criteria are shown. These weights are based on the opinions of the project group. Reading in a horizontal way shows that a ‘1’ means that the criteria on the left is more “important” then the criteria in the upper row. These weights have been used as an input to evaluate the different variants. In Table F-12 preferred order of the variants concerning the different functional criteria is shown. In the latter a ‘1’ stands for worse and a ‘6’ stands for better.
Table F-11 weight reference

<table>
<thead>
<tr>
<th>Trade-off weights</th>
<th>Recruitment</th>
<th>Maintenance</th>
<th>Sediment</th>
<th>Water transport</th>
<th>Outcome weights</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ecological</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>16.67%</td>
</tr>
<tr>
<td>Control</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0.00%</td>
</tr>
<tr>
<td>Retaining sediment</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>33.33%</td>
</tr>
<tr>
<td>Water transport</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td></td>
<td>50.00%</td>
</tr>
</tbody>
</table>

Table F-12 Trade-off matrix variants

<table>
<thead>
<tr>
<th>TRADE-OFF</th>
<th>Weights</th>
<th>null variant</th>
<th>Old main channel</th>
<th>Improving existing backchannel</th>
<th>Beach channel</th>
<th>Link channel</th>
<th>Pipe line</th>
</tr>
</thead>
<tbody>
<tr>
<td>Experience</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Use</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ecological linkage</td>
<td>16.7%</td>
<td>2</td>
<td>5</td>
<td>3</td>
<td>6</td>
<td>4</td>
<td>1</td>
</tr>
<tr>
<td>Control</td>
<td>0.0%</td>
<td>6</td>
<td>3</td>
<td>5</td>
<td>1</td>
<td>2</td>
<td>4</td>
</tr>
<tr>
<td>Technique</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Retaining sediment</td>
<td>33.3%</td>
<td>5</td>
<td>3</td>
<td>4</td>
<td>2</td>
<td>1</td>
<td>6</td>
</tr>
<tr>
<td>Water transport</td>
<td>50.0%</td>
<td>2</td>
<td>6</td>
<td>3</td>
<td>5</td>
<td>4</td>
<td>1</td>
</tr>
</tbody>
</table>

However, evaluating the variants based on this order is not a good way. This ordinal scale range does not take into account the scale ranges between the different variants. For example, how does this rating distance represent the actual distance between the variants? If the null variant is ranked as 6 and the Link channel as 3 the assumption that the null variant is twice as good as the link channel is not correct. Therefor the program Tetra is used in order to take into account the rating “distances”. This is the only software for evaluation, measurement and decision making that is based on sound mathematical foundations. The program constructs preference scales using a user’s criteria weights and alternatives’ ratings. The program input consists of specifying the alternatives and the criteria and sub-criteria on which the alternatives are rated. The alternatives are rated on the criteria in a simple and intuitive manner and the criteria are weighted by their relative importance. Tetra then rates the alternatives on a scale that takes into account all these pieces of information. (Scientific Metrics, 2011)

The “value” results of Tetra compared to the excel table are represented in Figure F-17. An interesting remark considering these outcomes is that the differences between the score ranges is bigger in the Tetra outcome then in the excel table. So taking into account their relative importance could have an impact on the outcome. This is an importance advantage of Tetra in comparison with the ordinal ordering of the excel trade-off table. In this project the Tetra outcome shows that the value of the “Old Main Channel” variant is higher compared to the other variants. The main criterion of water transport is the reason for this high score.

![Figure F-17 Outcome trade-off Excel versus Tetra](image-url)
F.7.2 Revenues

Most of the revenues are already evaluated in the chapter of value above. In addition there are the revenues of the sugar cane farmers versus the Wetland Park authorities. These revenues are subjective and based on assumptions of different scientific articles. These are briefly described below.

Revenues for the iSimangaliso Wetland Park are mainly related to the amount of water that goes from the Mfolozi into the St Lucia estuary. Van der Waal mentions in her article that through restoring the Mfolozi floodplain into wetlands the total added ecosystem service values (fisheries, tourism, existence, vegetation harvesting, sediment retention, cultural and education, flood alleviation, water purification, water provision) for the St Lucia system would increase by 50%. (Van der Waal)

Related to the variants mentioned in this report the impacts on the St Lucia estuary are hard to define. Therefore only the capacity of the different variants is taken into account (see chapter F.7.1).

Currently, the major parts of the floodplain are being farmed for sugarcane. Some parts of the farmed areas on the lower floodplains fall under the boundaries of the iSimangaliso Wetland Park. These lands are being used for small-scale subsistence farmers as part of poverty relief initiative. (Grenfell, Ellery, & Grenfell, Geomorphology and dynamics of the Mfolozi River floodplain, 2009). The impacts from the different variants on the floodplains are related to the berm height of the Mfolozi, which is for every variant currently the same. The berm height is related to the amount of water that is backing up in the Mfolozi. In this study the economic impacts on these farmlands have not been worked out in this report.

F.7.3 Costs

Cost analysis is kept relatively simple due to the relative simplicity of the technical artefact in this project. The costs of the variants consist mainly of equipment, materials, and labour. By taken into account the dimensions of the different variants compared to the current situation a global ratio of the variants cost has been defined. Therefor there has been made a couple assumptions:

- Soil specifications are unknown for the foundation of the spillway. The risk of different soil conditions has not been taken into account.
- Dimensions of the spillways should be more or less the same and so are the costs.
- The volumes of removed soil have been calculated to compare the amount of labour for excavation. Therefor there has been made assumptions on the current soil levels.
  - The null variant and the improving existing Back Channel variant do not take into account the excavation of the current Back Channel.
  - For the Old Main Channel 50% of the cross section has to be excavated. On basis of satellite photos it is assumed that the current location of the Old Main Channel requires less excavation for the desired cross section because some parts still lay below the water surface.
  - For the improvement of the existing Back Channel, 100% of the new extension has to be excavated.
  - For the Beach Channel 150% of the cross section has to be excavated. This value is assumed higher because of the sandy environment and that the heights of the current land can vary enormous.
  - For the link channel 50% of the cross section has to be excavated. Some parts will require less attention than other parts because they are probably not silted up.
For the pipeline 100% of the cross section has to be excavated.

- The impacts of the sandy environment of the Beach Channel and the pipeline could result in the need to stabilize the surrounding environment. These costs and risks are uncertain and therefore increase both prices with a factor of +1.

The output is used as input in the Tetra program. In Figure F-18 an overview of the global cost ratio per variant is represented. These have been rated in percentiles against each other. An interesting remark is that besides the pipe line variant, the cost ratio per m³ water of overflow does not have a big impact on the rating. Only the Old Main Channel variant scores a little bit better per m³ water than the Beach Channel variant (Figure F-19).

![Variants](image1)

**Figure F-18 Cost ratio estimation**

![variants](image2)

**Figure F-19 Cost ratio per m³ water**

### F.7.4 Scenario testing

The different variants are being evaluated by the different scenarios. Only the short term implications of the scenarios have been evaluated due to the current relative low amount of information that is available. Possible assumed events are described and possible unfavourable situations might show up. For all scenarios the Mfolozi berm height is set on two meters above GMSL. In an early phase of the project there could be anticipated on the outcomes of these possible scenarios. The null variant is described in chapter D.4 as the zero scenario and is therefore not taken into account in this comparison. The different variants are described briefly below. Finally, the different preferred variants per scenario are represented in the four scenarios in Figure F-20.

**Variants**

- The “Old Main Channel” is the variant with the biggest dimensions of all. The water capacity is very big and it is assumed that it could reach an overflow of 60 million m³ water in 11 days when fully operational. Everything above this value can be considered as an additional favourable advantage.
The improvement of the current Back Channel is besides the pipeline the variant with the smallest water capacity. In order to function properly there should be a long back-up of water in the Mfolozi to reach an outflow of 60 million m$^3$ water per year. It is assumed that this amount is reached when the Back Channel is fully operational for 46 days.

The Beach Channel is considered to reach an amount of 60 million m$^3$ water in 15 days when fully operational. This controllable variant has some of the most characteristics of the combined natural mouth conditions.

The link channel could reach an outflow of 60 million m$^3$ water in 19 days when fully functional. This variant was partly built in the 1980s. However, this variant became never operational as a result of the Demoina flood.

The pipeline is considered to be a very effective short term solution. However the required dimension of a pipeline could give some problems. Twelve pipelines with a diameter of 1 meter are assumed to be necessary to reach an inflow of 60 million m$^3$ water in three months to counteract the hyper saline conditions of the St Lucia lakes during droughts.

**Impacts**

The working capacity of all the variants depends on the back up of water into the Mfolozi. When the outflow of the Mfolozi is too high the Mfolozi mouth will breach. During a wet period the Mfolozi would probably stay open for a long time and the backup of water in the Mfolozi is reduced to a minimum. The mouth could also be breached by human interventions when the water backs up to much and sugarcane farmlands are being flooded. This would have a negative impact on the working capacity of the different variants as well.

The bigger the channel the more water and sediment could be transported. Due to their size and locations in comparison to the other variants it is assumed that the Old Main Channel and the link channel are most sensitive for sediment transport during floods. The risk of heavy sediment transport into St Lucia does exist when floods arise and the water levels in the St Lucia lakes are lower than in the Mfolozi. During droughts these inflow of sediments could lead to sediment problems in the St Lucia estuary, Narrows, and even probably the lakes. This could result in inevitable dredging activities in the Narrows.

The Beach and Back Channel variants are considered to be less sediment sensitive compared to the others. As a result of the flocculation near the inlets of these two variants the impacts of sediment could be lower. The sediment through the pipeline is depended on the water velocity related to the hydraulic head. During droughts a small flood from the Mfolozi could transfer a lot of sediment into the St Lucia Estuary when the water levels of the St Lucia Estuary are low. A remark concerning this sediment comments is that these statements are all based on assumptions.

During ecological favourable conditions more water would enter the systems. The water levels could rise and provide better living circumstances for wildlife and vegetation. A big inflow from the Mfolozi towards the St Lucia Estuary is not a crucial need.

During wet periods the water velocity in the Mfolozi could raise a lot. The increased water velocity has a negative impact on the sediment transport. In addition, high water velocities increase the chance of scouring in the channels and therefore could lead to higher maintenance costs. For the Old Main Channel the risk exists that the Mfolozi will search its way through the former inlet of the Old Main Channel. This has to be prevented in order to avoid the whole flood from flowing towards St Lucia. The link channel does not have a retention basin as well and is considered to be sensitive for floods and high water velocities.
The Beach and Back Channel inlets are located at the bottom of the Mfolozi “retention basin” near the sea and are less sensitive for the runoff of floods through the Mfolozi.

However, it could be possible that during wet periods the outflow of the St Lucia lakes could flush away the incoming sediment from the Mfolozi to counteract possible sediment problems. When the water level in the St Lucia systems would become high enough the St Lucia mouth could breach. The natural breaching of the St Lucia mouth would stimulate marine species distribution. During wet periods Beach Channel and pipe line could be extremely influenced by scouring of the silt environment. The surrounding area could flush away when none stabilizing measures are taken.

As the ecology deteriorates a drier environment has a negative impact on the living circumstances for wildlife and vegetation. Inflow of water from the Mfolozi towards the St Lucia system becomes very important to counteract none restorable damage in the environment. If the amount of water inflow to the St Lucia Estuary becomes the low the St Lucia mouth has to be breached to counteract the drought in the Lakes. Salinization would become a bigger problem and the need for fresh water becomes even higher. During dry periods the backup of water in Mfolozi will raise not very fast. The operational time until the berm breaches could take pretty long, which would be in favour of the inflow of water into the St Lucia Estuary. Drier periods could also mean a lower water velocity that could have a positive effect on sediment transport through the channels. However, the inflow of water into the St Lucia system would be very important to counteract the evaporation and the salinization of the Lakes and Narrows. The bigger the dimensions of the variants, the more water could flow into St Lucia. As mentioned before, a risk of these bigger dimensions is that during a small flood in dry periods a lot of sediment could flush towards the St Lucia systems. Therefore the biggest solution is probably not the most preferable in this situation unless several measures are taken to prevent siltation problems.

The economy could also have some major impacts on the functioning of the different variants. During an economic decline sugarcane/farming activities could become more important than the conditions of the Wetland Park. Park developments stay behind. Tourism is failing to provide enough work and money for the local communities. In order to survive locals are moving into the fertile grounds of the Wetland Park. Poverty could stimulate poaching and criminality and could have a negative impact on the reputation of the Park. Besides that, lower water levels in the Mfolozi could provide more area for farming activities. As a result the farmers might push to breach the Mfolozi earlier to protect their fields against flooding. During dry periods a small backup of water in the Mfolozi would be possible, but in wet periods the mouth would stay open longer and the operational periods of the variant can be put under pressure and are reduced to a minimum. In such a situation, a variant with a big capacity would be in favour compared to the others to protect the Wetland Park from deterioration. However, due to the recession there are minimum funds available so an (maintenance) expensive win-win solution has probably no chance of success.

When the economy is growing, the chance of more infrastructures developments in and around the Parks is much higher. The boundaries of the Parks could grow together with the parks’ reputation. Tourism is stimulating the local economies enough to provide them better living circumstances. The importance of a good functioning of the Wetland Park grows and breaching of the Mfolozi would be prevented as much as possible when water inflow from the Mfolozi to the St Lucia Estuary is required. Farmers would have to comply with measures to improve the functioning of the Wetlands Park. This could have a positive impact on the available operational time of the variants. However, this latter is still depended on the weather conditions. Funds for longer term win-win solutions could become available. If the
farmers would block the process to much the government could intervene in the process and push forward a decision that would be in favour of their most important interest. Possible solutions that interfere too much with the habitat are not in favour.

Summing up, the Old Main Channel, link channel, and Beach Channel have an advantage in an environment where droughts are threatening the functioning of the St Lucia Lakes and the fresh water inflow into St Lucia Estuary is preferred to be very important. A high amount of water inflow would prevent salinization and is assumed to improve the ecological situation of the Wetland Park. However, especially for the first two variants the risk of sediment transport during floods is available. The Beach and Back Channel have a lower risk of filling the St Lucia Estuary with sediment because the inlet of the Beach Channel has a more advantageous location (more flocculation) at the end of the retention basin of the Mfolozi.

The improved Back Channel has an advantage in an environment where there is less money available for a suitable solution and where droughts and salinization are threatening the functioning of the St Lucia Lakes. Considering the ecological functioning of the area the Back Channel would only prevent the hyper saline conditions during droughts. It is not assumed that the Back Channel would provide such an inflow that the ecological situation in the area improves. It would only counteract the deterioration as a result of salinization and droughts. Also the distribution of marine species is assumed to be very low in this variant.

The Beach Channel and pipe line variant score very low in wet circumstances due to the higher costs and uncertainties of the instable environment. The instable environment could require a foundation for the variants and makes these more difficult to be reversible. However, under some circumstances the Beach Channel has similarities of a controlled natural situation of a combined mouth estuary. Distribution of marine species is considered to be possible to a certain extent. The Beach Channel has only opportunities in an environment where the controlled combination of mouths is preferred and where money is available for maintenance. Another disadvantage of the pipe lines is that the distribution of species is being obstructed through filters.

**Structuring the variants into the scenarios**

The improvement of the existing Back Channel is the most preferable variant in the “save the Hippo” (orang box) scenario. Chance of sediment transport is low and it would counteract the hyper salinity of the St Lucia system. However, the Beach Channel variant would also be a good alternative. The capacity of the latter is higher and works as a controlled natural situation. It is assumed that the Beach Channel has the opportunity to improve the functioning of the St Lucia system.

In the second “Kruger 2” (green box) scenario the link channel and the Old Main Channel would probably suffer from a lot of scour and sediment transport. The risk that the Mfolozi “breaches” near the inlet of the Old Main Channel could result in a bypass for “sediment-full” water towards the St Lucia Estuary. In addition, the wet conditions require a stable underground for any constructions. In this scenario the improvement of the current Back Channel is preferred above the others for the short term. It has the least impact of the “sediment-full” floods and water inflow from the Mfolozi is not crucial, but is sustained.

The third “Hard battle” (yellow box) scenario the economic decline could lead to a funding problem. There is no money available for any “big” construction or measure or whatsoever. The Mfolozi would remain most of the time open due to floods or human breaches by farmers that protect their lands from flooding. However, like the “Kruger 2” scenario, due to wet conditions there is less inflow required from the Mfolozi towards the St Lucia Estuary. Again
the improvement of the existing Back Channel is preferred for this scenario to maintain the functioning of the Wetland Park.

The fourth and final “Alternative goldmine” (red box) scenario is one of the most negative scenarios. Droughts impede the good functioning of the St Lucia lakes and water inflow from the Mfolozi is crucial to counteract the evaporation and hyper salinization. Due to the low cost the improvement of the Back Channel would again be the most preferable short term solution, but for the well-functioning of the weir the berm height has to be maintained on 2 meters above GMSL. Without a 2 meter high Mfolozi berm the pipeline and Back Channel variants cannot provide enough water inflow. That is why the Beach Channel without an expensive stabilized environment would be the most preferable short term solution in this scenario.

Figure F-20 Variants divided in the different scenarios
F.7.5 Tetra results

Figure F-21 Criteria weighting

Figure F-22 Rating ecological function criterion
Figure F-23 Rating sediment criterion

Figure F-24 Rating water transport criterion
Figure F-25 Rating cost criterion

Figure F-26 final result Tetra based on costs and values
G. FINAL DESIGN

During the design synthesis it is important to examine if the designed objects still fulfill the desired functions as defined in the earlier phases. In Figure G-1 the functional/physical matrix is represented. This matrix has been used as a management tool to check the functions of the objects.

<table>
<thead>
<tr>
<th>Function performed</th>
<th>Ecological linkage</th>
<th>Protection habitats</th>
<th>Distribution of species</th>
<th>Control</th>
<th>Maintaining</th>
<th>x</th>
<th>x</th>
<th>x</th>
<th>x</th>
<th>x</th>
<th>Retaining</th>
<th>Sediment retention</th>
<th>x</th>
<th>x</th>
<th>x</th>
<th>Sediment during flood</th>
<th>x</th>
<th>x</th>
<th>Water transport</th>
<th>x</th>
<th>x</th>
<th>x</th>
<th>x</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water transport</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td></td>
<td></td>
<td></td>
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<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Sediment retention</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
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<td></td>
<td></td>
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<tr>
<td>Water transport</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
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<td></td>
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</tr>
</tbody>
</table>

*Figure G-1 Functional/physical matrix*
G.1 Berm height

This chapter contains the basic reasoning leading towards the design height and width of the weir. In addition advice is given on the maximum height of the Mfolozi berm and corresponding water levels related to weir discharge. Figure G-2 shows the project location consisting of the New Channel, Old Channel, and the mangrove forest lined connection.

![Figure G-2 Areal image of the new channel, old channel and existing back channel](image)

#### G.1.1 Retention area and volume

The maximum natural berm height is +3.5m GMSL, which is equal to the equilibrium berm height. There is no minimum berm height, but currently the berm is artificially breached when the water level rises above 1.5m GMSL. In Table G-1 the size of the retention basin and the corresponding berm height are given. The size of the retention basin is based on the hypsometry as proposed by Chrystal & Stretch (2009) in paragraph B.6.1. In addition the maximum volume that can be stored just after closure of the berm is given. This value is obtained by subtracting the volume of the basin right after closure from the volume at a certain berm height. The volume of the basin right after closure is equal to the base capacity at +1.20m GMSL as described in Appendix I.1.

<table>
<thead>
<tr>
<th>Berm height [m] above GMSL</th>
<th>Retention basin volume [m$^3$]</th>
<th>Storage after closure [m$^3$]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.20</td>
<td>2.2 * 10$^6$</td>
<td>0</td>
</tr>
<tr>
<td>1.50</td>
<td>3.7 * 10$^6$</td>
<td>1.5 * 10$^5$</td>
</tr>
<tr>
<td>2.00</td>
<td>12.03 * 10$^6$</td>
<td>9.8 * 10$^5$</td>
</tr>
<tr>
<td>2.50</td>
<td>28.58 * 10$^6$</td>
<td>26.4 * 10$^5$</td>
</tr>
<tr>
<td>3.00</td>
<td>48.85 * 10$^6$</td>
<td>46.7 * 10$^5$</td>
</tr>
</tbody>
</table>

The proposed threshold level of artificially breaching the berm depends on two factors; the amount of flooding allowed on the floodplains and the largest flood wave that has to be retained. Flooding has to be kept to a minimum in order to prevent large financial losses for the sugar cane farmers. On the other hand, a larger basin results in less breaching because larger flood waves can be retained inside the basin without overflowing the berm. Less breaching means a longer operational span for the Back Channel and would lead to more fresh water in St Lucia.
G.1.2 Retaining flood waves

The return period of flood waves higher than 50 m³/s has been computed with the 12 minute data obtained from the W2H032 gauge (see paragraph B.3). Only the data obtained from the dry period starting in 2002 until the first of January 2010 has been used. Earlier wet periods have been left out of the return period graph because the proposed solution is designed to counteract drought in St Lucia during dry periods. In total 34 floods higher than 50 m³/s have been recorded during the given period. In Table G-2 those floods have been combined in bins. In Figure G-3 the probability of reaching a certain discharge has been plotted as a Weibull function.

Table G-2 Floods from 1 January 2002 until 1 January 2010.

<table>
<thead>
<tr>
<th>Flow [m³/s]</th>
<th>Number of floods</th>
</tr>
</thead>
<tbody>
<tr>
<td>50 – 70</td>
<td>8</td>
</tr>
<tr>
<td>70 – 100</td>
<td>5</td>
</tr>
<tr>
<td>100 – 150</td>
<td>8</td>
</tr>
<tr>
<td>150 – 250</td>
<td>4</td>
</tr>
<tr>
<td>250 – 500</td>
<td>4</td>
</tr>
<tr>
<td>500+</td>
<td>5</td>
</tr>
</tbody>
</table>

Figure G-3 Probability of maximum discharge in m³/s (Weibull)

Based on paragraph B.3 the method of triangles could be used on small peaks. According to the theorem of triangles the time to peak $T_c$ is 28 hours. The total time of a flood wave is $3T_c$ that is equal to 84 hours. According to this theorem the discharge of the flood waves would be of the same order as shown in Table G-3. According to the 12 minute data obtained from Department of Water Affairs (2011) the average flood with a peak flow of up to 100 m³/s lasted up to four days. Four days equals 96 hours and is consistent with the theorem.

Table G-3 Discharge in 84 hours according to triangle theorem

<table>
<thead>
<tr>
<th>Flow [m³/s]</th>
<th>Total discharge in 84 hours [m³]</th>
</tr>
</thead>
<tbody>
<tr>
<td>70</td>
<td>$10.58 \times 10^8$</td>
</tr>
<tr>
<td>100</td>
<td>$15.12 \times 10^8$</td>
</tr>
<tr>
<td>150</td>
<td>$22.68 \times 10^8$</td>
</tr>
</tbody>
</table>
G.1.3 Flooding
Flooding behind the berm is directly correlated to the berm height. A higher berm height results in a larger setup behind the berm. The lower laying farms are protected by levees and make use of outlet vents (Figure G-4) to vent excess water. Excess water is a result of a high river runoff or rain. As soon as the water level behind the berm rises to +0.6m GMSL the one-way outlet vents stop functioning. This means the outlet vents are only effective during low tide, and the low laying sugar cane farms require an open mouth in order to prevent flooding.

![Outlet vents of low laying sugar cane farmers.](image)

Freshly planted sugarcane can survive up to 72 hours of being submerged. Mature sugarcane could survive up to five days with minimal effects on the yields (I.4.7) (De Jager, 2011). If the sugarcane is submerged for a longer period it will decay causing financial damage. It is important to note that all sugarcane farms are located within the 100 year flood line boundary and can be. However, due to politic influences and historical events it is difficult to force them to move. Alternative solutions in the form of protective measures are also possible. The location of the lower farms (below +1.5m GMSL) is subject to the discussion in Appendix C.

<table>
<thead>
<tr>
<th>Berm height + GMSL [m]</th>
<th>Extend of flooding</th>
<th>Figure</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.50</td>
<td>Current situation, some flooding</td>
<td>Figure G-5 left</td>
</tr>
<tr>
<td>2.00</td>
<td>Moderate flooding</td>
<td>Figure G-5 center</td>
</tr>
<tr>
<td>2.50</td>
<td>Extensive flooding</td>
<td>Figure G-5 right</td>
</tr>
</tbody>
</table>
G.1.4 Conclusion

According to the data shown in Table G-2, a berm with a height of +2.5m GMSL would be able to retain 62% of the floods without breaching. Of course the latter only applies when the retention basin is at its base capacity as the flood arrives. Retaining 62% of the floods would greatly improve the uptime of a Back Channel solution because the Back Channel needs a closed mouth to operate well. Water levels of +2.5m GMSL on the floodplains would also result in extensive flooding on the floodplains resulting in financial damage.

Two third of the floods during the last eight years occurred in December, January, February or March. On average there are four floods a year with a flood being larger than 500m$^3$/s once every two years. With a maximum water level behind the berm or a berm height of +1.5m GMSL even a flood wave with a peak of 50m$^3$/s would (artificially) breach the berm. In its current situation the berm is often artificially breached as soon as the water level behind the berm reaches +1.5m GMSL. Breaching has tremendous impact on the throughput of a Back Channel since it is dependent on backing up water behind the berm.

A berm height of +2.0m GMSL would reduce the risk of breaching due to floods. Floods with a peak height of up to 75m$^3$/s could be retained, which is equal to two out of four a year. In general small floods are not directly followed by large floods. Therefore retaining smaller floods might be favourable. A maximum water level of +2.0m GMSL behind the berm would also increase the level of to which water could back up which increases the flow through the Back Channel.

A higher berm results in higher water levels on the floodplains during floods. Flooding is expected to occur on sugar cane farms when the berm is maintained at +2.0m GMSL. Flooding is not necessarily a bad thing. Sugarcane can survive a flood, but farm yield will drop depending on the duration of the flood and growth stage of the cane. In general flood duration of up to three days is acceptable for freshly planted sugarcane (De Jager, 2011).

The lower sugarcane fields are not able to operate whilst the Mfolozi mouth is closed. Actively maintaining an open Mfolozi mouth is not a desired option in the eyes of biologists and the board of the iSimangaliso Park authority.

It is advised to maintain the Mfolozi berm height at +1.8m GMSL. This imposes that the Mfolozi berm height should be actively maintained in order to keep it at the desired level. The low laying sugarcane farms (below +1.5m GMSL) should be reallocated or they have to take countermeasures to keep the water out. They may have to use higher levees and different drainage systems like pumps instead of valves. After a flood the water level behind the berm should drop from +1.8m GMSL to +1.5m GMSL within a reasonable time frame of five days as described in appendix I.4.7.
G.2 Weir dimensions

The most important object in the system is the weir. The crest height and width of the overflow determine the size of the flow into St Lucia. In addition the height of the overflow also determines whether exchange of water due to tidal influences is possible in both directions. The height and shape of the overflow are based on the following parameters;

- Average operational time in a year.
- Type of weir.
- Average water level during operation.
- Amount of flow wished upon.
- The resonance time required to settle most of the sediment.
- Highest tidal levels and the desired amount of tidal exchange.
- The maximum flow velocity in the existing Back Channel.

G.2.1 Average operational time

The Back Channel only operates when the Mfolozi mouth is closed. The average yearly operational time of the Back Channel depends on the state of the mouth of the Mfolozi. During periods of low runoff that frequently occurs in autumn or winter the Mfolozi mouth closes and river runoff is stored in the retention basin. Recently the berm closed in April/May 2010 and during the 2011 winter period. The Mfolozi mouth remains closed until the next big flood that induces natural or artificial breaching of the berm. During periods of closure water will back up behind the berm and the Back Channel would start operating. The Back Channel should be designed to transfer at least sixty million cubic meters of water a year. In Table G-5 the average flow required to reach 60 million m$^3$ a year is shown.

<table>
<thead>
<tr>
<th>Berm closed in months / year</th>
<th>Required average flow to reach 60 million m$^3$/s a year</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 month</td>
<td>23.2 m$^3$/s</td>
</tr>
<tr>
<td>2 months</td>
<td>11.6 m$^3$/s</td>
</tr>
<tr>
<td>3 months</td>
<td>7.8 m$^3$/s</td>
</tr>
<tr>
<td>4 months</td>
<td>5.8 m$^3$/s</td>
</tr>
</tbody>
</table>

The current Back Channel only functions when the mouth of the Mfolozi is closed. According to Taylor (2011) it is impossible to define an average time of closure due to the large amount of processes involved. The state of the mouth depends on the average runoff, floods, political decisions, alongshore transport and wave action.

In addition it is in favour of the farmers keep the water levels behind the berm as low as possible. After a small flood the water levels behind the berm should drop to an acceptable level as soon as possible, without breaching the berm.

Taken into account these above described conditions leads to the following conclusion: the proposed solution should transport as much water as possible in a small period of time. Based on the previous durations of mouth closure the average flow should be higher than 7.5m$^3$/s.

G.2.2 Weir shape

In Appendix section F.4 the following weir types were elaborated;

- broad crested weir,
- sharp crested weir, and
- v-notch weir.
The most important design parameter is discharge. The weir must be able to discharge as much water as possible into St Lucia at water levels between +1.2m and +1.8m GMSL. In addition the weir should allow water levels to drop from +1.8m GMSL to +1.5m GMSL in 72 hours.

The discharge of a broad and sharp crested weir types scales non-linear with water levels. This can be explained due to the power law in the equation for crested weirs [1]. A sharp crested weir has a slightly higher capacity than a broad crested weir due to the sharp crest. A downside of a sharp crested weir is the higher turbulence encountered on the downstream end. From now on sharp crested weirs and broad crested weirs are combined as “crested weirs”. Flow over a v-notch weir (formula [2]) increases highly non-linear with the head level due to the shape of the weir and.

\[ Q = c \cdot b \cdot H^{3/2} \]  
\[ c = 1.7 \, m^{1/2}/s \text{ (broad crested)} \]  
\[ c = 1.9 \, m^{1/2}/s \text{ (sharp crested)} \]

In which:
- \( c \) = coefficient of discharge [m^{1/2}/s]
- \( b \) = width of weir [m]
- \( H \) = hydraulic head over weir [m]

\[ Q = c \frac{b}{15} \sqrt{2g \tan \left( \frac{\theta}{2} \right)} H^{5/2} \]  
\[ c = 0.59 \, m^{1/2}/s \text{ (broad crested)} \]

In which:
- \( c \) = coefficient of discharge [m^{1/2}/s]
- \( \theta \) = angle of v-notch [°]
- \( H \) = hydraulic head [m]

Discharges have been calculated based on formulas [1], [2], a schematization of a v-notch weir, and a broad crested weir as shown in Figure G-6. Figure G-7 shows the discharge of the different weir types with respect to head level over the weir.
The purpose of the weir and Back Channel is to deliver as much water as possible to the St Lucia estuary. Water levels behind the berm vary between the equilibrium after berm closure of +1.2m GMSL till the berm level. In comparison with a crested weir a v-notch weir requires higher water levels to work. In general, water levels behind the berm are low and only after small floods higher water levels could occur. A crested weir could supply significantly more water to the St Lucia estuary. Due to the higher discharge water levels behind the berm could drop quicker, which is favourable for the sugar cane farms.

Based on discharge a crested weir is the best solution. Whether this is a sharp crested weir or broad crested weir follows from the design of the structure in paragraph G.5.

G.2.3 Control level of weir
The flow over a broad weir responds quadratic to the head level over the weir. This means a slight increase in head level results in a more than linear increase of flow over the weir as shown in Figure G-8.
From paragraph G.2.1 it follows that the capacity of the weir should be as large as possible whilst keeping in mind the constraints of the existing Back Channel. From Figure G-8 follows the crest level of the weir should be as low as possible in order to achieve a large flow. In addition a low crest level would result in a longer time of operation because the weir already starts operating at a low water levels.

The water level in the Mfolozi estuary whilst the mouth is open is on average +0.45 GMSL (Stretch, 2011). Tidal action varies between -0.4 GMSL (LWSE) and +1.2 GMSL (HWSE) according to (w32tide 2011). An example of tidal data is shown in Figure G-9. The estuary mean water level EMWL is equal to +0.45 GMSL.

<table>
<thead>
<tr>
<th>Tide</th>
<th>Level [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>High Water Spring</td>
<td>+1.0 GMSL</td>
</tr>
<tr>
<td>Low Water Spring</td>
<td>-0.6 GMSL</td>
</tr>
<tr>
<td>High Neap Tide</td>
<td>+0.45 GMSL</td>
</tr>
<tr>
<td>Low Neap Tide</td>
<td>-0.05 GMSL</td>
</tr>
<tr>
<td>Estuary mean water level</td>
<td>+0.45 GMSL</td>
</tr>
</tbody>
</table>

The main purpose of the first step of the weir is to allow clear and fresh water to flow into the St Lucia estuary. The water should not flow back from the estuary into the ocean during periods of an open mouth and low tide. In order to prevent backflow of water the height of the crest should be at least +0.45m GMSL. In addition the overflow of salt water should be limited to prevent a steep increase of salinity in the estuary during periods with an open mouth. The height of the existing natural weir is +0.9m GMSL. At a weir height of +0.9m GMSL some salt water will flow into St Lucia during approximately 25% of the high tides (w32tide 2011). Inflow is limited due to the small head level difference and short duration. A weir height lower than +0.9m GMSL will significantly increase the inflow of salt water. A weir level higher than +0.9m GMSL will reduce the inflow of fresh water when the mouth is closed. Estimating the best threshold level of the weir involves a lot of uncertainties. Advice is given to maintain the height of the original and natural threshold level. The weir height of the first step is set at +0.9m GMSL.
G.2.4  Width of the weir
The width of the weir is estimated in section I.4.7 with SOBEK. The weir must act as a control notch, the point that controls the flow in the entire channel. The width of the weir is based on the maximum flow velocity at which erosion occurs in mangrove forests, and on the minimum flow required to fulfill the requirements. The minimum width of the weir can be estimated with formula [1] in paragraph G.2.3 and required flow capacity of 14 m$^3$/s at +1.65 m GMSL. The minimum width is estimated to be 10 meter.

G.2.5  Tidal exchange gap
A tidal exchange between the St Lucia estuary and the ocean could be desired in order to allow migration of marine species between the ocean and the estuary. When the mouth of St Lucia is closed this is only possible when the Mfolozi mouth is open and the Back Channel allows an exchange of flow. In order to keep salinity levels down to acceptable values only a limited amount of salt water is allowed to pass through. Extensive monitoring of the tidal exchange gap would allow biologists to study the effects of a combined mouth on a small scale.

The amount of tidal exchange flow depends on two factors:
- The hydraulic head level difference (non-linear);
- The size of the gap

There are two ways to realize such a tidal exchange gap namely;
- A compound weir type of structure, and
- A pipe or culvert.

The required dimensions of the tidal exchange area are hard to determine without sufficient research and input of biologists. In order to solve this question a variable solution is offered that is easy to alter in the field. In Figure G-10 the tidal exchange gap using a compound weir is elaborated. The depth of the tidal exchange gap can be altered by means of sliding in wooden boards into the sockets found on both ends of the gap. The bottom level of the tidal exchange gap is set at 0m GMSL. The width of the tidal exchange gap is 2.0 meter. The tidal exchange gap can be either completely open (0m GMSL), closed (+0.9m GMSL) or somewhere in between.
In Figure G-10 the tidal exchange gap using a pipe or culvert is elaborated. The pipe or culvert should feature a device to close by hand.

Flow velocity and associated degree of scour is directly connected to the water level head over the culvert or gap. A head level difference of 1.5 meter and associated flow velocities could lead to serious damage. As soon as the mouth of the Mfolozi closes the tidal exchange gap does no longer fulfill its duty. The culvert is preferred over the gap due to the easiness of construction and the absent curved flow pattern around the sand bag construction. In addition a culvert is more robust as compared to a wooden constriction.

A tidal exchange gap or culvert should only be realized if biologists agree. Therefor this culvert is considered as an additional option on the design and it not further elaborated.

G.3 Weir construction methods
This section treats several construction methods for the weir. The weir ought to meet the requirements listed in appendix D. The most important requirements are listed below:

R202) The solution should have a low impact on the local ecology over time. The footprint of the solution should be as small as possible. Mangrove forest should be spared whenever possible.
R401) The solution should be composed of soft materials.
R402) The solution should be such that nature can be restored into its original state at the end of its lifespan or any intermediate moment.

Construction methods that fulfill these requirements make use of soft and natural materials or elements. Sandbags, gabions or wooden structures are considered soft and are elaborated below.
G.3.1 Sandbags

A sandbag weir very is cost-efficient because sandbags are not expensive and can be filled with locally available sand. A sandbag weir is permeable, but the structure becomes almost watertight when polyethylene geotextile is used. Use of sandbags in submerged structures is scarce. In the Changjiang River, China, sandwich bags were used to construct dikes to prevent channel erosion and divert flow (Zhu, Wang, Cheng, Ying, & Zhang, 2004).

**Hydrodynamic characteristics of a sandbag**

Important characteristics of sandbags are the settling time and the critical velocity at incipient motion Figure G-11.

![Figure G-11 Incipient motion and corresponding forces of a sandbag (Zhu, Wang, Cheng, Ying, & Zhang, 2004)](image)

Research on the hydrodynamic characteristics of sandbags has been performed by Zhu et al. (2004). When submerged in water a sandbag is exposed to three forces being; lift force \(F_L\), drag force \(F_D\) and its submerged weight \(G\), shown in Figure G-11. Sliding of sandbags occurs at the specific critical velocity. The lift, drag and submerged weight forces can be written like:

\[
F_D = \beta (G - F_L) \tag{3}
\]

In which \(\beta\) is a friction factor. The forces can be written as

\[
F_D = C_D \rho BD \frac{v_{bc}^2}{2} \tag{4}
\]

\[
F_L = C_L \rho BL \frac{v_{bc}^2}{2} \tag{5}
\]

\[
G = (\rho_s - \rho) g BD L
\]

In which:

- \(C_D\) = drag coefficient
- \(C_L\) = lift coefficient
- \(v_{bc}\) = flow velocity near the sandbag for the critical incipient condition.

By substituting the formulas of the drag force, lift force and the submerged weight the equations can be rewritten into:

\[
\frac{v_c}{\sqrt{\Delta g D}} = \alpha \left( \frac{H}{D} \right)^{1/6} \frac{g}{\sqrt{D}} \tag{6}
\]

In which:

- \(v_c\) = critical depth averaged flow velocity [m/s]
- \(H\) = flow depth [m]
- \(\Delta\) = relative density [-] \((\rho_s - \rho)/\rho\)
\[ g = \text{gravitational acceleration [m/s}^2]\]
\[ D = \text{the sandbag height [m]}\]
\[ B = \text{the sandbag width [m]}\]
\[ \alpha = \text{coefficient (\(\alpha \approx 0.44\)) [-]}\]

In Figure G-12 experimental observation for the 0 degree mode (longest axis parallel to the flow direction) and the 90 degree mode (the longest axis perpendicular to the flow direction) are plotted with a linear fit. By means of iteration the dimension for a stable sandbag can be determined with the graph in Figure G-12.

![Graph showing critical velocity for incipient sandbag motion](image)

**Figure G-12 Critical velocity for incipient sandbag motion (Zhu, Wang, Cheng, Ying, & Zhang, 2004)**

**Sandbag structure**

Around the world sandbags are used for various purposes. A lot can be learned from reference projects. One of the projects is a project in Emilia Romagna, Italy. In Emilia Romagna submerged sandbags are used to reduce coastal erosion. (Martinelli, Zanuttigh, De Nigris, & Preti, 2011). The base of the sandbags is very wide to keep the sandbags stable. A cross section is given in Figure G-11.

Table G-7 lists the width-to-height ratios of various reference projects. The width-to-height ratios used in reference projects can be used to estimate a width and height.

![Cross section of submerged sandbags Emilia Romagna](image)

**Figure G-13 Cross section of submerged sandbags Emilia Romagna (Martinelli, Zanuttigh, De Nigris, & Preti, 2011).**

**Table G-7 width-to-height ratios of sandbags in reference projects.**

<table>
<thead>
<tr>
<th>Reference project</th>
<th>Width-to-height ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Emilia Romagna</td>
<td>8.5:1</td>
</tr>
<tr>
<td>Hellevang (minimum)</td>
<td>2:1</td>
</tr>
<tr>
<td>US army</td>
<td>3:1</td>
</tr>
</tbody>
</table>
Elsewhere abroad, sandbags are often used for emergency flood protection. A sandbag dike is easy and fast to construct when fast response is required. The US Army Corps of Engineers recommends a minimum width-to-height ratio of 2:1 for sandbag levees (Figure G-14). (Hellevang, 2011).

![Cross section of a Sandbag levee](image)

**Figure G-14 Cross section of a Sandbag levee (U.S. Army Corps of Engineers, 2004)**

Correct placement of the bags is of great importance. Incorrectly filled bags are permeable allowing water to seep through the levee. One of the most important factors is the bond between the subsoil and the bags. When polyethylene sheeting is used this bond is the weakest spot in the levee. In particular when a horizontal layer is created with polythene, the interface shear strength is much smaller. (Krahn, Blatz, Alfaro, & Bathurst, 2007). Laying the first layers of sandbags below ground level is the easiest way to solve this problem. Sandbags should filled half full, tied near the top and placed according to Figure G-15.

![Placing of the bags](image)

**Figure G-15 Placing of the bags (Hellevang, 2011)**

**G.3.2 Gabions**

Gabions are another economic and durable solution for a weir structure. At a location with limited access gabions are particularly suited because of the ease of construction. Its maximum height is two to four meter. Higher gabions could cause downstream erosion (Tricoli, 2004). Furthermore a gabion is a robust structure, stable, and reasonably flexible. It offers a good alternative for a ‘hard’ structure. (Nautilius, 2011)
Gabion weir alternatives

According to Tricoli (2004) there are four basic ways to construct a weir with gabions. The four alternatives are described below.

**Simple weir**
Weirs without stilling basin could be constructed in a river with a limited flow (see Figure G-17). This structure could only be used if the bed material is resistant enough to prevent scour.

**Weir with counter weir and unlined stilling basin**
The only difference with the simple weir is the counter weir (see Figure G-17). The counter weir reduces erosion caused by energy dissipation downstream of the weir. A counter weir could be constructed when scour related problems arise with a simple weir.

**Weir with counter weir and lined stilling basin**
When the bed material is not resistant enough the stilling basin has to be lined in order to prevent scour from undermining the weir (Figure G-17).
Weir with counter weir and lined stilling basin located below the natural river bed
Almost the same as the previous one, only the lined stilling basin and the counter weir are located below the natural river bed as shown in Figure G-17. This solution is required when the basins functioning is influenced by subcritical flow.

Weir Shapes
The shape of a gabion weir only differs on the downstream end. Possible shapes are vertical, stepped or battered (Tricoli, 2004). The alternatives are elaborated below.

<table>
<thead>
<tr>
<th>CLASSIFICATION OF WEIRS</th>
<th>VERTICAL</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACCORDING TO THE SHAPE OF THEIR DOWNSTREAM SIDE</td>
<td></td>
</tr>
<tr>
<td>STEPPED</td>
<td>BATTERED</td>
</tr>
</tbody>
</table>

Figure G-18 Weir shapes (Tricoli, 2004)

The vertical shape is appropriate when the drop height does not exceed 3 to 5 meter. When the drop height higher a battered or a stepped weir should be used. The battered shaped only functions when the specific flow is lower than 1 m$^2$/s. A stepped weir is suitable when the specific flow is lower than 3 m$^2$/s. Higher specific discharges damage the gabions.

Check weirs
Check weirs could be used on small streams to reduce runoff transportation by decreasing the bed gradient.

Figure G-19 (Kierbeck Thames Limited)

G.3.3 Flexible gabions
Flexible gabions are widely used in coastal and river regions. A flexible structure offers great possibilities with irregular surfaces found in riverbeds or coastal areas. Flexible gabions are cheap and fast to construct. In case of erosion the scour holes can be filled up with new flexible gabions. Flexible gabions are environmentally friendly because the stones feature a porous structure. Small water animals could live and plants could grow inside these porous structures. (Kyowa Co., LTD., 2010)
Choice of flexible gabion

Kyowa Co., LTD. offers three different kinds of flexible gabions. With Table G-8 and Figure G-21 the required gabion is selected based on the flow velocity.

Table G-8 Technical specifications of flexible gabions (Kyowa Co., LTD., 2010)

<table>
<thead>
<tr>
<th>Type</th>
<th>Mesh size</th>
<th>Unit weight, Filter Unit empty</th>
<th>Dimensions in meters, Filter Unit Installed</th>
<th>Applicable velocity</th>
<th>The stuffing material, Particle diameter</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Height (m)</td>
<td>Diameter (m)</td>
<td>Surface (m²)</td>
</tr>
<tr>
<td>Eco-Green 2t</td>
<td>25mm</td>
<td>6kg</td>
<td>0.4</td>
<td>1.9</td>
<td>2.8</td>
</tr>
<tr>
<td>Eco-Green 4t</td>
<td>25mm</td>
<td>13kg</td>
<td>0.6</td>
<td>2.4</td>
<td>4.5</td>
</tr>
<tr>
<td>S Type 8t</td>
<td>50mm</td>
<td>40kg</td>
<td>0.7</td>
<td>3.0</td>
<td>7.0</td>
</tr>
</tbody>
</table>

G.3.4 Wooden weir

One of the most frequently used ways to construct a weir is by using timber. Wooden piles are driven into the ground and crossbeams are placed on the piles to create the weir. If settlement occurs additional crossbeams can be placed to heighten the weir.

G.3.5 Combination of sandbags and timber

The final alternative is a combination of sandbags and timber. Sandbags can be used to stabilize the wooden weir, and offer protection against scour. Sandbag – Timber solutions offer the best of both, being both flexible and robust. The design formulas given in G.3.1 and G.3.4 can be used for this combination option.

G.4 Selecting a weir

Only one of the five weir alternatives discussed in Chapter G.3 can be built. In the following sections the advantages and disadvantages of each alternative are described (G.4.1). In addition a trade-off is made and the best alternative is chosen (G.4.2).

G.4.1 Disadvantages and advantages of weir types

For each method a list of advantages and disadvantages is made.

Sandbag weir

Advantages:

- Simple to construct;

...
• Low costs;
• Flexible structure;
• Sustainable construction method;
• Applicable on an irregular subsoil;
• Most of the labour can be done manually;
• Sand is available at construction site
• Construction in situ;
• Resistant against hippos, and
• Easy to remove.

Disadvantages:
• Easily damaged by a sharp object;
• Relatively large settlements due to high weight;
• A crane is needed to place large sandbags, and
• Hard to create a horizontal crest.

Gabion weir

Advantages:
• Low costs, but more expensive then sandbags;
• Sustainable construction method;
• Durable construction method;
• Recyclable;
• Flexible structure (less than a sandbag);
• Robust structure;
• Stable structure;
• Resistant against hippos;
• Most of the labour can be done manually, and
• The porous structure allows plants to grow on it and fish to live in it.

Disadvantages:
• Can easily be damaged by uneven settlement;
• Needs a lot of rock which is not available nearby;
• A lot of transport is needed which is hard a this area;
• Large settlements due to high weight;
• Hard to remove;
• A crane is needed to place the gabions, and
• Hard to maintain a horizontal crest due to uneven settlements.

Flexible Gabion

Advantage
• Low costs, but more expensive then sandbags;
• Could be used at an irregular subsoil;
• Sustainable construction method;
• Durable construction method;
• Recyclable;
• Almost as flexible as an sandbag;
• Robust structure;
• Relative stable structure;
• Resistant against hippos;
• Most of the labour can be done manually, and
• The porous structure allows plants to grow on it and fish to live in it.
Disadvantages
- Needs a lot of rock which is not available nearby;
- A lot of transport is needed which is hard in this area;
- A crane is needed to place the gabions, and
- Hard to maintain a horizontal crest due to uneven settlements and the flexible structure.

Wooden weir
Advantage
- Simple to construct;
- Relative low costs;
- Little material is needed;
- Construction can take place at the project site;
- Easy to remove;
- Relative light construction method, and
- A wooden weir has a constant horizontal crest.

Disadvantages
- Enhances scour;
- Wood degrades over time;
- Sensitive to damage by hippos, and
- Piles have to be driven into the subsoil.

Combination
Advantage
- Enhanced scour is counteracted by sandbags;
- Easy to remove;
- A wooden weir has a constant horizontal crest;
- Construction can take place at the project site;
- Relative low costs;
- Resistant against hippos;
- Simple to construct;
- Stable structure;
- Most of the labour can be done manually, and
- Sand is available at construction site.

Disadvantages
- Wood degrades over time;
- Piles have to be driven into the subsoil;
- Sandbags are easily damaged by a sharp object;
- Large settlements due to high weight, and
- A crane is needed to place the sandbags;

G.4.2 Choice of the weir
Supported by Tetra a trade-off of the alternatives is made. Six different criteria are used to compute the best alternative. The following subsections elaborate the different criteria and sum up the results of the computation.

Criteria weights
The weight of a criterion indicates which criteria are important and which criteria are not. In Figure G-22 the ratio between the different criteria is shown. The figure shows that
implementability is the most important criterion and durability is considered as the least important criterion.

For the short term solution the durability of the structure is considered the least important criterion. Maintainability is also considered less important because of the link with durability, but is still considered more important than durability. If the structure settles the old situation should be easy to restore.

Costs are considered to be more important than durability, maintainability, and sustainability. Due to the fact that it is a short-term solution it is important that the structure is not too expensive.

Implementability is considered the most important criterion. The project site is hard to reach by car or truck. Manual labour is favoured and cheap. The weir should be easy to construct in situ. The structure should also be easy to remove because it is a short term solution.

Reliability is also an important criterion. Failure could cause unwanted sediment and water inflow into St Lucia. The last criterion is sustainability. This criterion is also considered quite important given the location of the weir.

![Criteria weights in Tetra](image)

Figure G-22 Criteria weights in Tetra.

**Cost**

Figure G-23 shows the rating of the different variants for the cost criterion. The cost of the weir structure depends most on the material used. The gabion variants are pretty expensive because these structures need a lot of rock that is not available nearby. Large transports will be needed. Flexible gabions are more expensive than normal gabions because they need special plastic fabrics. Normal gabions have to be built and filled on the bottom of the new Back Channel. Flexible gabions are filled on the sides of the channel and have to be lifted into place by a crane.

The cheapest variant is the sandbag weir. There is enough sand available at the construction site. To fill the sandbags a lot of manual labour is needed and is inexpensive. Wood is more expensive than sand. Wood also has to be transported to the construction site. Less wood
than stone is needed to construct a weir. The costs of a wooden weir are therefor in between the sandbag weir and the gabion weirs. The combination variant is more expensive than the sandbag structure because it needs wood. The combination variant is considered less expensive than the wooden weir because it needs less maintenance.

![Diagram of weir types and ratings](image)

**Figure G-23 Cost criteria ratings in Tetra.**

**Durability**

Figure G-24 shows the rating of the different variants for the durability criterion. The wooden weir is considered the least durable. The structure is alternately submerged and dry, which will enhance rotting of the wooden beams. A wooden structure is also not very resistant against hippos and will need a relative large amount of maintenance.

Flexible gabions are considered the most durable. Stone is a very durable material and the structure lays stable on the subsoil. Flexible gabions are resistant against hippos. The flexible gabions are made of a plastic. The plastics are elastic and can easily cope with the load of a hippo. Normal gabions are slightly less durable because the wired mesh could corrode.

The sandbag weir is not very durable because sandbags are easily damaged by sharp objects. Still, sandbags are fairly resistant against hippos because they can deform under a load. Geotextile furthermore degrades due to UV light from the sun. When sandbags are combined with a wooden structure the total durability rises. Sandbags can support the wooden structure and protect it from hippos. The sandbags can also protect the wooden structure against enhanced scouring. This will lead to less maintenance over time.
Figure G-24 Durability criteria ratings in Tetra.

**Implementability**

Figure G-25 shows the rating of the different variants for the implementability criterion. The sandbag weir is considered easiest to implement. The sandbags could be filled on site and then be placed with a crane. Gabions are the hardest structures to implement. Stones have to be transported to the construction site over a considerable distance. Non-flexible gabions are not readily available and will have to be constructed at the construction site.

A wooden weir is harder to implement than a sandbag weir because piles have to be driven into the subsoil. Still, this is easier than the construction of gabions. The combination variant is easier to implement then the wooden weir. Because sandbags give the wooden weir more structural stability so that shorter piles have to be driven into the ground.

Figure G-25 Implementability criteria ratings in Tetra.

**Maintainability**

Figure G-26 shows how the different variants are rated for the maintainability criterion. A sandbag structure is easiest to maintain. When a sandbag is damaged or washed and/or the weir is too low, it is easy to replace a new sandbag. For a gabion this is very hard. When a...
flexible gabion is damage it is possible to pull it out with a crane and replace it with a new one. The latter is not really an maintenance friendly measure.

A wooden weir is also pretty simple to maintain. When a wooden beam is damaged it could be replaced by a new beam. The combination variant is even easier to maintain because it is less easily damaged.

![Diagram of weir types]

**Figure G-26 maintainability criteria ratings in Tetra**

**Reliability**

From Figure G-27 it follows that the combination weir is the most reliable. Sandbags provide the wooden weir with stability. Simultaneously the wooden structure provides a reliable flow into St Lucia. When only a wooden weir is implemented little damage could lead to unreliable water supply.Cracks in the structure could lead to a lot of seepage that may deteriorate over time.

Gabions, flexible gabions, and sandbags are less reliable because uneven settlements or moving of components due to flow could lead to large changes in the amount of flow. Flexible gabions are less likely to move due to their flexibility and large weight. These gabions are therefore considered more reliable in providing a set discharge.
The wooden weir scores the best on the criterion of sustainability because no non-indigenous materials are added to the environment. A sandbag weir only adds geotextile of the sandbags as non-indigenous material. Though there is not much sand present at the new Back Channel, it is indigenous for the region.

Flexible and non-flexible gabions consist of rock and are non-indigenous to the region. Flexible gabions are made of plastics that do not degrade over time. The wire mesh of the non-flexible gabions corrodes over time and causes rust. If the wire mesh fails due to corrosion rocks could flow into St Lucia. It is impossible to remove all rocks when they are dumped in the St Lucia estuary.

When all rankings for the different criteria are combined it follows that the combination of a wooden and sandbag weir scores best. This option is elaborated in chapter G.5.
G.5 Design of the weir

This chapter contains the final design of the weir. In chapter G.4 the sandbag timer weir was selected. The overall design of the weir has been discussed in chapter 4.3 of the main report. In following paragraphs the different components are calculated.

G.5.1 Design of the sandbags structure

With Figure G-12 the dimensions of the sandbag can be iterated with the critical depth averaged flow velocity according to the incipient sandbag motion (Zhu, Wang, Cheng, Ying, & Zhang, 2004). The critical depth averaged flow velocity is 4.0 m/s at the top of the weir. This is the most critical point for a sandbag. The flow depth $H$ is 0.4 m, the gravitational acceleration is 9.81 m/s² and the relative density is 1.0. By filling these values in the formulas below, the dimensions can be iterated.

\[
\frac{V_c}{\sqrt{5gD}} = \sqrt{\frac{4.0}{10 \times 3.81 \times 1.5}} \times \left(\frac{1.5}{0.4}\right)^{1/6} = 1.30
\]

\[
\sqrt{L/1.5} = 2.85 \rightarrow L = 12.18m
\]

Following this method sandbags of 1.5 m high and with a length of 12.18 m are needed. This is far too large for a sandbag. For this reason an alternative design has been made.
G.5.2 Design of the wooden structure

The forces on the structure are based on the worst situation to design the structure. There are two possible situations that can be leading. In the first situation the water level at the left side is +1.8m GMSL and the water level on the right side is set at 0m GMSL. This is a hypothetical situation which cannot really occur. For this situation the lowest plank near the bottom is checked.

In the second situation the water level at the left side is +0.9m GMSL and the water level on the right side is set at +0.25m GMSL. This situation occurs when there is just no flow over the weir. For this situation the neutral soil pressure of the sandbags in the weir is taken into account. In this situation the lowest plank at the right side of the wooden weir is checked.

On the left side of the structure the maximum water level and flow velocity are used. On the right side the minimum water level and flow velocity are used. First the dimensions of the planks are calculated. And after that the dimensions of the vertical columns are calculated.

To calculate the dimensions of the columns and of the diagonals the forces in those beams have to be calculated. With those forces the maximum moment and shear force can be calculated. With these values a unity check can be performed to calculate the dimensions of the beams. Below the unity checks are given with its parameters (De Vries & Van de Kuilen, 2010).

**Unity check for the bending strength:**

\[
\frac{\sigma_{m,d}}{f_{m,d}} \leq 1
\]  \[\text{[5]}\]

In which:

\[
\sigma_{m,d} = \frac{M_{x,d}}{W_z}
\]

\[
f_{m,d} = f_{m,0,k} \frac{k_{mod}}{\gamma_m} k_h
\]

**Unity check for shear force**

\[
\frac{\sigma_{v,d}}{f_{v,d}} \leq 1
\]  \[\text{[6]}\]

In which:

\[
\sigma_{v,d} = \frac{3V_d}{2bh}
\]

\[
f_{v,d} = f_{v,k} \frac{k_{mod}}{\gamma_m}
\]

The different parameters needed in formulas above, are given below. First the representative material properties of wood are given in Table G-9 and Table G-10.
Table G-9 Representative values of material properties of sawn poplar wood (EN 1995-1-1 Eurocode 5, 2005)

<table>
<thead>
<tr>
<th>Grootheid</th>
<th>Symbol</th>
<th>C14</th>
<th>C16</th>
<th>C18</th>
<th>C20</th>
<th>C22</th>
<th>C24</th>
<th>C27</th>
<th>C30</th>
<th>C35</th>
<th>C40</th>
<th>C45</th>
<th>C50</th>
</tr>
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<tbody>
<tr>
<td>buigsterkte</td>
<td>( f_{0.85} )</td>
<td>16</td>
<td>16</td>
<td>18</td>
<td>20</td>
<td>22</td>
<td>24</td>
<td>27</td>
<td>30</td>
<td>35</td>
<td>40</td>
<td>45</td>
<td>50</td>
</tr>
<tr>
<td>Volumsche massa</td>
<td>( h_{\text{vol}} )</td>
<td>410</td>
<td>420</td>
<td>450</td>
<td>480</td>
<td>500</td>
<td>530</td>
<td>550</td>
<td>580</td>
<td>600</td>
<td>640</td>
<td>700</td>
<td>800</td>
</tr>
<tr>
<td>volumsche massa</td>
<td>( h_{\text{v}} )</td>
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<td>350</td>
<td>370</td>
<td>380</td>
<td>400</td>
<td>420</td>
<td>450</td>
<td>480</td>
<td>500</td>
<td>530</td>
<td>550</td>
<td>600</td>
</tr>
<tr>
<td>treksterkte (evenwijdig)</td>
<td>( f_{0.85} )</td>
<td>8</td>
<td>10</td>
<td>11</td>
<td>12</td>
<td>13</td>
<td>14</td>
<td>16</td>
<td>20</td>
<td>24</td>
<td>27</td>
<td>30</td>
<td>35</td>
</tr>
<tr>
<td>treksterkte (loodrecht)</td>
<td>( f_{0.85} )</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
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<td>0.8</td>
</tr>
<tr>
<td>druksterkte (evenwijdig)</td>
<td>( f_{0.85} )</td>
<td>16</td>
<td>17</td>
<td>18</td>
<td>19</td>
<td>20</td>
<td>21</td>
<td>22</td>
<td>23</td>
<td>25</td>
<td>26</td>
<td>27</td>
<td>29</td>
</tr>
<tr>
<td>druksterkte (loodrecht)</td>
<td>( f_{0.85} )</td>
<td>2.0</td>
<td>2.2</td>
<td>2.2</td>
<td>2.3</td>
<td>2.4</td>
<td>2.5</td>
<td>2.6</td>
<td>2.7</td>
<td>2.8</td>
<td>2.9</td>
<td>3.1</td>
<td>3.2</td>
</tr>
<tr>
<td>afshuifsterkte</td>
<td>( f_{0.85} )</td>
<td>1.7</td>
<td>1.8</td>
<td>2.0</td>
<td>2.2</td>
<td>2.4</td>
<td>2.5</td>
<td>2.8</td>
<td>3.0</td>
<td>3.4</td>
<td>3.8</td>
<td>3.8</td>
<td>3.8</td>
</tr>
</tbody>
</table>

Elasticiteitmodulus in de bruikbaarheidsgrensoordestand \( E_{0.05} \)

<table>
<thead>
<tr>
<th></th>
<th></th>
<th>7000</th>
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<th>9000</th>
<th>9500</th>
<th>10000</th>
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<th>12000</th>
<th>13000</th>
<th>14000</th>
<th>15000</th>
<th>16000</th>
</tr>
</thead>
</table>

Table G-10 Representative values of material properties of sawn hardwood (EN 1995-1-1 Eurocode 5, 2005)

<table>
<thead>
<tr>
<th>Grootheid</th>
<th>Symbol</th>
<th>D30</th>
<th>D35</th>
<th>D40</th>
<th>D50</th>
<th>D60</th>
<th>D70</th>
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</thead>
<tbody>
<tr>
<td>buigsterkte</td>
<td>( f_{0.85} )</td>
<td>30</td>
<td>35</td>
<td>40</td>
<td>50</td>
<td>60</td>
<td>70</td>
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<tr>
<td>volumsche massa</td>
<td>( h_{\text{vol}} )</td>
<td>640</td>
<td>670</td>
<td>710</td>
<td>780</td>
<td>840</td>
<td>1080</td>
</tr>
<tr>
<td>volumsche massa</td>
<td>( h_{\text{v}} )</td>
<td>530</td>
<td>560</td>
<td>590</td>
<td>650</td>
<td>700</td>
<td>900</td>
</tr>
<tr>
<td>treksterkte (evenwijdig)</td>
<td>( f_{0.85} )</td>
<td>18</td>
<td>21</td>
<td>24</td>
<td>30</td>
<td>36</td>
<td>42</td>
</tr>
<tr>
<td>treksterkte (loodrecht)</td>
<td>( f_{0.85} )</td>
<td>0.6</td>
<td>0.6</td>
<td>0.6</td>
<td>0.6</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
<td>druksterkte (evenwijdig)</td>
<td>( f_{0.85} )</td>
<td>23</td>
<td>25</td>
<td>26</td>
<td>29</td>
<td>32</td>
<td>34</td>
</tr>
<tr>
<td>druksterkte (loodrecht)</td>
<td>( f_{0.85} )</td>
<td>8.0</td>
<td>8.4</td>
<td>8.4</td>
<td>9.7</td>
<td>10.5</td>
<td>13.5</td>
</tr>
<tr>
<td>afshuifsterkte</td>
<td>( f_{0.85} )</td>
<td>3.0</td>
<td>3.4</td>
<td>3.8</td>
<td>4.6</td>
<td>5.3</td>
<td>6.0</td>
</tr>
</tbody>
</table>

In the unity check for bending moment the following height factor for rectangular sawn solid timber has to be taken into account:

\[
k_h = \min \left\{ \left( \frac{150}{h} \right)^{0.2} \right\}
\]

The modification factor has to be chosen from Table G-11. The partial factor is derived from Table G-12.

Table G-11 Values for \( k_{\text{mod}} \) (De Vries & Van de Kuilen, 2010)

<table>
<thead>
<tr>
<th>Material</th>
<th>Service class</th>
<th>Permanent action</th>
<th>Long term action</th>
<th>Medium term action</th>
<th>Short term action</th>
<th>Instantaneous action</th>
</tr>
</thead>
<tbody>
<tr>
<td>Solid timber</td>
<td>1</td>
<td>0.60</td>
<td>0.70</td>
<td>0.80</td>
<td>0.90</td>
<td>1.10</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.60</td>
<td>0.70</td>
<td>0.80</td>
<td>0.90</td>
<td>1.10</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>0.50</td>
<td>0.55</td>
<td>0.65</td>
<td>0.70</td>
<td>0.90</td>
</tr>
</tbody>
</table>
In the formulas mentioned earlier the following parameters are used:

- \( M_{z,d} \) = governing moment [kNm]
- \( W_z \) = moment of resistance [mm³]
- \( \sigma_{m,d} \) = Design bending stress about the principal z-axis [N/mm²]
- \( f_{m,d} \) = Design bending strength [N/mm²]
- \( f_{v,d} \) = Design shear strength [N/mm²]
- \( \gamma_m \) = partial factor material properties [-]
- \( k_h \) = height factor [-]
- \( f_{m,0,k} \) = Characteristic bending strength along the grain [N/mm²]
- \( f_{v,k} \) = Characteristic shear strength [N/mm²]
- \( k_{mod} \) = Modification factor for duration of load and moisture content [-]
- \( b \) = width of beam [m]
- \( h \) = height of beam [m]

**Plank dimensions**

The force on the most heavily loaded plank has to be calculated to design the wooden plank. The parameters that have been used are:

- \( \gamma_c,z \) = saturated volumetric weight clay ground = 17.5 kN/m³
- \( \gamma_w \) = volumetric weight water = 10 kN/m³
- \( h \) = height plank = 0.3m
- \( u \) = 0.75 m/s

\[
K_p = \frac{1 + \sin \phi'}{1 - \sin \phi'} = \frac{1 + \sin 27.5}{1 - \sin 27.5} = 2.72
\]

\[
K_u = \frac{1}{2.72} = 0.37
\]

**Situation 1:**

The forces are:

\[
F_w = \frac{1}{2} (\gamma_w)(h_1)^2 + \frac{1}{2} (\gamma_w)(h_2)^2 \times h = \frac{1}{2} 10 \times 1.8^2 + \frac{1}{2} 10 \times 1.5^2 \times 0.3 \times 0.3 = 4.12 \text{ kN/m}
\]

With a balance of impulse the force on the weir per unit length can be calculated.

\[
F_w - q_{plank} + \rho q \Delta u = 0
\]

\[
4.12 - q_{plank} + 1000 \times (0.75 \times 0.3) \times 0.75/1000 = 0
\]

\[
q_{plank} = 4.29 \text{ kN/m}
\]
The width of the weir is 10 meters and is divided in four parts with vertical columns. The planks are 2.5 meter long and are nailed on the vertical columns. It is assumed that the connection between the vertical column and the plank is hinged. The maximum moment in the plank is then:

\[ M_{\text{max.plank}} = \frac{1}{2} \gamma_{f,q} q_{\text{plank}} l^2 = \frac{1}{8} (1.5 \times 4.29) \times 2.5^2 = 5.02 \text{kNm} \]

With the unity check for bending moment the thickness of the column can be calculated. The unity check for bending moment is usually decisive.

\[ \frac{\sigma_{m,d}}{f_{m,d}} \leq 1 \]

In which:

\[ \sigma_{m,d} = \frac{M_{x,d}}{W_x} = \frac{M_{x,d}}{\frac{1}{6} b h^2} \]
\[ f_{m,d} = f_{m,0,k} \frac{k_{\text{mod}} k_h}{\gamma_m} \]
\[ k_h = \min \left\{ \frac{150}{h}, \frac{150}{300}, 1.3 \right\} = 0.87 \]

The same wood is used as for the log stilling basing. Strength class C18 gives according to Table G-9 a characteristic bending moment strength of \( f_{m,0,k} = 18 \text{N/mm}^2 \).

Load factors (Vrijling, et al., 2011):
\[ \gamma_{f,q} = 1.5 \]
\[ \gamma_{f,\delta} = 1.2 \]
\[ \gamma_{f,v} = 0.9 \]

It is a short-term and submerged solution (service class 3) this gives according to Table G-11 a modification factor of \( k_{m,0} = 0.7 \)

And for solid timber Table G-12 gives partial factor of: \( \gamma_m = 1.3 \)

Filling these values into the unity check gives:

\[ \frac{M_{x,d}}{\frac{1}{6} b h^2} \frac{k_{\text{mod}} k_h}{\gamma_m} \leq 1 \]
\[ \frac{5.02 \times 10^6}{\frac{1}{6} b \times 0.87} \leq 1 \]

The minimum width of the plank to satisfy this check is: \( b = 39.69 \text{mm} \)

This gives the dimensions of the planks: 300mm high, 40mm wide and 2500mm long.

These dimensions are checked with the unity check for shear force, which is usually not decisive.

\[ \frac{\tau_{v,d}}{f_{v,d}} \leq 1 \]

The maximum shear force in the plank is:
\[ V_{d} = \frac{1}{2} \gamma_{f,q} q l = \frac{1}{2} \times 1.5 \times 4.29 \times 2.5 = 8.04 \text{kN} \]

In which:
For situation 2:

The forces in this situation are:

\[ F_w = \frac{1}{2} (y_w (h_3) + \frac{1}{2} (y_0) (h_4))^2 \times h = \frac{1}{2} (10 \times 18.2 + 10 \times 15.2) \times 0.3 = 4.12 \text{ kN/m} \]

\[ F_{s,n} = \frac{1}{2} k_n (y_{s,n} - y_w) h_3 \times h = \frac{1}{2} \left( \frac{20 - 10}{2} \right) (18.2 + 15.2) \times 0.3 = 4.12 \text{ kN/m} \]

\[ F_{s,p} = \frac{1}{2} k_p (y_{s,p} - y_w) h_4 \times h = \frac{1}{2} \left( \frac{17.5 - 10}{2} \right) (18.2 + 15.2) \times 0.3 = 1.79 \text{ kN/m} \]

\[ M_{max,plank} = \frac{1}{4} (y_{f,p} F_{w} + (y_{f,p} F_{s,n}) + (y_{f,w} + F_{s,p}) = \frac{1}{4} (1.5 \times (4.12) + 1.2 \times (4.12) - 0.9 \times (1.79)) \times 2.5^2 = 7.43 \text{ kNm} \]

With the unity check for bending moment the thickness of the column can be calculated. The unity check for bending moment is usually decisive.

\[ \frac{M_{z,d}}{f_{m,0,k} k_{mod} k_h} \leq 1 \]

\[ \frac{7.43 \times 10^6}{18 \times 0.7 \times 0.87} \leq 1 \]

The minimum width of the plank to satisfy this check is: \( b = 58.76 \text{ mm} \)

In this situation the following dimensions of the planks are chosen: 300mm high, 60mm wide and 2500mm long. These dimensions are larger than the first situation therefore this situation is decisive. Next the dimensions of this situation are checked with the unity check for shear.

\[ \frac{\sigma_{v,d}}{f_{v,d}} \leq 1 \]

The maximum shear force in the plank is:

\[ V_d = \frac{1}{2} q l = \frac{1}{2} \times (1.5 \times (4.12) + 1.2 \times (4.12 - 1.79)) \times 2.5 = 11.22 \text{ kN} \]

In which:

\[ \sigma_{v,d} = \frac{3V_d}{2bh} = \frac{3 \times 11.22 \times 10^3}{2 \times 60 \times 300} = 0.935 \]

\[ f_{v,d} = f_{v,k} k_{mod} k_h = 2.0 \times 0.7 \times 1.3 = 1.08 \]

\[ \frac{\sigma_{v,d}}{f_{v,d}} = \frac{0.935}{1.08} = 0.87 \leq 1 \]

Also this unity check satisfies.
**Columns dimensions**

To design the dimensions of the vertical columns and the diagonal brace the forces in the cross-section are calculated with Matrixframe, see Figure G-30. With the maximum moment and shear force the same unity checks as for the planks is done. Below the input and the results from Matrixframe are shown.

To design the vertical columns the decisive situation that has been described for the girder dimensions is used. The following loads are applied on the structure:

**Loads on the left wall:**
- Left: \( q_{l,t} = \gamma_f q \times \rho gh_7b \) \( = 1.5 \times 1000 \times 9.81 \times 1.8 \times 2.5 = 66.23 \text{ kN/m} \)
- Left: \( q_{l,tz} = \gamma_f q \times K_a (\gamma_{c,s} - \gamma_w)h_b b = 1.5 \times 0.37 \times (17.5 - 10) \times 0.9 \times 2.5 = 9.36 \text{ kN/m} \)
- Right: \( q_{r,t} = \gamma_f q \times (\rho gh_7b + K_n (\gamma_{s,s} - \gamma_w)h_7b) = 1.5 \times (1000 \times 9.81 \times 1.8 \times 2.5 + 1(20 - 10) \times 1.8 \times 2.5) = 129.23 \text{ kN/m} \)

**Loads on the right wall:**
- Left: \( q_{r,t} = \gamma_f q \times (\rho gh_7b + K_n (\gamma_{s,s} - \gamma_w)h_7b) = 1.5 \times (1000 \times 9.81 \times 1.8 \times 2.5 - 1(20 - 10) \times 1.8 \times 2.5) = 129.23 \text{ kN/m} \)
- Right: \( q_{r,t} = \gamma_f q \times (K_p (\gamma_{s,s} - \gamma_w)h_b + \rho gh_6 b) = 1.5 \times (2.72 \times (17.5 - 10) \times 0.9 \times 2.5 + 1000 \times 9.81 \times 0.9 \times 2.5) = 102.6 \text{ kN/m} \)

**Loads on the floor:**
- Left: \( q_{l,f} = 66.23 \text{ kN/m} \)
- Right: \( q_{r,f} = \gamma_f q \times \rho gh_6b \) \( = 1000 \times 9.81 \times 0.9 \times 2.5 = 33.11 \text{ kN/m} \)

![Figure G-30 Dimensions and the loads in Matrixframe](image-url)
The forces on the structure are currently known. However, the combined bending and axial tension/compression unity check is used because there is a large moment and tensile/compression stress are acting on the columns. (EN 1995-1-1 Eurocode 5, 2005)

**Vertical beam**

The right beam of the cross section has the largest moment and a large compressive load. With the combined bending and axial compressive unity check the dimensions of this beam are determined.

\[
\left( \frac{\sigma_{c,d}}{f_{c,d}} \right)^2 + k_m \frac{\sigma_m}{f_{m,d}} \leq 1
\]

[10]

In which:

- For rectangular sections: \(k_m = 0.7\)
- \(\sigma_{m,d} = \frac{M_{x,d}}{W_z} = \frac{M_{x,d}}{(1/6b \cdot h^2)} = \frac{19.27 \times 10^6}{1/6 \cdot b \cdot h^2}\)
- \(f_{m,d} = f_{m,0,k} \frac{k_{mod}}{\gamma_m} \frac{k_h}{0.7} = \frac{18}{1.3} \cdot 0.87 = 8.43 \text{N/mm}^2\)
- \(\sigma_{c,0,d} = \frac{N_d}{A} = \frac{13.37 \times 1000}{b \times h}\)
- \(f_{c,0,d} = f_{c,0,k} \frac{k_{mod}}{\gamma_m} \frac{k_h}{0.7} = \frac{18}{1.3} \cdot 0.87 = 8.43 \text{N/mm}^2\)

Solving this system for 2b=h:
On both sides of the girder is a beam so the dimension of a single beam is: 140x140mm$^2$. This cross section is now checked for shear force.

\[
\sigma_{v,d} = \frac{3V_d}{2(2bh)} = \frac{3\times31.07\times1000}{2\times(2\times140\times140)} = 1.19
\]

\[
f_{v,d} = f_{vk} k_{\text{mod}} \frac{f_m}{f_m} = 2.0 \times \frac{0.7}{1.3} = 1.077
\]

\[
\frac{\sigma_{v,d}}{f_{v,d}} = \frac{1.19}{1.08} = 1.10 \leq 1
\]

So this cross-section does not withstand the shear force. The cross section of the beam is increased to 160x160mm$^2$. Check the cross section for shear force again:

\[
\sigma_{v,d} = \frac{3V_d}{2(2bh)} = \frac{3\times31.07\times1000}{2\times(2\times160\times160)} = 0.910
\]

\[
f_{v,d} = f_{vk} k_{\text{mod}} \frac{f_m}{f_m} = 2.0 \times \frac{0.7}{1.3} = 1.077
\]

\[
\frac{\sigma_{v,d}}{f_{v,d}} = \frac{0.910}{1.077} = 0.845 \leq 1
\]

This cross section is large enough to withstand the shear force.

**Horizontal beam**

The bottom beam has large tensile force a little moment. To check this beam the combined bending and axial tensile unity check is used. For this beam the same dimensions as for the vertical beam are taken. The cross section now only consists of one beam.

\[
\frac{\sigma_{t,0,d} + k_m \sigma_{m,d}}{f_{t,0,d}} \leq 1 \tag{11}
\]

In which:

For rectangular sections: $k_m = 0.7$

\[
\sigma_{m,d} = \frac{M_{zd}}{w_z} = \frac{M_{zd}}{(1/f_d) (h)^2} = \frac{3.25\times10^6}{1/6 \times 160 \times 160} = 4.76 \text{ N/mm}^2
\]

\[
f_{m,d} = f_{m,0,k} k_{\text{mod}} k_h = 18 \times \frac{0.7}{1.3} \times 0.87 = 8.43 \text{ N/mm}^2
\]

\[
\sigma_{t,0,d} = \frac{N_d}{A} = \frac{7.08\times1000}{150\times150} = 3708\times1000 = 1.648 \text{ N/mm}^2
\]

\[
f_{t,0,d} = f_{t,0,k} k_{\text{mod}} k_h = 11 \times \frac{0.7}{1.3} \times 0.87 = 5.15 \text{ N/mm}^2
\]

\[
\frac{1.648}{5.15} + 0.7 \times 4.76 = 0.72 \leq 1
\]

This cross section is large enough to withstand the bending moment and tensile force. The shear force in this beam is very low so this doesn’t have to be checked. The dimensions found for the bottom beam are also used for the upper beam.
**Diagonal bracing**

The only force in the bracing is a tensile force. The dimensions of the bracing are therefore determined by unity check for tensile force.

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} \leq 1$$  \[12\]

$$\sigma_{t,0,d} = \frac{N_d}{A} = \frac{37.08 \times 1000}{b \times h} = \frac{19.99 \times 1000}{b \times h}$$

$$f_{t,0,d} = f_{t,0,k} \frac{k_{mod}}{r_m} k_h = 11 \times \frac{0.7}{1.3} \times 0.87 = 5.15 \text{ N/mm}^2$$

$$\frac{19.99 \times 1000}{b \times h} \leq 5.15$$

After some iteration dimensions of 100mmx40mm are found for the diagonal bracing.

Now all the dimensions for the wooden structure are determined. In Figure G-45 a cross sections of the wooden weir is shown with the dimensions determined in this appendix.

### G.6 Stability of the weir and subsoil

The weir structure has to be stable, due to the loads the structure must not move. The following formulas have to satisfy:

$$\Sigma H = 0$$  \[13\]

$$\Sigma V = 0$$  \[14\]

$$\Sigma M = 0$$  \[15\]

This equilibrium of forces is examined in sections G.6.1 to G.6.1. Piping is checked in G.6.4. The stability of the sand bags is checked in G.6.5 and settlements are calculated in G.7.

#### G.6.1 Horizontal stability

First, the horizontal force equilibrium is checked. The total of horizontal forces acting on the structure has to be zero. So the friction force of the subsoil has to be larger than the forces acting on the structure. When the friction force of the subsoil is not large enough, this must be made larger or the structure needs a better foundation.

In Figure G-33 the loads on the structure are displayed. Assumed is that no active and passive sand wedges can be activated through the geotextile of the sandbags. Though, the sandbags above the log base are taken into account in the shape of the structure. On the left side of Figure G-33 the maximum water level is +1.8 meter GMSL and on the right side the minimum water level is +0.25 meter GMSL. The critical water level on the weir is 0.42 meter. In the calculation of the horizontal stability safety factors are taken into account. This is done to take in account the uncertainties in the stability of the structure. Loads with a positive effect on stability are lowered with a factor 0.9. Loads with a negative effect on the stability are heightened with a factor 1.1.
The result shows that the structure is stable in the horizontal direction.

G.6.2 Rotational stability
The ground cannot transfer tension, so soil forces must be only compressive for rotational
stability. This is the case when the resulting force intersects the core of the structure, see
Figure G-34. So the following formula has to satisfy for rotational stability (Vrijling, et al.,
2011):

\[
\gamma_{f,q} F_{w,1} + \gamma_{f,q} F_{w,2} - \gamma_{f,v} F_{w,3} + \gamma_{f,q} \rho q \Delta u \leq \gamma_{f,v} F_f
\]

\[
1.1 \times 16.2 + 1.1 \times 16.2 - 0.9 \times 6.6 + 1.1 \times 1000 \times 1.5 \times 0.75/1000 \leq 0.9 \times 79.39
\]

\[
30.94 \text{ kN/m} < 71.46 \text{ kN/m}
\]

The result shows that the structure is stable in the horizontal direction.

G.6.2 Rotational stability
The ground cannot transfer tension, so soil forces must be only compressive for rotational
stability. This is the case when the resulting force intersects the core of the structure, see
Figure G-34. So the following formula has to satisfy for rotational stability (Vrijling, et al.,
2011):

\[
\sum M \leq \frac{1}{6} \times (b_1 + b_2 + b_3)
\]
This means that the structure will rotate. One extra layer of sandbags is added at the Mfolozi side of the weir. This results in:

\[ \sum M = 1.1 \times F_{w,1} \times \frac{1}{2} (h_2 + h_3) + 1.1 \times F_{w,2} \times \frac{1}{3} (h_2 + h_3) - 0.9 \times F_{w,3} \times \frac{1}{3} (h_4 + h_5) - 0.9 \times F_{w,4} \times \frac{1}{2} (b_1 + b_2) + 1.1 \times F_{w,6} \times \frac{1}{2} (b_2 + b_3) + 1.1 \times F_{w,7} \times \frac{1}{6} (b_1 + b_2 + b_3) + 1.1 \times \rho q \Delta u \times \frac{1}{2} (h_2 + h_3) = 1.1 \times 16.2 \times 0.9 + 1.1 \times 16.2 \times 0.6 - 0.9 \times 6.6 \times \frac{1}{3} \times 1.15 - 0.9 \times 18 \times 1.5 + 1.1 \times 2.5 \times 1.5 + 1.1 \times 31 \times 0.66 = 26.78 \text{ kNm/m} \]

\[ \sum V = 0.9 \times G + 0.9 \times F_{w,4} + 0.9 \times F_{w,5} + 0.9 \times F_{w,6} - 1.1 \times F_{w,7} - 1.1 \times F_{w,8} = 0.9 \times (2 \times 1.8 + 2 \times 0.9 \times 1) \times 20 + 0.9 \times (18 + 8.4 + 2.5) - 1.1 \times (46 + 31) = 38.51 \text{ kN/m} \]

\[
\frac{26.78}{38.51} \leq \frac{1}{6} \times 4
\]

\[ 0.70 \leq 0.66 \]

This is sufficient. In Appendix J an AutoCAD drawing is given in which the total structure is displayed.
G.6.3 Vertical stability

To satisfy vertical stability the effective soil stress \( (\sigma_{k,max}) \) caused by the acting loads on the structure may not exceed the maximum bearing capacity of the soil \( (p_{\text{max}}') \). The following formula has to satisfy:

\[
\sigma_{k,max} < p_{\text{max}}'
\]

In which the maximum acting load on the soil can be calculated with (Vrijling, et al., 2011):

\[
\sigma_{k,max} = \frac{\sum V}{A_{f}} + \frac{\sum M}{W_{y} \times i_{b} \times h_{b}^{2}}
\]

For determining the maximum bearing of a foundation plate the Brinch Hansen method can be used (Vrijling, et al., 2011). The area over which the bearing capacity is calculated is limited to the area of the wooden box of the weir. The formula of Brinch Hansen is:

\[
p_{\text{max}}' = c'N_{c} s_{c} i_{c} + q'N_{q} s_{q} i_{q} + \frac{1}{2} \gamma' B N_{y} s_{y} i_{y}
\]

In which the bearing capacity factors are:

\[
N_{c} = (N_{q} - 1) \cot \phi'
\]

\[
N_{q} = \frac{1 + \sin \phi'}{1 - \sin \phi'} e^{\pi \tan \phi'}
\]

\[
N_{y} = 2(N_{q} - 1) \tan \phi'
\]

The shape factors are:

\[
s_{c} = 1 + 0.2 \frac{B}{L}
\]

\[
s_{q} = 1 + \frac{B}{L} \sin \phi'
\]

\[
s_{y} = 1 - 0.3 \frac{B}{L}
\]

And the inclination factors:

For \( H \) parallel to \( L \) and \( L/B > 2 \):

\[
i_{c} = \frac{i_{q}N_{q}^{-1}}{N_{q}^{-1}}
\]

\[
i_{q} = i_{y} = 1 - \frac{H}{F + A c' \cot \phi'}
\]

For \( H \) parallel to \( B \):

\[
i_{c} = \frac{i_{q}N_{q}^{-1}}{N_{q}^{-1}}
\]

\[
i_{q} = \left(1 - \frac{0.70H}{F + A c' \cot \phi'}\right)^{3}
\]

\[
i_{y} = \left(1 - \frac{H}{F + A c' \cot \phi'}\right)^{3}
\]

In the above formulas the following parameters are used:

\[
\sum V = \text{total acting vertical force [N]}
\]

\[
A = \text{area of the foundation [m}^2\text{]}
\]

\[
W = \text{section modulus of the contact area of the foundation [m}^3\text{]}
\]

\[
b = \text{width of the foundation [m]}
\]

\[
l = \text{length of the foundation [m]}
\]
$\Sigma M$ = total of the acting moments [kN/m]
$c'$ = (weighted) cohesion [kPa]
$q'$ = (weighted) surcharge next to the weir [kPa]
$\gamma'$ = (weighted) effective volumetric weight of the soil below construction depth. [kN/m$^3$]
$x_c$ = factor for the influence of cohesion [-]
$x_q$ = factor for the influence of the soil cover [-]
$x_{\gamma'}$ = factor for the influence of the effective volumetric weight of the soil [-]
$\phi'$ = (weighted) effective angle of internal friction [°]
$H$ = the shear force [kN]
$F$ = component of the exerted force perpendicular to the surface [kN]

A low but safe value for the (weighted) cohesion $c'$ of the subsoil is 2.0 kPa which follows from table 1 of the Dutch norm NEN6740 (heavily sanded clay). The effective angle of internal friction of the subsoil is set at 27.5°. This value is low but safe. The effective volumetric weight of the subsoil is 17.5 kN/m$^3$ (G.7). The length and width of the weir are set at 15 m and 2 m respectively. The surcharge next to the weir is caused by 0.90 m of sandbags with a saturated volumetric weight of 20 kN/m$^3$: $0.90 \cdot 20 = 18$ kPa.

\begin{align*}
N_q &= \frac{1+\sin \phi'}{1-\sin \phi'} e^{\pi \tan \phi'} = \frac{1+\sin 27.5}{1-\sin 27.5} e^{\pi \tan 27.5} = 13.9 \\
N_c &= (N_q - 1) \cot \phi' = (13.93 - 1) \cot 27.5 = 24.9 \\
N_{\gamma'} &= 2(N_q - 1) \tan \phi' = 2(13.93 - 1) \tan 27.5 = 13.5 \\
s_c &= 1 + 0.2 \frac{B}{L} = 1 + 0.2 \frac{2}{15} = 1.027 \\
s_q &= 1 + \frac{B}{L} \sin \phi' = 1 + \frac{2}{15} \sin 27.5 = 1.062 \\
s_{\gamma'} &= 1 - 0.3 \frac{B}{L} = 1 - 0.3 \frac{2}{15} = 0.960
\end{align*}

The inclination factors are not used because the load is not inclined. The maximum bearing capacity of the subsoil is:

\[ p'_{\text{max}} = c'N_c s_c + q'N_q s_q + \frac{1}{2} \gamma' BN_{\gamma'} s_{\gamma'} \]

\[ = 2.0 \cdot 24.85 \cdot 1.027 + 18 \cdot 13.93 \cdot 1.062 + \frac{1}{2} \cdot 17.5 \cdot 2 \cdot 13.47 = 553 \text{ kPa} \]

The load on the subsoil is:

\[ \sigma_{K,\text{max}} = \frac{\Sigma V}{b \times l} + \frac{\Sigma M}{I_6 \times (b^2)} = \frac{40.76}{4 \times 1} + \frac{23.41}{\frac{1}{6} \times 1 \times 4^2} = 18.97 \text{ kPa} \]

The result shows that the bearing capacity is sufficient.

G.6.4 Piping

When there is a potential difference across the structure it is possible that piping occurs. Ground water flow can occur in loose grain layers. Piping is a flow through a pipe-like channel made by internal erosion.

Piping is depended on what kind of soil is below the structure and difference head across the structure. There are some empirical formulas to calculate the safe seepage length. The formulas from Bligh and Lane are the common. In Table G-13 these formulas are given, including their coefficients. Bligh’s method is mostly used for the design of dikes and Lane’s
method is mostly used for water retaining structures, because it takes also vertical piping lines into account. (Vrijling, et al., 2011)

Table G-13 Seepage distance (Vrijling, et al., 2011)

<table>
<thead>
<tr>
<th>Piping method:</th>
<th>Bligh</th>
<th>Lane</th>
</tr>
</thead>
<tbody>
<tr>
<td>Safe seepage distance</td>
<td>$L \geq \gamma \times C_b \times \Delta H$</td>
<td>$L \geq \gamma \times C_L \times \Delta H$</td>
</tr>
<tr>
<td>True seepage distance</td>
<td>$L = \sum L_{\text{vert}} + \sum L_{\text{hor}}$</td>
<td>$L = \sum L_{\text{vert}} + \frac{1}{3} \sum L_{\text{hor}}$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Soil type</th>
<th>$C_b$</th>
<th>$i_{\text{max}}$</th>
<th>$C_L$</th>
<th>$i_{\text{max}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very fine sand/ silt/ sludge</td>
<td>18</td>
<td>5.6%</td>
<td>8.5</td>
<td>11.8%</td>
</tr>
<tr>
<td>Fine sand</td>
<td>15</td>
<td>6.7%</td>
<td>7.0</td>
<td>14.3%</td>
</tr>
<tr>
<td>Middle fine sand</td>
<td>-</td>
<td>-</td>
<td>6.0</td>
<td>14.7%</td>
</tr>
<tr>
<td>Coarse sand</td>
<td>12</td>
<td>8.3%</td>
<td>5.0</td>
<td>20.0%</td>
</tr>
<tr>
<td>(Fine) Gravel (+sand)</td>
<td>5-9</td>
<td>11.1-20.0%</td>
<td>4.0</td>
<td>25.0%</td>
</tr>
</tbody>
</table>

In which:

$C_b$ = Bligh’s constant, depends on soil type [-]
$C_L$ = Lane’s constant, depends on soil type [-]
$\Delta H$ = differential head across structure [m]
$\gamma$ = safety factor (=1.5) [-]
$i_{\text{max}}$ = maximum (allowed) hydraulic gradient (=\(\Delta H/L\)) [-]

Figure G-35 Vertical and horizontal seepage paths (Vrijling, et al., 2011)

When the structure’s true seepage distance is not long enough some measures have to be taken. Possible measures are (Vrijling, et al., 2011):

1. using (longer) sheet piling upstream as a screen against seepage
2. grout columns (making the soil impermeable and cohesive) (upstream)
3. inserting a diagonal protective textile in the ground (in front of the structure)
4. inserting a filter structure (downstream)

The subsoil around the weir consists of heavily sanded clay and is not mainly granular and is also not composed of cracks. The methods of Bligh and Lane are therefore not applicable in this situation. The permeability of heavily sanded clay is much lower than the permeability of very fine sand or silt. This does not mean that piping could not occur. Though the resistance is much larger, some water will flow through the subsoil from the Mfolozi side of the new Back Channel towards the St Lucia side. Piping is not seen as a failure mode of the weir structure. Still several safety measures have been taken to prevent piping underneath the structure.
The weir will be supported by sandbags which lie on geotextile. Water will be able to seep through and between the sandbags which behave as a filter. The geotextile underneath the sandbags prevents soil to seep out from underneath the geotextile. Next to vertical seepage underneath the structure horizontal seepage does also occur. Horizontal seepage is water flowing through the banks alongside the weir. The weir will be extended at both sides into the banks to anchor the weir in the banks and to prevent horizontal seepage. The extension will be 2.5 meter at both sides because the compartments of the weir are 2.5 m wide. 2.5 Meter is deemed to be enough for lengthening the horizontal seepage path. Sandbags on top of geotextile around the weir heads will act as a filter for horizontal water flow. The subsoil around the weir has to be compacted during construction to prevent seepage of water through uncompacted dumped soil.

The weir approach at the Mfolozi side contracts over 10 meters length from a width of 15 meter to a width of 10 meter. At a water level of +1.8 m GMSL at the Mfolozi side the water accelerates from 0.7 m/s to 4 m/s at which it flows over the weir. Scouring of the bed occurs at flow speeds of above 0.7 m/s (B.5). To elongate the seepage length and to protect the bottom and sides of scouring the channel will be lined with sandbags over a length of 5 meters. The sandbag lined approach of the weir is 10 meter in length. The transition of the natural channel towards the sandbag lined channel has to be monitored at set times. Little scouring and damage at this location is accepted. Maintenance should be carried out when this scouring endangers the weir.

G.6.5 Bottom protection
The dimension of the sandbags used for the bottom protection can be iterated with the critical depth averaged flow velocity according to the incipient sandbag motion with Figure G-12 (Zhu, Wang, Cheng, Ying, & Zhang, 2004).

First this is done for a water level of 1.8 meter in the Mfolozi mouth. Probably this is the most decisive situation. This is checked to the same calculation for a water level of 1.6 meter in the Mfolozi mouth.

Water level of 1.8 meter
On the left in Figure G-36 the depth averaged flow velocity of the river is plotted from 10 meters in front of the weir until the weir and on the right the corresponding water level is plotted. A depth averaged flow velocity of 1.1 m/s and a water level of 1.6 m are taken to iterate the dimensions of the sandbags. The gravitational acceleration is 9.81 m/s² and the relative density is \( \Delta = \frac{\rho_{s,\text{w}} - \rho_{\text{w}}}{\rho_{\text{w}}} = 1.0 \). Calculations have been done for sandbags of 0.25 m high. By filling these values in the formulas below, the dimensions can be iterated.
Figure G-36 flow velocity and waterdepth in front of the weir at a water level of 1.8 meter in the Mfolozi mouth (data from SOBEK)

\[
\frac{V_c}{\sqrt{gD}} = \frac{1.1}{\sqrt{10 \times 9.81 \times 0.25}} \left(\frac{0.25}{1.6}\right)^{\frac{1}{6}} = 0.52
\]

Figure G-37 Critical velocity for incipient sandbag motion (Zhu, Wang, Cheng, Ying, & Zhang, 2004)

\[
\sqrt{\frac{L}{D}} = 1.1 \Rightarrow L = 0.30 \text{m}
\]

When a height of 0.25 m is taken for the sandbag the length has to be at least 0.30 m. For reasons of safety and simplicity sandbag dimension of 1 m length by 0.25 m high by 0.5 m wide are chosen at the Mfolozi side of the weir. Sandbags of 2 m by 1 m by 0.5 m will be applied at the sides of the downstream end of the channel because the flow can be supercritical at that side. Underneath the log stilling basin smaller bags can be used because these bags are protected by the log structure.

**Water level of 1.6 meter**

For a water level of 1.6 m in the Mfolozi mouth the depth averaged flow velocity and the corresponding water level are given in Figure G-38. A depth averaged flow velocity of 0.76 m/s and a water level of 1.4 m are taken to iterate the dimensions of the sandbags.

Figure G-38 Flow velocity and waterdepth in front of the weir at a water level of 1.8 meter in the Mfolozi mouth (data from SOBEK)

\[
\frac{V_c}{\sqrt{gD}} = \frac{0.76}{\sqrt{10 \times 9.81 \times 0.25}} \left(\frac{0.25}{1.4}\right)^{\frac{1}{6}} = 0.36
\]
\[
\sqrt{\frac{L}{D}} = 0.75 \rightarrow L = 0.14 \text{m}
\]

For a height of 0.25 m the sandbag the length has to be at least 0.14 m. This is much less than for the situation with a water level of 1.8 m in the Mfolozi mouth. Therefore this situation is not decisive.

### G.7 Settlement

Loading of the subsoil by a weir causes the subsoil to settle. This appendix treats the acquisition of data required for the settlement calculation. In paragraph G.7.1 the calculation method is discussed and in paragraph G.7.2 the results are discussed.

#### G.7.1 Settling of the subsoil

Loading of the subsoil will cause settlements. It is important to know by how much and how fast the soil will settle. Settlement can harm the structural integrity of the superstructure. Calculations have been performed with D-settlement 9.2. D-Settlement is a program developed by Deltares which calculates the settling of the soil on basis of methods described in the Dutch norms and Eurocode 7. The results describe a range of settlement values due to the assumptions made on the soil characteristics. The following subsections describe the set-up of the model, the assumptions made and the results obtained.

**Model set-up**

D-settlement can compute settlements with three different methods. One model is the NEN-Bjerrum method which is known internationally. The NEN-Bjerrum method makes use of the compression index \( C_c \), the swelling index \( C_s \) and the coefficient of secondary compression \( C_{\alpha} \). Linear strain is assumed by the model. A starting point of the model is that creep accelerates when the load is increased and slows down over time or when a load is taken away (Deltares, 2009). The concept of the model is depicted in Figure G-40. A complete mathematical description of the method is given in Den Haan (Den Haan, 1994).
The consolidation model of Darcy implemented in D-settlement is used to model consolidation of the soil. Darcy’s model is applied to find the development of the pore pressure in the settling soil. The Darcy model is only applicable in fully saturated soil.

The load of the weir construction is modelled as a sand body with a dry specific density of 18kN/m$^3$ and a wet specific density of 20kN/m$^3$. The width of the weir is 15 m. When a culvert is applied, the load will be lower at that location. This lower load is modelled as a lower section of the weir. The dimensions of the culvert are assumed as 2 m wide, 0.50m high, with walls of 0.05 m and two openings. The weight of the culvert is equal to $(2 \cdot 0.5 - (2 - 3 \cdot 0.05) \cdot 0.40) \cdot 24 = 6.24$ kN/m ($\gamma_{\text{concrete}} = 24$ kN/m$^3$). The culvert is placed on top of a sandbag with a height of 0.30 m. One sandbag is placed on top of the culvert. When the total construction is submerged its weight is $6.24 + 2 \cdot 0.30 \cdot 20 = 18.24$ kN/m. This can be represented as a sand body of $18.24 / 20 = 0.92$ m high. The height of the weir is set at +0.90 m GMSL. The bottom under the culvert has to be excavated to a depth of $0.90 - 2 \cdot 0.30 - 0.50 = -0.20$ m GMSL to fit. The bottom of the construction is lined with sandbags. The channel bottom under the construction is lowered with 0.30 m. The model set-up is depicted in Figure G-41. The settlements are calculated along nine verticals spread over the cross section of the weir.

Assumptions

Little is known of the soil characteristics near the new Back Channel. The Dutch norm NEN 6740 gives save ranges of representative soil characteristics of several different soil types. This norm has been incorporated in the Eurocode 7 as national appendix NEN 9997-1. The representative soil characteristics are listed in table 1 of the NEN 6740. The values of the characteristics are derived such that the probability of a lower value is smaller than 5%. From
previous sections (appendix B.4.4) it follows the soil is composed of clay with a large amount of sand. This is listed in table 1 of the NEN 6740 as heavily sanded clay. The ranges of characteristics needed for the calculation are listed in Table G-14. The characteristics of weak, weakly sanded clay are added for comparison. High values of $C_c$, $C_\alpha$ and $C_r$ indicate a weak soil. The soil characteristics found in B.4.4 match with the characteristics of heavily sanded clay.

Table G-14 Representative soil characteristics derived from table 1 of NEN 6740

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Specific density, dry, $\gamma$ [kN/m$^3$]</th>
<th>Specific density, wet, $\gamma_{sat}$ [kN/m$^3$]</th>
<th>Compression index, $C_c$ [-]</th>
<th>Coefficient of secondary compression, $C_\alpha$ [-]</th>
<th>Swelling index, $C_r$ [-]</th>
<th>Undrained shear strength, $f_{undr}$ [kPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay, weakly sanded, weak</td>
<td>15</td>
<td>15</td>
<td>0.759</td>
<td>0.009</td>
<td>0.253</td>
<td>40</td>
</tr>
<tr>
<td>Clay, heavily sanded</td>
<td>18 or 20</td>
<td>18 or 20</td>
<td>0.190 or 0.027</td>
<td>0.004</td>
<td>0.063 or 0.025</td>
<td>0 or 10</td>
</tr>
</tbody>
</table>

It is assumed only one type of soil is present near the new channel. This assumption can be supported by the fact the project site knows a long history of ever changing river courses. The ancient St Lucia estuary was filled over time by a combination of marine and fluvial sediments (Bate, Whitfield, & Forbes, 2011).

The dry and wet specific densities have been set at 17.5kN/m$^3$. The initial void ratio of the soil, $e_0$ has been set at 0.7 (Das, 2010). It is assumed the soil can creep. The vertical permeability is assumed to be constant over time and lies within the range of $10^{-9}$ - $10^{-6}$ m/s. The soil has not been loaded in the past: the over-consolidation ratio is 0. The specific density of water is set at 9.81kN/m$^3$. 
Results
Three sets of calculations have been performed. Per set of calculation the value of the vertical permeability has been differed between $10^9$ and $10^6$ m/s (Bear, 1972). The performed sets of calculations are:

- Clay, heavily sanded, low values of $C_c$, $C_0$ and $C_r$
- Clay, heavily sanded, high values of $C_c$, $C_0$ and $C_r$
- Clay, weakly sanded, weak.

The results of the calculations are given in Table G-15. More results of the data from the first row in Table G-15 can be found in appendix I.5.

Table G-15 Results settlement calculation

<table>
<thead>
<tr>
<th>Calculation set</th>
<th>Vertical permeability, $k_v$ [m/s]</th>
<th>Compression index, $C_c$ [-]</th>
<th>Coefficient of secondary compression, $C_0$ [-]</th>
<th>Swelling index, $C_r$ [-]</th>
<th>Maximum settlement after 10000 days [m] with vertical</th>
<th>Maximum settlement after 365 days [m] with vertical</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay, heavily sanded, low values of $C_c$, $C_0$ and $C_r$</td>
<td>$10^9$</td>
<td>0.027</td>
<td>0.004</td>
<td>0.025</td>
<td>0.270 (1 and 9)</td>
<td>0.136 (5)</td>
</tr>
<tr>
<td></td>
<td>$10^6$</td>
<td>0.027</td>
<td>0.004</td>
<td>0.025</td>
<td>0.271 (1 and 9)</td>
<td>0.174 (1 and 9)</td>
</tr>
<tr>
<td>Clay, heavily sanded, high values of $C_c$, $C_0$ and $C_r$</td>
<td>$10^9$</td>
<td>0.190</td>
<td>0.004</td>
<td>0.063</td>
<td>0.297 (5)</td>
<td>0.091 (5)</td>
</tr>
<tr>
<td></td>
<td>$10^6$</td>
<td>0.190</td>
<td>0.004</td>
<td>0.063</td>
<td>0.306 (3.5 and 7)</td>
<td>0.222 (5)</td>
</tr>
<tr>
<td>Clay, weakly sanded, weak</td>
<td>$10^9$</td>
<td>0.759</td>
<td>0.009</td>
<td>0.253</td>
<td>0.706 (5)</td>
<td>0.134 (5)</td>
</tr>
<tr>
<td></td>
<td>$10^6$</td>
<td>0.759</td>
<td>0.009</td>
<td>0.253</td>
<td>0.818 (5)</td>
<td>0.628 (5)</td>
</tr>
</tbody>
</table>

G.7.2 Discussion
The results of Table G-15 show quite some scatter. When the vertical permeability is large the settlements occur faster than when the vertical permeability is low. The reality will lie somewhere in between. Though there is quite some difference between the high and low values of $C_c$, $C_0$ and $C_r$ of heavily sanded clay, the maximum settlements after 10000 days or after 1 year do not differ that much. Problems occurring due to settlements can easily be solved by heightening of the weir with an extra sandbag. Maintenance of the complete weir might be needed when the settlements lead to inefficacy of the weir. In addition the embankments of the channel have to be heightened over time.

The settlements occurring for weak, weakly sanded clay are larger than those of heavily sanded clay. Results obtained to not indicate this kind of soil. It is not likely for this soil type to occur due to the history of the area. Regular fluvial flooding, tidal working of the sea (during open mouth conditions of the St Lucia estuary) and dumping of dredge spoil have led to a mixed soil type of clay, sand and silt.

D-Settlement is meant to calculate settlements for sand bodies stretched over some distance like road embankments. The weir is only 2m wide in the direction of the flow. The soil directly next to the weir is likely to be pushed up by soil underneath the weir. To counteract this type of behaviour some additional load next to the weir will be needed. A solution would be to add extra sandbags at the bottom of the deepened channel as in the final design of Appendix J.
When geotextile underneath the weir is applied the loads will be spread more evenly and uneven settling is less likely to occur.
The calculations ignored the removal of in situ soil for the excavation of the channel and the replacement by a smaller load. The weight of the removed soil is larger than the replaced load of the sandbags. When this smaller load is taken into account the settlements calculated in Table G-15 are likely to be smaller.

When excavated soil from the channel is dumped directly next to the channel lateral settlements could occur. It is advised to place the excavated soil sufficiently far away from the channel sides to prevent lateral settlements of occurring.

G.8 Energy dissipator
An energy dissipater is required whenever excess energy poses a threat to the stability of the unlined channel. Excess energy may pose a threat when the gradient of the terrain is steeper than the maximum permissible slope and a fall structure is required. Fall structures consist of two parts, one being the control point or crest and the other being the stilling basin. Water can be guided into the stilling basin with a vertical or inclined drop depending on the available materials and preference of the designer. A sketch of both options can be found in Figure G-42.

![energy level](energy_level.png)

VERITCAL DROP STRUCTURE

INCLINED DROP STRUCTURE

*Figure G-42 energy dissipaters. (CT3410-09, 2007)*

G.8.1 Energy dissipation in the New Back Channel.
The design of the new Back Channel weir includes a cofferdam structure with vertical walls. Water drops down vertically unless a chute or slideway is constructed right after the control notch. The basic purpose of a stilling basin is offering a fixed and rigid location where a supercritical flow is transformed into a subcritical flow. Forcing the hydraulic jump to stay inside the structure prevents scour erosion near the downstream channel end. A hydraulic jump is formed when the tail water depth is equal to the equilibrium depth. A basin is required when the tail water depth is less than the equilibrium depth. Figure G-43 elaborates the basic principle of a stilling basin.

![Figure G-43 Basic principle of a stilling basin.](basic_principle.png)
The required length of the basin is based on the empirical design formula for vertical drop basins [20] (Ankum, 2002).

\[ L_b = 11.6 \left( \frac{z}{H_a} \right)^{0.21} \] [20]

In which:
- \( L_b \) = The minimum length of the basin
- \( z \) = The drop height
- \( H_a \) = The energy height before the weir

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Highest water level Mfolozi side</td>
<td>+1.8m GMSL</td>
</tr>
<tr>
<td>Water level St Lucia estuary</td>
<td>+0.25m GMSL</td>
</tr>
<tr>
<td>Energy loss in Back Channel</td>
<td>0.1m</td>
</tr>
<tr>
<td>Drop height</td>
<td>1.45m</td>
</tr>
<tr>
<td>Flow per running meter</td>
<td>2.0m²/s</td>
</tr>
</tbody>
</table>

From [20] and Table G-16 follows that the length must be equal to or longer than 12.5 meter.

During certain conditions waves may be able to damage the construction. Wave formation is expressed by the Froude number \( F_{ri} \) that depends on the energy depth \( H_j \) of the incoming jet. It is defined as a dimensionless number that tells us something about the characteristic velocity with respect to wave velocity. Flow is supercritical for Froude numbers larger than one. In general, incoming jets with Froude numbers below 1.7 are considered safe. Jets with Froude numbers between 1.7 and 2.5 require a stilling basin and may develop small roller waves. Froude numbers between 2.5 and 4.5 should be avoided due to the highly unstable and not well established hydraulic jump (Ankum, 2002).

Water depth Jet:

\[ y_j = \frac{q}{\sqrt{2g(H_j-y_j)}} \] [21]

Froude number incoming jet:

\[ F_{ri} = \frac{q}{(g\ y_j^3)^{0.5}} \] [22]

In which:
- \( y_j \) = water depth of the incoming jet [m]
- \( q \) = flow per running meter [m²/s]
- \( g \) = gravitational acceleration [m/s²]
- \( H_j \) = upstream energy height [m]

Table G-16 lists the water levels during the governing condition. Calculations with formula [21] and [22] show that the Froude number of the jet never exceeds 2.2. This means the supercritical flow stays within the safe range with respect to wave formation.

Due to the high turbulence near hydraulic jumps most stilling basins consist of concrete elements. Near the Back Channel use of hard materials is not allowed and use of rock is not preferred. Furthermore, sandbags are unstable in the supercritical flow passing through the control notch (G.3.1).
As an alternative to a concrete stilling basin a soft structure build from logs could be used. When logs fall into a river they cause a lot of drag. This drag could be used to break the flow over the weir and turn supercritical flow into subcritical flow. Logs of eucalyptus trees are a semi-natural phenomenon in the region. Especially the *Eucalyptus grandis* is used abundantly in plantations. This eucalyptus specie grows as a straight and tall tree and could reach heights of around 50 m (Wikipedia, 2011). First some remarks will be made on pros and cons of log structures. Secondly some calculations will be done on the log structure and thirdly a design will be presented of the log stilling basin.

G.8.2 Pros and cons of log structures

Using logs as a construction material instead of concrete is preferable. There are also downsides to the use of logs.

Pros of log structures are:
- Length of logs over 15 meters;
- *Eucalyptus grandis* is abundantly available in the region;
- Soft construction material;
- Strong material;
- Structure can be anchored in the banks.

The length of a eucalyptus tree at a plantation is on average more than 15m. A log can easily cross the channel bed given the dimensions of the channel. The costs of the trees are not that high because the trees are grown at plantations in the region. When logs are flushed away they will degrade over time and will not cause permanent damage to the nature reserve. Eucalyptus trees are quite strong and are able to cope with the forces of water. Research on Argentinian *Eucalyptus grandis* shows that the bending strength of the material lies in between C18 and C30 depending on the quality of the logs (Piter, Zerbino, & Blass, 2003).

Cons of log structures are:
- Dry densities are lower than 1000 kg/m$^3$;
- Logs can be attacked by borers and termites;
- Timber will decay over time due to microbial attack;
- Material will become less dense over time;
- May influence scour underneath the structure;

When logs are located above the water they could dry out. The wet density of logs is over 1000 kg/m$^3$. When the logs are dry their density will drop below 1000 kg/m$^3$. According to (Brooks, 2006) a safe assumption of the dry density of eucalyptus species is 900 kg/m$^3$. When the logs become wet again it takes time before they reach their wet density.

G.8.3 Stability

Three possible failure modes are applicable on the log structure when placed in the channel:
- The buoyancy force of the logs is larger than the counteracting ballast weight;
- The resisting forces are smaller than the imposing forces, and
- Scour undermines the structure.
Figure G-44 Simple model of forces on the cohesive log structure. $F_b$ = buoyant force; $W_{bl}$ = weight of ballast material; $F_d$ = total drag force; $F_r$ = friction between the total structure and the river bed (Brooks, 2006)

Figure G-44 represents the forces acting on the log structure. If $F_b$ is larger than $W_{bl}$, the structure floats and there is no drag force on the bottom. It is therefore important to anchor the structure. The calculations use a log density of 900 kg/m$^3$ to represent the worst case scenario of a flood occurring when the logs are completely desiccated.

The stability calculation only checks the first two failure modes. To counteract the third failure method the length of the sand bag lining will be extended to a safe distance downstream of the structure. Scour of the sides of the channels will also be counteracted by lining of the channel with sandbags.

The computational procedure followed for the stability of the log structure is that of Shields et al. (2000) and D’Aoust & Millar (2000). The drag force on the structure exerted by the flow of water is:

$$F_D = 0.5v^2A\rho C_D$$  \[23\]

In which:
- $F_D$ = drag force [N];
- $v$ = mean approach velocity [m/s];
- $A$ = cross sectional area of structure projected into the flow [m$^2$];
- $\rho$ = fluid density (1000 kg/m$^3$);
- $C_D$ = drag coefficient (assumed to be 1.2 (Shields & Knight, 2000)).

The buoyant force of the total volume of logs in the structure is:

$$F_B = (\sum^n K)\rho g(1 - S_L)$$  \[24\]

In which:
- $F_B$ = buoyancy force [N];
- $K$ = total volume of n logs [m$^3$];
- $\rho$ = fluid density (1000 kg/m$^3$);
- $g$ = gravitational acceleration (9.81 m/s$^2$);
- $S_L$ = dry density of logs (0.9 g/cm$^3$).

The volume of a log, $K$, can be calculated with (taper is neglected):

$$K = \pi l \left(\frac{d}{2}\right)^2$$  \[25\]

In which:
- $l$ = log length [m];
- $d$ = diameter of log measured in the log centre [m].

The immersed weight of the ballast material (sand bags) is calculated with:

$$W_{BL} = \rho g \psi(S_S - 1)$$  \[26\]
In which: \( W_{BL} = \) ballast weight \([\text{N}]\);
\( \psi = \) ballast volume \([\text{m}^3]\);
\( S_S = \) specific gravity of ballast \([\text{g/cm}^3]\).

The effect of friction between the log structure and the sandbags is assumed to be represented by a friction angle of 10°. This can be compared with a structure on very weak subsoil. The critical frictional force to initiate sliding of the log structure can be estimated by:

\[
F_{FS} = (W_{BL} - F_B) \tan(\varphi)
\]

In which: \( \varphi = \) friction angle (10°).

The structure is stable when it has a net negative buoyancy \((W_{BL} > F_B)\) and when the structure does not slide \((F_{FS} > F_D)\).

G.8.4 Design of log stilling basin

The length of a concrete stilling basin has to be at least 12.5m to ensure the water returns from supercritical to subcritical flow. To add extra safety the log structure will be elongated to 15m. The structure is composed in three levels. The lowest level consists of 0.20m diameter logs placed taut next to each other perpendicular to the flow. For this level 75 logs of 15m long are required. The logs on the lower level add drag to the flow and protect the sandbags against supercritical flow attack. The middle level consists of 3 logs of 16m (0.20 m diameter) in the direction of the flow. These logs support the logs on the top level. The top level consists of 37 logs of 0.20m diameter evenly spread out over the length of the structure. The logs on the top level allow water to flow through the meshes and slow down the water.

Two extra logs of 7.2 m are added at the end of the structure on the second level perpendicular to the flow. These logs ensure the water returns to subcritical flow before the end of the log structure (Appendix J). The log structure is placed under an angle of 6° in the direction of the flow. The difference between the upstream part and the downstream part of the structure is 1.5 m. By placing the structure under an angle the water level difference between the water flowing over the weir and the water level in St Lucia (+0.45 GMSL) is low enough to avoid supercritical flow at the end of the log stilling basin. When the structure is not placed under angle water will drop a second time. This second drop could cause a lot of scour when it is not counteracted.

The logs of the top level are connected to the logs at lower levels with bolts. The bolts traverse through all three layers of logs. Every second log on the lowest level will be connected to the logs of the middle level by PPE (polyphenylene ether) rope with a cross lashing.

The structure is ballasted by sandbags at both sides of the channel to avoid buoyancy. The width of the channel directly behind the weir is 10 meter at bottom level. The sandbag lined slopes have an angle of 1:1. The bottom directly behind the weir is located on 0m GMSL. At the end of the structure the bottom is located on –1.5 m GMSL.

G.8.5 Stability calculations

When the maximum flow over the weir of 20m³/s is reached the maximum drag on the structure is attained. A simplification of the flow is made because the flow over the weir is supercritical and turns into subcritical flow behind the weir. The equilibrium depth of the flow behind the weir is 1.6 meter. The width of the channel is schematized as 10 meter. The
schematized cross sectional area of the channel is \( 10 \cdot 1.6 = 16 \text{ m}^2 \). The schematized flow velocity through the channel then becomes \( 20/16 = 1.25 \text{ m/s} \).

The cross sectional area of the structure projected into the flow is \( 2 \cdot (0.2 \cdot 10) + (0.25 \cdot \pi \cdot 0.2^2) = 4 \text{ m}^2 \). The drag force becomes:

\[
F_D = 0.5 \cdot 1.25^2 \cdot 4 \cdot 1000 \cdot 1.2 = 3750 \text{ N} = 3.75 \text{ kN}.
\]

The volume of the total log structure is:

\[
K = (75 + 37) \cdot \pi \cdot 15 \cdot 0.25 \cdot 0.2^2 + 3 \cdot \pi \cdot 16 \cdot 0.25 \cdot 0.2^2 + 2 \cdot \pi \cdot 7.2 \cdot 0.25 \cdot 0.2^2 = 55 \text{ m}^3.
\]

The buoyant force becomes:

\[
F_B = 55 \cdot 1000 \cdot 9.81 \cdot (1 - 0.9) = 53955 \text{ N} = 54 \text{ kN}.
\]

As a simplification for the calculation of the ballast weight the angle of the structure is neglected. Only the sandbags above the structure are assumed to be effective as ballast. When the ground level is assumed at +1.8m GMSL the effective cross section of one side becomes: \((2.5-0.6) \cdot 1.2 - 0.5 \cdot 1.2^2 = 1.56 \text{ m}^2 \). The total effective volume becomes: \((2 \cdot (15-1.56)) = 46.8 \text{ m}^3 \). When the specific gravity of ballast is set at 2g/cm\(^3\) the effective ballast weight becomes:

\[
W_{BL} = 1000 \cdot 9.81 \cdot 46.8 \cdot (2 - 1) = 459108 \text{ N} = 459 \text{ kN}.
\]

The ballast weight is large when compared to the buoyant force. The effectiveness of the structure is only ensured when it does not flow. The ballast weight ensures that the structure stays at the bottom of the channel in the worst case scenario. This is the scenario in which a flood hits a complete desiccated structure.

The critical frictional force to initiate sliding of the log structure becomes:

\[
F_{FS} = (459 - 54) \cdot \tan 10^\circ = 71.4 \text{ kN}.
\]

The critical frictional force is not exceeded by the drag force. The negative simplifications of the resisting forces do not interfere with the stability of the total structure.

G.8.6 Scour protection

According to section B.5 the critical flow velocity for the muddy soil in the Back Channel is approximately 0.7 m/s. The flow velocity should remain below 0.7m/s in the unlined channel in order to prevent scour. With a discharge of 20m\(^3\)/s the minimal required cross section is 29m\(^2\) when the desired flow velocity is 0.7m/s. The equilibrium depth after the hydraulic jump is 1.6m. The required width of an unlined prismatic channel is at least 18 meter. The width of the channel near the energy dissipater is 10 meter. A transition structure is required to bring flow velocities down to an acceptable level without causing turbulence. A sudden increase in width causes large vortexes that could damage the construction. Width should be increased gradually in order to keep the flow pattern regular. In general angles larger than 13 degrees should be avoided (Schiereck). The AutoCAD drawing of Appendix J shows the transition structure.
G.9 Construction method

This paragraph elaborates the construction of the new Back Channel and weir. The first section describes the required materials (section G.9.1). In Section two the types and amount of equipment needed for construction are described (section G.9.2). A proposed construction sequence is given in section G.9.3. Finally site depending remarks are given in section G.9.4.

G.9.1 Materials

The weir structure is build up out of 4 construction materials: timber, logs, beach sand and geotextile. This section consists of coarse calculations on the amount of materials involved in the construction of the weir. The materials that are required to make joints or connections are not taken into account.

Timber

The skeleton of the box shaped structure is build up of timber. All parts are displayed in Figure G-45.

- Top and bottom planks: 96 times 1880 x 300 x 60 mm;
- Top and bottom planks: 12 times 1880 x 100 x 60 mm;
- Side planks and baffle planks: 78 times 2500 x 300 x 60 mm;
- Column standard outside: 10 times 1800 x 160 x 160 mm;
- Column standard inside: 10 times 1680 x 160 x 160 mm;
- Column baffle outside: 4 times 2700 x 160 x 160 mm;
- Column baffle inside: 4 times 2640 x 160 x 160 mm;
- Girders (parallel to flow): 14 times 1560 x 160 x 160 mm;
- Girders (perpendicular to flow): 24 times 2340 x 160 x 160 mm;
- Diagonals: 14 times 1960 x 100 x 40 mm.

Logs

The log stilling basin is fabricated with 0.20m diameter *Eucalyptus grandis* logs. The base layer consists of 75 logs with a length of 15m. The middle layer consists of 3 logs with a
length of 16m and two smaller logs with a length of 7.2m each. The top layer consists of 37 logs with a length of 15m. Next to the logs of the stilling basin, the wooden box of the weir is supported on 31 logs with a length of 4 m and a diameter of 0.20m.

**Beach sand**
The volume of beach sand needed for the construction is calculated by roughly dividing the total structure in different parts and adding up these separate volumes. The volume of all parts summed up gives the total volume of beach sand which is approximately 300m³ (295 m³). When possible, the volume was calculated by multiplying the number of sandbags with its volume. When assuming a dump truck load of 20 m³, 15 loads are required.

- Bottom and embankment upstream side $18.60 \times 10 \times 0.25 = 46.5$ m³;
- Volume weir: $3.16 \times 15 = 47.4$ m³;
- Directly next to weir: $6 \times 18 \times 1 \times 0.5 \times 0.25 = 13.5$ m³;
- Under the weir: $32 \times 20 \times 0.2 \times 0.4 \times 0.5 = 25.6$ m³;
- Embankment downstream side: $48 \times 2 \times 1 \times 0.5 = 48$ m³;
- Bottom downstream side: $15 \times 15 \times 0.25 = 56.25$ m³;
- Directly behind log stilling basin: $15 \times 0.75 \times 1 = 11.25$ m³;
- Widening behind stilling basin: $18.60 \times 10 \times 25 = 46.5$ m³.

**Geotextile: Underlayment / Sandbags**
The amount of geotextile can be divided into two parts: the geotextile sub layer and the sandbags. The surface needed for the underlayment of the total structure has been calculated by calculating the overall length needed and multiplying this by a width of 30 m. The width includes the geotextile required for anchoring the geotextile with sandbags. It also includes some spare geotextile. The geotextile needed for the sandbags is then calculated by multiplying the surface for the underlayment by 2 (top and bottom) and a surplus factor of 2.0 for the layers of sandbags. This adds up to a total surface of 6800 m².

- Under layer: $(2.5 + 8.0 + 1.0 + 0.5 + 6.0 + 13.0 + 1.0 + 0.75 + 10.0 + 2.5) \times 30m = 1360m^2$;
- Sandbags: $1360 \times 2 \times 2 = 5440$ m².

**Excavated soil**
In order to estimate the amount of soil that has to be excavated the total structure is divided into parts. When the volumes of all parts are summed up and then subtracted with the volume of the existing new Back Channel, one gets the volume which needs to be excavated. The total length of the new Back Channel is assumed to be 40 m. This leads to a total excavation volume of 860 m³ (856.2 m³).

- New Back Channel until narrowing part of channel: $1/2 \times (15 + 18.6) \times 1.8 \times 3 = 91$ m³;
- New Back Channel narrowing part: $1/2 (1/2 (18.6 + 15) + 1/2 (15 + 10)) \times 2.05 \times 10 = 300$ m³;
- Weir itself: $15 \times 2.7 \times 2 = 81$ m³;
- Log stilling basin: $15 \times 16.2 \times 2.55 = 620$ m³;
- Widening of the new Back Channel: $1/2 (15 + 10) \times 1.8 \times 10 = 225$ m³;
- Old Back Channel: $1/2 (4.8 + 8) \times 1.8 \times 40 = 460.8$ m³.

**Summary**
A summary of all materials involved in the construction of the weir is given in Table G-17. The volumes of beach sand, excavated soil and the surface of the geotextile/sandbags are coarse figures.
### Table G-17 Summary of materials

<table>
<thead>
<tr>
<th>Material</th>
<th>Amount</th>
<th>Dimensions/Area/Volume</th>
</tr>
</thead>
<tbody>
<tr>
<td>Timber</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>96</td>
<td>1880 x 300 x 60 mm</td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>1880 x 100 x 60 mm</td>
</tr>
<tr>
<td></td>
<td>78</td>
<td>2500 x 300 x 60 mm</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>1800 x 160 x 160 mm</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>1680 x 160 x 160 mm</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>2700 x 160 x 160 mm</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>2640 x 160 x 160 mm</td>
</tr>
<tr>
<td></td>
<td>14</td>
<td>1560 x 160 x 160 mm</td>
</tr>
<tr>
<td></td>
<td>24</td>
<td>2340 x 160 x 160 mm</td>
</tr>
<tr>
<td></td>
<td>14</td>
<td>1960 x 100 x 40 mm</td>
</tr>
<tr>
<td>Logs</td>
<td>31</td>
<td>4.00 x Ø0.20 m</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>7.20 x Ø0.20 m</td>
</tr>
<tr>
<td></td>
<td>112</td>
<td>15.00 x Ø0.20 m</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>16.00 x Ø0.20 m</td>
</tr>
<tr>
<td>Beach sand</td>
<td>-</td>
<td>300 m³</td>
</tr>
<tr>
<td>Geotextile: Underlayment/</td>
<td>-</td>
<td>1360 / 5440 m²</td>
</tr>
<tr>
<td>Sandbags</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Excavated soil</td>
<td>-</td>
<td>860 m³</td>
</tr>
</tbody>
</table>

### G.9.2 Equipment and labour

During the construction sequence of the weir several different types of equipment are required. In this section a list is given of equipment which could be used during construction.

- **Bulldozer:**
  A bulldozer can be used for the enlargement of the construction road towards the new Back Channel. Further, it can be used for moving earth at the construction site. One bulldozer is sufficient.

- **Excavator / Backhoe:**
  An excavator is needed for the excavation of the new channel. The excavator can also be used for hoisting loads on the worksite. One excavator is sufficient.

- **Dump trucks:**
  Dump trucks are needed for moving loads from an towards the construction site. Two dump trucks are sufficient.

- **Land survey equipment:**
  To ensure a proper construction, a land survey team is needed to survey construction depths.

- **Geotextile/Sandbag equipment:**
  Equipment for the handling of geotextile is required to make connections in the geotextile. When needed, this equipment can also be used for the manufacturing of sandbags.

- **Sandbag filling equipment:**
  Special equipment is needed for filling of the large sandbags. Small sandbags can be filled by hand.

- **Woodworking equipment:**
  Woodworking equipment is needed for the construction of all woodwork. This equipment should be handled by a skilled labour force.

- **Labour:**
  Skilled labour is needed for handling of the different types of equipment. Next to that, additional workmen are required for other activities like placing sandbags.
G.9.3 Construction sequence
This section describes the construction sequence of the weir structure. A drawing clarifies the stage of construction in Figure G-47.

1. Making the supply route of Figure G-46 passable for the construction equipment
   The current route towards the new Back Channel is too small for trucks to drive on. This route has to be enlarged to allow the passage of one piece of the largest equipment at a time.

2. Removal of vegetation and earlier excavated soil
   Vegetation and earlier excavated soil at the construction site has to be removed to ensure an object free working space.

3. Worksite preparation
   The project site has to be prepared for the construction to take place. This includes: installing a shack; applying steel planking to allow for equipment circulation at the worksite and overnight storage of the equipment and organizing a storage site for the building materials.

4. Apply closures at both sides
   First, the Mfolozi side of the new Back Channel has to be closed to stop the flow. When the flow of water into St Lucia is stopped, the St Lucia side has to be closed. Both closures can be done by excavator.

5. Excavate weir part of channel
   To preserve working space only the weir part of the channel will be excavated at first.

6. Installation of drainage equipment if needed
   When water seeps through the closure at the Mfolozi side, drainage equipment has to be installed to create a dry working space in the channel.

7. Leveling excavated unlined channel
   The bottom of the excavated part has to be 'compacted' to create a solid surface on which the weir can be constructed. This can be done by 'compacting' the subsoil by using the weight of the crane. After the compaction the subsoil has to be leveled to
create a flat surface. Checks have to be carried out with land surveying equipment if the correct heights are applied.

8. Installation of geotextile
Geotextile has to be installed at the bottom and the sides. It is important to check the geotextile does not contain folds or tears to ensure the structure is watertight. All joints or seals have to be checked on strength and quality.

9. Apply base layer of sandbags
Before the construction of the wooden structure can start, the first layer of sandbags will have to be applied. Sand is transported by dump truck from the beach towards the project site. Filling of the sandbags takes place at the construction site itself. Placing of the sandbags can be done by hand or by crane.

10. Construction of woodwork
All woodwork has to be constructed on top of the base layer of sand bags. It is important that all structures are levelled and placed on the correct heights. The wooden logs of the stilling basin will have to be hoisted in by excavator (> 500 kg per log). The joints of the wooden box will be made with simple connections like nails, screws and bolts. The first log on the lower layer will be connected to the second and top layer with threaded bolts. The second log of the lower layer will be connected to the second layer by lashing with polyethylene rope. The third log is again connected with threaded bolts to the second and top layer, etcetera.

11. Filling weir with sandbags (compact!)
When the wooden box is ready it can be filled with sandbags. It is of utmost importance this happens correctly. No gaps between sandbags or between a sandbag and the weir are allowed. When the box is filled it can be closed off.

12. Apply top layer of sandbags
When all woodwork is finished the next layers of sandbags can be applied where they are needed. The placing should be done precisely to assure a stable structure.

13. Excavate remaining part new Back Channel
When the construction of the weir structure is finished the remaining part of the new Back Channel can be excavated.

14. Final check
The total structure has to be checked on flaws. These flaws have to be repaired and have to be checked again. Only when all checks are passed the next step may be executed.

15. Remove drainage equipment
When all checks are passed, the drainage equipment can be removed. Water levels will rise to the level of the weir.

16. Remove closures
When the channel is filled with water the closures at both sides can be removed. First, the closure at the St Lucia side has to be removed. The closure at the Mfolozi side has to be removed during low tide to avoid rapid flows.

17. Clean-up of worksite
All equipment has to be removed and the project site has to be cleaned up.
The Mfolozi floodplains and the St Lucia estuary are very dynamic locations. The weather, floods and the local flora and fauna reshape the area constantly on minor and major scales. This is something to keep in mind during construction of the weir. Construction should take place during dry periods. During this time the subsoil is much stronger than during wet periods. Figure G-48 shows the excavator which got stuck during the excavation of the small new Back Channel. It is recommended to place equipment on steel plates at some distance from the excavation site to avoid overnight sinking of equipment. This also avoids damage to the construction site in case equipment does slide in the construction pit. The GEF project, of
which the new Back Channel is partly funded, also supports local communities. By making use of local labour for the construction of the new Back Channel these communities can also be stimulated.

Figure G-48 Excavator that got stuck in the mud

G.10 Verification of discharge requirement

Besides dealing with a design flood peak as described in G.2, the system must also divert enough water into the St Lucia Estuary. The requirement as described in the list of requirements states:

“R100) The total amount of water delivered to the iSimangaliso wetland park should be of the order of \(60 \times 10^6\) m\(^3\) per year or approximately \(5 \times 10^6\) m\(^3\) per month.”

In order to verify this requirement the rates of flow of the dry season of 2011 (1 June 2011 till 19 October) are used as input data in the SOBEK model. The data used for this verification is listed in Table B-1 to Table B-5 in section B.1. In addition it is displayed in Figure G-49. The data used for this graph is daily data, which has a drawback compared to the detailed 12 minute data. The maximum discharge is probably not included in the daily data series dailty data only features one point in time. This has been described in detail in section B.3. The daily data is a good estimate for the average rate of flow during a season.

Figure G-49 Daily discharge during the dry period of 2011
The Mfolozi berm should not be breached during the dry period according to paragraph G.1. When the Mfolozi berm remains closed most of the Mfolozi runoff will flow through the Bak Channel into the St Lucia estuary. To verify this the water level at the berm over time has been calculated using SOBEK. Figure G-50 shows that the water level at the berm and the floodplains remains lower than the +1.8m GMSL. The berm did not breach in the model run. If the New Channel and weir were constructed before June 2011 the total flow through the Back Channel would have amounted to 60Mm$^3$.

Some remarks have to be addressed:

- Subtraction of water by the sugar cane farmers is not taken into account.
- It is assumed that the hypsometric curve of Clinton Chrystal is correct.
- Direct data of the discharges of this period were not available. The discharge has been calculated from water levels measured by the W2H032 gauge and converted into discharges based on the measurements done on the field trip and Figure I-62. The relation between water level and discharge is assumed to be linear during low flows.
- It is assumed that the reading of the gauge is correct.
- Maximum discharges of a flood peak are not included in the daily data.
- No seepage occurs through the berm.

![Figure G-50 Modeled water level with the new Back Channel in place during the dry period of 2011.](image)

From Figure G-50 it can be concluded that water levels behind the berm would not have exceeded +1.5m GMSL during the period of closure. As stated earlier the maximum flows of the flood peaks are not taken into account. Short duration peaks can cause high water level behind the berm. The Mfolozi berm should be maintained at +1.8m GMSL to prevent it from breaching during dry periods.

According to the model the total discharge of the Mfolozi between June and October is equal to 65Mm$^3$. A great part of this discharge would have flowed into the St Lucia estuary if the Mfolozi berm would have remained closed after the first of June 2011. The requirement of 60 Mm$^3$ would be met if the new channel and weir was constructed before June 2011.
In addition the maximum flows have been calculated for the dry seasons in the years 1998 to 2011. The results are shown in Table G-18. From this table can be concluded that the maximum discharge does not exceed 75m³/s during the dry period. Due to small floods with a discharge of up to 15m³/s the water level behind the berm rises to +1.5m GMSL as shown in section I.4.6. Larger floods will cause higher water levels than +1.5m GMSL.

In order to keep the Back Channel operational a higher artificial berm breach threshold height is required. With a Mfolozi berm height of +1.8m GMSL it is possible to retain the floods with a peak discharge lower than 75m³/s. By retaining these floods the uptime of the channel and weir can be extended greatly to divert more fresh water into St Lucia.

Table G-18 maximum discharge and during June till September

<table>
<thead>
<tr>
<th>year</th>
<th>data</th>
<th>maximum discharge (m³/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1998</td>
<td>good</td>
<td>15</td>
</tr>
<tr>
<td>1999</td>
<td>good</td>
<td>10</td>
</tr>
<tr>
<td>2000</td>
<td>missing data</td>
<td></td>
</tr>
<tr>
<td>2001</td>
<td>missing data</td>
<td></td>
</tr>
<tr>
<td>2002</td>
<td>Good but spillway active</td>
<td>783</td>
</tr>
<tr>
<td>2003</td>
<td>missing data</td>
<td></td>
</tr>
<tr>
<td>2004</td>
<td>good</td>
<td>35</td>
</tr>
<tr>
<td>2005</td>
<td>good</td>
<td>12</td>
</tr>
<tr>
<td>2006</td>
<td>good</td>
<td>13</td>
</tr>
<tr>
<td>2007</td>
<td>good</td>
<td>58</td>
</tr>
<tr>
<td>2008</td>
<td>good</td>
<td>9</td>
</tr>
<tr>
<td>2009</td>
<td>good</td>
<td>26</td>
</tr>
<tr>
<td>2010</td>
<td>missing data</td>
<td></td>
</tr>
<tr>
<td>2011</td>
<td>good</td>
<td>11</td>
</tr>
</tbody>
</table>
G.11 Risk analysis

In this chapter integral safety is implemented into the systems engineering process. A lot of the risks that have been described are related to the requirements in paragraph 2.1. Implementing integral safety into the design process does not mean that the draft design is complete safe and all risks are covered. Integral safety must be applied in every phase of the project life cycle. Uncertainties are never completely covered and new uncertainties could arise in future.

G.11.1 Definitions
First a brief description of safety, risks, and uncertainty will be given. Safety is perceived differently from person to person, from one group to another. According to Suddle & Waarts (2003) safety is being able to live without the risk of being injured in any way. The term safety is divided into two groups, which are social safety and physical safety. Furthermore, physical safety is split into natural and man-made hazards, and the latter can be distinct in internal and external safety (see Table G-19). In Suddle & Waarts (Suddle & Waarts, 2003), safety includes both subjective (psychological) and objective (mathematical) parts.

<table>
<thead>
<tr>
<th>Main groups</th>
<th>Sub-groups</th>
<th>Sub-sub-groups</th>
<th>Elements:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Integral safety</td>
<td>Man-made hazards</td>
<td>Internal safety</td>
<td>Internal safety of buildings</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Traffic safety</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Labour safety</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Tunnel safety</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Fire safety</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Transport safety</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Construction safety</td>
</tr>
<tr>
<td></td>
<td>External safety</td>
<td></td>
<td>Stationary installations</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Windmills</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Aviation safety</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Transport of hazardous materials</td>
</tr>
<tr>
<td>Natural hazards</td>
<td></td>
<td></td>
<td>Floods</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Earthquakes</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Meteorites</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Remaining climatic factors</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Diseases and epidemics</td>
</tr>
<tr>
<td>Social safety</td>
<td></td>
<td></td>
<td>Terrorism</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Criminology</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Institution</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Design</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Sociology</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Perception</td>
</tr>
</tbody>
</table>

Table G-19 Safety (Suddle & Waarts, 2003)

“Project risk is an uncertain event that, if it occurs, has a positive or a negative effect on project objectives. A risk has a cause and, if it occurs, a consequence.” (Project Management Institute, Inc., 2000). A risk has a likelihood, impact, and expected value. The likelihood is a number that describes the certainty of occurring. Secondly, the impact can be translated in the cost associated with a risk event. Finally, the expected value can be calculated by multiplying the likelihood with the impact and could be calculated in a financial quantity.

The relationship between safety and risk circles around trying to give safety a numerical value. The numerical value could be quantified to assess it in an acceptable way. Risk
prevention versus the consequence of an occurring risk could be compared through financial expected values.

Uncertainty refers to the uncertainty of the outcome. “One may have uncertainty without risk but not risk without uncertainty” (Wikipedia, 2011) Risk is considered to be dependent on the probabilities and quantified losses. Uncertainty refers only to the probabilities assigned to outcomes. Uncertainties that do not have an impact on the project promises could be described as a possible event instead of a risk.

It is never too early to start the risk management plan for a project, it is vital to know the risks associated with a project to be able to decide whether to invest in it or not. To complete a risk management process there are several steps that need to be taken. These are, according to PMBOK (Project Management Institute, Inc., 2000):

1. Risk management planning
2. Risk identification
3. Qualitative risk analysis (overall risk ranking)
4. Quantitative risk analysis
5. Risk response planning
6. Risk monitoring and control

For this project step 2 and 3 have been described briefly below. In addition, different response strategies have been described briefly to counteract the risks’ probability and impact.

G.11.2 Risk identification & qualitative risk analysis

“The risk identification is an iterative process which uses the risk management plan, the project plan, risk categories, and historical information to produce a risk register that needs to be reviewed on a regular basis by the project team and key stakeholders. The risk identification will produce a list of risks and their triggers which will be used in the next steps.” (Project Management Institute, Inc., 2000)

A part of the risk management plan is the categorizing the risks (Project Management Institute, 2000). The PMBOK distinct the categories as:

- Technical, quality or performance risks,
- Project management risks,
- Organizational risks,
- External risks.

The impact of the risk (e.g. the cost) and its likelihood is estimated and a risk assessment matrix is made (see Figure G-51). The red risks are the most important. They have a high probability and a high impact. The orange risks are moderate risks and the green risks are the least important. On basis of the results of the risk impact matrix decisions are made to take or manage the risk(s). “Black swans” should be taken into account with cautiousness. The latter are risks that have a high or very high impact or consequence, but a low or very low probability. The chance exists that these black swans might be overlooked.
According to the PMBOK2000 “the qualitative risk analysis uses the risk management plan and the identified risks along with the status and type of the project, the scale of probability and impact (e.g. probability/impact matrix and FMEA below) and assumptions made during the process so far. The output of this step is an overall risk ranking for the project with a list of prioritized risks and list of risk that need additional analysis and trends in risk analysis results.” (Project Management Institute, Inc., 2000)

Risk evaluation is determining whether to avoid, accept or manage a risk. The 4T model describes four considered actions for managing risks. The model distinct “Take” (e.g. evaluate whether expected returns are worth paying (‘residual’ or ‘net’ risk), developing contingency/recovery plans for when the risk does occur), “Treat” (e.g. by hedging the risk, or allocation of risk across other areas), “Transfer” (Insure, share, contract out, diversify, hedge the risk), and “Terminate” (Ceasing activity, sale, divestment, recalibrate, reduce scale of the risk). (Verbraeck, 2009)

The “actual” risks are not the only risks of a project. Secondary risks are risks that are the consequence of actions of the contingency plan to counteract a primary risk. An evaluation between the values of the primary and secondary risks would be recommended. However, the secondary risk is sometimes hard to predict. Finally, there is also a residual risk. A residual risk is the risk that is “left” after the contingency plan has been implemented.

G.11.3 Scope of the risk analyses
A qualitative risk analysis has been worked out. The impact and the frequencies are translated into a five point scale and the risks are prioritized by giving each risk a priority and an impact quantity. The probability and the impact will be assigned by a five point scale: VL (very low), L (low), M (medium), H (high), and VH (very high). After evaluating the risks a response strategy is chosen. Actions or measures have to be determined to achieve the strategy. Comparing the latter with a global cost effectiveness analysis could help to decide
what action/measure has to be implemented. The output of the risk analysis is used as an additional input in the requirements list.

*Risk register and risk impact matrix*

Only those risks will be assessed that will have a consequence on the allocation of the different functions in the area. The risks will be identified and a selection was made about which risks have to be assessed by looking at the consequences of the risks.

The results of the analysis are represented below. To define the risks the following sentence has been used: “Because of cause, this event which might happen, will have potential effects ‘a’ and ‘b’ on project promises.”

The risk impact matrix that is relevant to the start-up of the project risks is shown in Table G-21. The risks are described in the risk register that is shown in Table G-20. Risks that are relevant to the implementation of the design are shown in Table G-22 and Table G-23. Finally, the risk impact matrix of the latter risk register is shown in Table G-24.

**G.12 Verification table**
### Table G-20 Risk register pre design phase

<table>
<thead>
<tr>
<th>Promise</th>
<th>Category</th>
<th>Cause</th>
<th>Risk event</th>
<th>Consequences</th>
<th>Likelihood</th>
<th>Impact</th>
<th>Strategy</th>
<th>Residual risk</th>
<th>Secondary risk</th>
<th>Risk score</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Quality</td>
<td>Construction</td>
<td>Lack of good communication</td>
<td>Interactions between the clusters fail</td>
<td>The design has to be adjusted</td>
<td>L</td>
<td>M</td>
<td>Treat (e.g. maintaining communication and collaboration between the two design groups)</td>
<td>Always chance of human error</td>
<td>Process takes longer</td>
</tr>
<tr>
<td>2</td>
<td>Quality</td>
<td>Safety</td>
<td>A lack of data</td>
<td>There are inconsistencies in the design</td>
<td>Decreases the reliability of the research</td>
<td>H</td>
<td>H</td>
<td>Take (e.g. develop a contingency plan)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Time</td>
<td>Safety</td>
<td>Inadequate specifications</td>
<td>The structure encounters more problems during the further design phases</td>
<td>The design has to be adjusted</td>
<td>M</td>
<td>VH</td>
<td>Treat (e.g. putting a lot of effort in the beginning of the project to counteract all problems)</td>
<td>Always chance of human error</td>
<td>Process takes longer</td>
</tr>
<tr>
<td>4</td>
<td>Scope</td>
<td>Organization</td>
<td>Actors perception change</td>
<td>The process faces more opposition</td>
<td>The design process takes longer</td>
<td>VL</td>
<td>M</td>
<td>Treat (e.g. informing the different actors)</td>
<td>Actors might hold on to their own morals and</td>
<td>The design process becomes more</td>
</tr>
<tr>
<td>5</td>
<td>Scope</td>
<td>Political</td>
<td>Change in EIA policies</td>
<td>The Back Channel design requires an EIA</td>
<td>The speed of implementation could suffer</td>
<td>VL</td>
<td>H</td>
<td>Take (e.g. develop a contingency plan in case something goes wrong)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Notes:**
- VL = Very Low, L = Low, M = Moderate, H = High, VH = Very high
- **Strategy** sections may contain suggestions for dealing with identified risks, such as developing contingency plans or enhancing communication and collaboration.

### Table G-21 Risk impact matrix pre design phase

<table>
<thead>
<tr>
<th>Probability</th>
<th>Consequences</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 – Insignificant</td>
<td>A</td>
<td>B</td>
<td>C</td>
<td>D</td>
<td>E</td>
<td></td>
</tr>
<tr>
<td>2 – Minor</td>
<td>A</td>
<td>B</td>
<td>C</td>
<td>D</td>
<td>E</td>
<td></td>
</tr>
<tr>
<td>3 – Moderate</td>
<td>A</td>
<td>B</td>
<td>C</td>
<td>D</td>
<td>E</td>
<td></td>
</tr>
<tr>
<td>4 – Major</td>
<td>A</td>
<td>B</td>
<td>C</td>
<td>D</td>
<td>E</td>
<td></td>
</tr>
<tr>
<td>5 – Catastrophic</td>
<td>A</td>
<td>B</td>
<td>C</td>
<td>D</td>
<td>E</td>
<td></td>
</tr>
<tr>
<td>Promise</td>
<td>Category</td>
<td>Cause</td>
<td>Risk event</td>
<td>Consequences</td>
<td>Likelihood</td>
<td>Impact</td>
</tr>
<tr>
<td>---------</td>
<td>----------</td>
<td>-------</td>
<td>------------</td>
<td>--------------</td>
<td>------------</td>
<td>--------</td>
</tr>
<tr>
<td>1</td>
<td>Quality</td>
<td>Safety</td>
<td>During floods</td>
<td>A big object flushes over the structure</td>
<td>M M</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Quality</td>
<td>Safety</td>
<td>Erosion under the structure</td>
<td>The geotextile underneath the structure is damaged</td>
<td>VL M</td>
<td>Take (e.g. develop a contingency plan to close the Mfolozi berm)</td>
</tr>
<tr>
<td>3</td>
<td>Quality</td>
<td>Safety</td>
<td>There is a fault in the design calculation</td>
<td>Structural strength of the weir not sufficient</td>
<td>L VH</td>
<td>Treat (e.g. providing enough sandbags, anchoring the weir); Transfer (e.g. responsibility to contractor)</td>
</tr>
<tr>
<td>4</td>
<td>Quality</td>
<td>Safety</td>
<td>Mfolozi berm does not breach at +1.8m GAMS</td>
<td>Flooding over and around the weir</td>
<td>VL VH</td>
<td>Treat (e.g. monitor water levels)</td>
</tr>
<tr>
<td>5</td>
<td>Quality</td>
<td>Safety</td>
<td>The lack of professional laborers</td>
<td>Sand bags are not placed correctly</td>
<td>L H</td>
<td>Treat (e.g. Providing instructions for the placement of sand bags)</td>
</tr>
<tr>
<td>6</td>
<td>Quality</td>
<td>Safety</td>
<td>Hippo walk over the stilling basin Wooden logs rot away</td>
<td>The wooden log structure could break</td>
<td>L H</td>
<td>Treat (e.g. design hippo unfriendly slopes; use wood with a longer durability)</td>
</tr>
<tr>
<td>7</td>
<td>Quality</td>
<td>Safety</td>
<td>Different site conditions</td>
<td>The construction settles even more than calculated</td>
<td>M VH</td>
<td>Treat (e.g. taking high safety factors in the design calculations)</td>
</tr>
<tr>
<td>8</td>
<td>Quality</td>
<td>Ecologic</td>
<td>High water discharge in the Mfolozi</td>
<td>The berm may be (artificially) breached</td>
<td>M VH</td>
<td>Take (e.g. develop a contingency plan to close the Mfolozi berm)</td>
</tr>
<tr>
<td>9</td>
<td>Quality</td>
<td>Ecologic</td>
<td>Lack of rain</td>
<td>The water levels in the Mfolozi are very low</td>
<td>L H</td>
<td>Take (e.g. develop a contingency plan to close the Mfolozi berm)</td>
</tr>
<tr>
<td>10</td>
<td>Quality</td>
<td>Ecologic</td>
<td>Mfolozi berm is closed</td>
<td>Marine distribution between does not take place</td>
<td>H L</td>
<td>Take (e.g. develop a contingency plan)</td>
</tr>
<tr>
<td>Table G-23 Risk register design implementation part 2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>-----------------------------------------------------</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>11 Quality Ecologic</strong></td>
<td>The Back Channel dimensions are small</td>
<td>There is a lack of marine distribution between the sea and St Lucia</td>
<td>The opportunities for fishery do not improve sufficient</td>
<td>H</td>
<td>Take (e.g. develop a contingency plan) or Treat (e.g. realising a culvert construction in the Weir)</td>
<td>Impact on the lakes could be small due to the small dimensions</td>
</tr>
<tr>
<td><strong>12 Quality Technology</strong></td>
<td>Monitoring equipment is very valuable and the local inhabitants are very poor</td>
<td>Monitoring equipment might be stolen</td>
<td>There are data gaps and the accuracy of research drops</td>
<td>Reliability of the data decreases</td>
<td>L</td>
<td>Treat (e.g. placing them on hidden places, placing anti theft locks)</td>
</tr>
<tr>
<td><strong>13 Quality Political</strong></td>
<td>The farmers are allowed to artificially breach below +1.5m GMSL</td>
<td>The Mfolozi berm may be (artificially) breached below +1.5m GMSL</td>
<td>Decreases the functionality of the Back Channel</td>
<td>Salinity levels increase in the St Lucia Lakes</td>
<td>L VH</td>
<td>Terminate (e.g. by proposing a law that prevents the farmers from breaching below +1.5m GMSL)</td>
</tr>
<tr>
<td><strong>14 Cost Project Management</strong></td>
<td>There is no money available for the project</td>
<td>The new Back Channel and Weir are not being implemented</td>
<td>Small floods cannot be kept into the system</td>
<td>The natural control notch of the old Back Channel could scour away</td>
<td>The water levels in the Mfolozi are rising faster when the berm is closed</td>
<td>Farmers face more drainage problems</td>
</tr>
<tr>
<td><strong>15 Time Project Management</strong></td>
<td>Political influences during decision making and conflicts about required solution occur</td>
<td>The construction of the Back Channel could be postponed</td>
<td>Lack of fresh water into St Lucia estuary</td>
<td>Increase of salinity levels in St Lucia Lakes</td>
<td>M VH</td>
<td>Treat (e.g. putting a lot of pressure on the need for speed in the design process)</td>
</tr>
<tr>
<td><strong>16 Scope External effects</strong></td>
<td>A massive flood that occurs once in the 25 years</td>
<td>The area gets flushed away</td>
<td>The complete system dynamics could change</td>
<td>The Mfolozi and St Lucia mouth combine</td>
<td>VL VH</td>
<td>Take (e.g. develop a contingency plan in case something goes wrong)</td>
</tr>
<tr>
<td><strong>17 Scope Organization</strong></td>
<td>Conflicts and mismanagement in the iSimangaliso Wetland Park in the past</td>
<td>The park authority does not want to intervene in the natural situation</td>
<td>The new Back Channel and Weir are not implemented Design process suffers from a lack of speed Small floods cannot be kept into the system The natural control notch of the old Back Channel could scour away The water levels in the Mfolozi are rising faster when the berm is closed Farmers face more drainage problems</td>
<td>The decision remains at the iSimangaliso Wetland Park authority</td>
<td>H H</td>
<td>Treat (e.g. putting a lot of pressure on the need for urgency and speed in the design process)</td>
</tr>
</tbody>
</table>

VL = Very Low, L = Low, M = Moderate, H = High, VH = Very High
Table G-24 Risk impact matrix implementation design

<table>
<thead>
<tr>
<th>Probability</th>
<th>A: almost certain to occur</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E: rare - occurs only in exceptional circumstances</th>
</tr>
</thead>
<tbody>
<tr>
<td>Consequences</td>
<td></td>
<td>10</td>
<td>12</td>
<td>2</td>
<td>5, 6, 9, 14</td>
</tr>
<tr>
<td>1 - Insignificant</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2 - Minor</td>
<td>11</td>
<td>1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3 - Moderate</td>
<td>17</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4 - Major</td>
<td></td>
<td></td>
<td></td>
<td>3, 13</td>
<td></td>
</tr>
<tr>
<td>5 - Catastrophic</td>
<td></td>
<td></td>
<td>7, 8, 15</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Measures implementation phase**

In this paragraph the measurement that could be taken for the most important risks during the implementation phase are elaborated. The measurements or response strategy could reduce the risk of occurrence and/or its impact. It follows from Table G-24 that the risks 3, 4, 7, 8, 13, 15, 16, and 17 fall in the red and “black swan” area. The measurements that could be taken to decrease the probability of these eight risks or their impact is elaborated briefly. The different 4T’s strategies, which were Take, Treat, Terminate, Transfer, have been used to describe the response strategies (see Appendix G.11.2).

The third risk is defined as:
Because of a fault during the design calculations, there is a chance that the structural strength of the weir is not sufficient, which might happen, leading to the need for repairs or rebuilding of the weir, erosion, damage of the reputation of the Dutch students, and injured or dead wildlife and compromises the projects’ quality and costs.
The strategy that has been chosen to counteract this risk is to treat and/or transfer the risk. Treating the risk has been done by evaluating the design to the requirements and by validating the structures calculations by multiple persons. Safety measures have been taken by providing enough stability to the structure by anchoring it in the soil and through extra sandbags. In addition, the responsibility of the structure could be transferred to a contractor if preferred.

However, the chance of a human error always exists and scour cannot be prevented. Scour can only be distributed to another location, which is preferred to be as far as possible from the construction.

The fourth risk is defined as:
Because the Mfolozi berm does not breach at +1.8m GMSL, there is a chance of flooding over and around the weir, which might happen, will lead to the need for repairs or rebuilding of the weir, the perception of safety, flushing away of sand bags, and flooding on the sugarcane yields on compromises the project quality and costs.
The strategy that has been chosen to counteract this risk is to treat the risk. By monitoring the water levels and discharge of the Mfolozi and the Back Channel insight is gained in the systems functioning. Whenever the levels arise +1.8m GMSL the berm has to be (artificially) breached in order to prevent damage. However, whether or not the berm is breached, during big floods there is always a chance of flooding.

The seventh risk is defined as:
Because of different site conditions, there is a chance that the construction settles even more then calculated which might happen, will lead to the loss of the control notch function of the weir and requires repairing or rebuilding of the weir and compromises the project quality.

The strategy that has been chosen to counteract this risk is to treat the risk. The risk is treated by taking into account high safety factors in the design calculations. However, accidents cannot be fully prevented and therefore the residual risk of settlements always remains.

The eighth risk is defined as:
Because of a high water discharge in the Mfolozi, there is a chance that the berm may be (artificially) breached, which might happen, will lead to a decreased functionality of the Back Channel and an increase of the salinity levels of the St Lucia lakes during droughts and compromises the projects’ quality.

The strategy that has been chosen to counteract this risk is to take the risk and develop a contingency plan. The water discharge of the Mfolozi is depended on the weather and measures to lower the discharge are unreasonable. The contingency plan could consist of a plan to close the Mfolozi after the discharge dropped to restore the functioning of the Back Channel.

The thirteenth risk is defined as:
Because the farmers are allowed to artificially breach below +1,5m GMSL, there is a chance that the berm may be (artificially) breached below +1.5m GMSL, which might happen, will lead to a decreased functionality of the Back Channel and an increase of the salinity levels of the St Lucia lakes during droughts and compromises the projects’ quality.

The strategy that has been chosen to counteract this risk is to terminate the risk. The risk of a frequently breaching of the Mfolozi berm when the farmers want to has too much impact on the Back Channel. This is because the farmers are in favour of an open Mfolozi mouth during the whole year. By proposing a law that prevents the farmers from breaching below +1.5m GMSL the risk could be terminated. However, the residual risk that the farmers could illegally artificially breach the berm anyhow remains.

The fifteenth risk is defined as:
Because of political influences during decision making and conflicts about required solution occur, there is a chance that the construction of the Back Channel could be postponed, which might happen, will lead to a lack of fresh water inflow into St Lucia estuary and an increase of salinity levels in St Lucia Lakes and compromises the projects’ schedule.

The strategy that has been chosen to counteract this risk is to treat the risk. Political debates could take a lot of time and could hamper the process. The decision making could speed up by putting a lot of pressure on the need for speed in the design process. On the other hand,
political influences cannot be fully prevented. Also by putting a lot of pressure on an actor the relationship between the actors could become sensitive for conflicts.

The sixteenth risk is defined as:
Because of a massive flood that occurs once in the 25 years, there is a chance that the whole area gets flushed away, which might happen, will lead to the change of the complete system dynamics and the combination of the Mfolozi and St Lucia mouth and compromises the projects’ scope and quality.

The strategy that has been chosen to counteract this risk is to take the risk. A contingency plan could be made in order to have a plan when something goes wrong. The weather dynamics cannot be influenced. A one in a twenty-five year flood, like the Demoina flood in the 1980s, will destroy anything on its path.

The finally the seventeenth risk is defined as:
Because of conflicts and mismanagement in the iSimangaliso Wetland Park in the past, there is a chance that the park authority does not want to intervene in the natural situation, which might happen, will lead to the decision not to implement the new Back Channel, a decrease of speed in the current design process, that small floods cannot be kept into the system, scouring of the control notch, higher water levels in the Mfolozi, and more drainage problems near the sugarcane yields and thus compromises the projects’ quality and scope.

The strategy that has been chosen to counteract this risk is to treat the risk. By putting a lot of pressure on the need of urgency of the project the risk could be counteracted. However, the decision remains at the iSimangaliso Wetland Park authority. Actors’ perceptions could change and would still have a negative impact on the need of urgency. Even by putting a lot of pressure on the authority the relationship could be harmed.
H. MONITORING PLAN

A monitoring plan has been set up to monitor the behaviour of the Back Channel system. Monitoring of the Back Channel is required to get a better understanding of this system. Data acquired by monitoring can be used in decision making or in the improvement of models. This plan consists of descriptions of the needed monitoring equipment (H.1), the lay-out of the monitoring system (H.2), how to maintain the system (H.3) and a decision flow chart for daily management (H.4).

H.1 Monitoring equipment

Important data which can be acquired by monitoring are water levels, rate of flows, turbidity, sediment, load and water depth. Different types of monitoring equipment can be applied to monitor these parameters. In the sections H.1.1 till H.1.5 some of these devices will be described.

H.1.1 Water level gauges

Several types of gauges exist to measure the water level. The simplest gauge is an analogue gauge fixed on a stable structure or ground. To get a reading from such a gauge, someone will have to go to this gauge and make a reading of the water level. By using different gauges along the channel a water level gradient can be calculated. A more sophisticated digital gauge can store the data on a memory stick or a hard drive. Occasionally someone has to go to the gauge and download the data to a computer. The time step between the readings of a digital gauge compared with the analogue gauge is much longer which is an advantage. The best gauge which can be used is a digital online gauge which measures the water level and sends it directly to a computer nearby. If an irregularity in the flow, like a flood peak, occurs the manager of the system is informed directly. If necessary, measures can be taken instantly.

H.1.2 Flow meters

To improve the reliability of the flow measurements in situ flow measuring devices like a Swoffer flow/velocity meter or an acoustic Doppler meter (H1.3) can be used. The Swoffer flow/velocity meter features a prop which is fixed on a beam as shown in Figure H-1. The sensor, a small propeller, can be moved up and down along the beam to an exact percentage of the depth. This allows one to measure the flow or velocity at certain depth ratios. The measured data are saved on the hard drive of the flow/velocity meter or is shown on its display. When using a more expensive Swoffer flow/velocity meter the flow is calculated directly after a measurement. When using a less expensive Swoffer flow/velocity meter, the data has to be processed after measuring. A major drawback of this method is that someone must go in to the water. Measurements cannot be done if dangerous wildlife is present.

Figure H-1 Swoffer flow velocity meter (http://www.swoffer.com , 2011)
Another method for measuring the flow is using the gauges. To establish this, the water levels in the channel and weir have to be calibrated with the flow in the river/channel. With the use of the SOBEK model, described in section I.4, Figure H-2 has been made. This figure shows the relationship between the water level at the Mfolozi mouth and the discharge through the old and new Back Channel.

This graph is an estimate of the total flow rate through the channels. Once build, the outcomes of this graph will have to be checked for different water levels. The SOBEK model can be calibrated with the measured data. When properly calibrated, the model can be used to link the water level measured by a gauge at the Mfolozi to the total discharge into St Lucia.

**H.1.3 Depth measurements**

It is important to measure the channel depth on a regular basis. Based on measured depths a cross sectional area can be derived which can be used to calculate the flows. Measuring the depth can be done with a measuring pole. When no wildlife is present near the channel someone has to enter the water and take some readings of predefined cross sections. A more sophisticated device is the acoustic Doppler-meter which has also been used during the fieldtrip (B.4.1). A Doppler-meter measures the flow velocity and depth while it is moved perpendicular to the stream. The on-board computer calculates the flow through the channel.

**H.1.4 Turbidity and salinity measuring equipment**

Turbidity and salinity can be measured with an in-situ hand held multi-parameter water quality meter (Figure H-3). Next to turbidity and salinity, a multi-parameter water quality meter can measure several other parameters like pH, pH(mV), depth and dissolved oxygen. All parameters are measured by submerging a sensor probe in the water. The turbidity is measured in Nephelometric Turbidity Units (NTU). The salinity is generally measured in parts per thousand (ppt) or Pracitical Salinity Unit (PSU).
H.1.5 Measuring of TSS

Measuring of Total Suspended Solids (TSS) is done by taking water samples of flowing water at a depth of 0.6 to 0.8 times the water depth. An obtained sample is poured through a pre-weighed glass fibre filter with a pore size smaller than the smallest suspended particles. The filter is weighed again after drying. The difference in weight of the filter before and after the test gives a dry weight measure of the solids that were suspended. The amount of solids is generally expressed as the weight of suspended solids per volume (mg/L). Before drying, the filter and sample have to be washed with deionized water to get rid of the salts. In general, TSS measurements correlate with the turbidity as described in H2.2. This correlation is site specific. When enough data points are collected and a satisfactory correlation is found the TSS can be derived from turbidity measurements. To avoid bed load in a TSS sample, samples have to be taken at relatively calm spots, typically in straight sections without curved streamlines. In bends or in accelerating currents bed load sediment can get temporarily suspended, disturbing the TSS measurement.

H.2 Monitoring system lay-out

Devices like a gauge, flow meters, and depth meters have to be used at specific points along the Back Channel system. The depth and the velocity meters must be used at the same reach for calculating the flow. The turbidity meter and the TSS have to be used at set locations.

H.2.1 Flow meters and gauges

Gauges will have to be installed at four locations. These locations are:

- at the upper boundary of the Back Channel;
- at the constriction of the old Back Channel;
- on top of the weir at the new Back Channel, and
- at the upper boundary of the new Back Channel.

The locations are displayed in Figure H-4. The gauge near the mouth measures the water level behind the beach berm. This data is necessary to determine the height of the water behind the berm. Furthermore, gauges have to be installed in the old and new Back Channel to measure the water level. Based on the reading of the gauge, discharges can be calculated. Below a calibration has been executed for the gauge at the upper boundary of the Back Channel and the old and the new Back Channel. These calibrations give an indication of the formulas that are applicable on the Back Channel system. Once the gauges are installed the...
formulas have to be verified to ensure that they are correct. Because of the dynamic behaviour of the environment, the gauges have to be recalibrated every now and then. Especially shortly after construction. The biggest amount of settlement will occur in this period. When the weir settles, the distribution of the discharges between the old and the new Back Channel will change. Because of the settlements, the new Back Channel will discharge water at a lower water level than +0.9m GMSL. It is wise to recalibrate the gauge every month after construction. When the rate of settlement slows down, the time steps between recalibrations may become large. To calibrate the gauges, the discharge and water levels at the upper boundary of the Back Channel, the new Back Channel and the old Back Channel should be measured.

![Gauges at the Back Channel](Google; AfriGIS Ltd, 2011)

The existing W2H032 gauge along the Mfolozi river can be used to measure the upstream water level and flow. It is cheaper to use this one, instead of installing a new gauge. The flow measured with this gauge can be used to make decisions on breaching the berm. If the rate of flow is too high, the water cannot be stored behind the berm and the system has to be breached or will breach naturally. Exact numbers of this rate of flow are given in the following subsection.

All levels of the described gauges are relative to GMSL.

**Gauge at the upper boundary of the Back Channel**

The gauge at the upper boundary of the Back Channel can be used to measure the flow through the total system. It is possible to calculate the total discharge from the water level at the upstream boundary of the Back Channel. The formula to be used is the weir formula shown in section G.2. Because the width is not yet known, the calibration constant, c, and the width of the weir, b, will be combined in one unknown coefficient (\(c_{\text{total}}\)) which has to be determined by calibration. Also, the power has to be determined by calibration. The weir formula is:

\[
Q = c \cdot b \cdot H^{3/2}
\]  

[1]

In which:

- \(c\) = coefficient of discharge [m\(^{1/2}\)/s]
- \(b\) = width of the weir [m]
- \(H\) = water depth above the weir [m]
This can be rewritten in a general formula applicable for calibration. This general discharge formula is:

$$Q = c_{\text{total}} H^x$$  \[2\]

In which:

- $c_{\text{total}}$ = calibration coefficient [m\(^3\)/s]
- $x$ = calibration coefficient [-]
- $H$ = waterdepth above the constriction [m]

Total discharges have been calculated with SOBEK and are given in the second column of Table H-1. By chancing $c_{\text{total}}$ and $x$ in formula [2], the calibrated discharge has been calculated. The resulting calibration coefficients are:

- $c_{\text{total}} = 26 \text{ m}^{1.62}/\text{s}$
- $x = 1.38$

The formula for the gauge located at the upstream boundary of the Back Channel becomes:

$$Q = 26 \times H^{1.38}$$  \[3\]

In which:

- $H$ = measured water level at the upstream boundary - 0.9 m

The value 0.9 meter in formula [3] originates from the height above GMSL of the weir crest and the estimated height of the constriction in the old Back Channel.

<table>
<thead>
<tr>
<th>Water level at the upstream boundary of the Back Channel above GMSL (m)</th>
<th>Modeled total Discharge (m(^3)/s)</th>
<th>Calibrated discharge (m(^3)/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0,9</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>1</td>
<td>1,1</td>
<td>1,1</td>
</tr>
<tr>
<td>1,1</td>
<td>2,8</td>
<td>2,8</td>
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<tr>
<td>1,2</td>
<td>4,9</td>
<td>4,9</td>
</tr>
<tr>
<td>1,3</td>
<td>7,3</td>
<td>7,3</td>
</tr>
<tr>
<td>1,4</td>
<td>10</td>
<td>10,0</td>
</tr>
<tr>
<td>1,5</td>
<td>12,9</td>
<td>12,8</td>
</tr>
<tr>
<td>1,6</td>
<td>15,9</td>
<td>15,9</td>
</tr>
<tr>
<td>1,7</td>
<td>19,3</td>
<td>19,1</td>
</tr>
<tr>
<td>1,8</td>
<td>22,8</td>
<td>22,5</td>
</tr>
</tbody>
</table>

Gauges located halfway old and new Back Channel

The gauges located halfway old and the new Back Channel can be used as an extra/back-up system for measuring the flow. First the calibration of the gauge halfway of the old Back Channel will be described; secondly the calibration of the gauge halfway the new Back Channel will be described.
The gauge at the old Back Channel will be placed at the constriction of the old Back Channel. The precise location of this constriction is yet unknown; this has to be sorted out during the installation of the gauge. The discharges through the old Back Channel have been calculated with SOBEK. The results for different water levels at the upstream boundary are shown in Table H-2. For calibrating the gauge formula [2] will be used. By finding the best values of $c_{total}$ and $x$ in formula [2], the calibrated discharge has been calculated. The resulting calibration coefficients are:

$$c_{total} = 6.2 \text{ m}^{1.06}/\text{s}$$

$$x = 1.94$$

The formula for the gauge located at the old Back Channel becomes:

$$Q = 6.2 * H^{1.94}$$

In which:

$H$ = measured water level at the upstream boundary -0.9m

<table>
<thead>
<tr>
<th>Water level at the upstream boundary of the Back Channel above GMSL (m)</th>
<th>Water level at the constriction above GMSL (m)</th>
<th>Modelled discharge old Back Channel (m$^3$/s)</th>
<th>Calibrated discharge (m$^3$/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0,9</td>
<td>0,9</td>
<td>0</td>
<td>0,0</td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>0,1</td>
<td>0,1</td>
</tr>
<tr>
<td>1,1</td>
<td>1,1</td>
<td>0,2</td>
<td>0,3</td>
</tr>
<tr>
<td>1,2</td>
<td>1,18</td>
<td>0,5</td>
<td>0,5</td>
</tr>
<tr>
<td>1,3</td>
<td>1,28</td>
<td>0,9</td>
<td>0,9</td>
</tr>
<tr>
<td>1,4</td>
<td>1,36</td>
<td>1,4</td>
<td>1,4</td>
</tr>
<tr>
<td>1,5</td>
<td>1,45</td>
<td>2</td>
<td>1,9</td>
</tr>
<tr>
<td>1,6</td>
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<td>2,6</td>
</tr>
<tr>
<td>1,7</td>
<td>1,63</td>
<td>3,4</td>
<td>3,4</td>
</tr>
<tr>
<td>1,8</td>
<td>1,72</td>
<td>4,2</td>
<td>4,2</td>
</tr>
</tbody>
</table>

The discharges through the new Back Channel have been calculated with SOBEK. Based on the output a discharge formula like [2] can be calibrated. However this gauge can be placed on two different places along the channel. The first place is at the inflow point of the channel near the main Back Channel. Based on the reading of the water level above 0 m GMSL a discharge can be calculated. The second option is a gauge on top of the weir. The gauge then measures the water depth on top of the weir which can be related to the discharge over the weir.

When choosing for both the options one can check the flows more precisely. If one gauge is off, it is clear that something is broken or the weir has settled. In this case it is time for new recalibration of the gauges.

First the gauge on top of the weir will be calibrated. The calculated water depth on top of the weir and the discharges are shown in Table H-3. Because the width of the weir is known a formula like formula [1] will be used. This formula is:

$$Q = c_{c} \cdot b \cdot d^{x}$$
In which:
\[ c_c = \text{calibrated discharge coefficient [m}^{(2-x)/s}] \]
\[ b = \text{width of the weir [m]} \]
\[ d = \text{water depth on the weir [m]} \]
\[ x = \text{calibration coefficient [-]} \]

The resulting calibration coefficients are:

\[ c_c = 7.2 \text{ m}^{0.45}/s \]
\[ b = 10 \text{ m} \]
\[ x = 1.55 \]

The formula for the gauge located at the crest of the weir at the new Back Channel becomes:

\[ Q = 7.2 * 10 * d^{1.55} \] \[6\]

<table>
<thead>
<tr>
<th>Water level at the upstream boundary of the Back Channel above GMSL (m)</th>
<th>Water depth on the weir (m)</th>
<th>Modelled discharge new Back Channel (m³/s)</th>
<th>Calibrated discharge (m³/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.9</td>
<td>0</td>
<td>0</td>
<td>0</td>
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<tr>
<td>1</td>
<td>0.064</td>
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<td>1.1</td>
<td>0.118</td>
<td>2.6</td>
<td>2.6</td>
</tr>
<tr>
<td>1.2</td>
<td>0.167</td>
<td>4.4</td>
<td>4.5</td>
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<tr>
<td>1.3</td>
<td>0.212</td>
<td>6.4</td>
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<tr>
<td>1.4</td>
<td>0.255</td>
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<td>1.5</td>
<td>0.296</td>
<td>10.9</td>
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<tr>
<td>1.6</td>
<td>0.338</td>
<td>13.3</td>
<td>13.3</td>
</tr>
<tr>
<td>1.7</td>
<td>0.378</td>
<td>15.9</td>
<td>15.9</td>
</tr>
<tr>
<td>1.8</td>
<td>0.418</td>
<td>18.6</td>
<td>18.5</td>
</tr>
</tbody>
</table>

Secondly the gauge on at the inflow of the channel will be calibrated. The calculated water depth at the upper boundary of the new Back Channel is shown in Table H-4. Because the width of the upstream cross section is not precisely known formula [2] will be used. The following values have been obtained during calibration.

\[ c_{total} = 21.3 \text{ m}^{1.69}/s \]
\[ x = 1.31 \]

The formula for the gauge located at the inflow point of the new Back Channel becomes:

\[ Q = 21.3 * H^{1.31} \] \[7\]

In which:
\[ H = \text{measured water level at the upstream boundary - 0.9 m} \]

The value 0.9 meter in formula [7] originates from the height above GMSL of the weir crest and the estimated height of the constriction in the old Back Channel.
Table H-4 Calibration data of the gauge at the upper boundary of the new Back Channel

<table>
<thead>
<tr>
<th>Water level at the upstream boundary of the Back Channel above GMSL (m)</th>
<th>Water at the upper boundary of the new Back Channel (m)</th>
<th>Modelled discharge new Back Channel (m³/s)</th>
<th>Calibrated discharge (m³/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0,9</td>
<td>0,9</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1,0</td>
</tr>
<tr>
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<td>1,09</td>
<td>2,6</td>
<td>2,6</td>
</tr>
<tr>
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<td>13,3</td>
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<td>1,7</td>
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<td>15,9</td>
</tr>
<tr>
<td>1,8</td>
<td>1,72</td>
<td>18,6</td>
<td>18,6</td>
</tr>
</tbody>
</table>
H.2.2 Turbidity and TSS measurements
The turbidity and TSS measurements have to be executed every week in the first year. By doing this, 52 data points for every site will be obtained. With this data a site specific relation can be found between TSS and turbidity. When this relation is obtained a turbidity measurement is sufficient to obtain both the turbidity and the TSS.

The turbidity and TSS have to be measured at four places to get an insight in the turbidity and TSS development along the Mfolozi. These locations are located at the bridge of the N2 highway crossing the Mfolozi, at the natural mouth of the Mfolozi, at the old Back Channel and at the new Channel. With this data the manager of the system can make decisions about how to operate and manage both Back Channels.

![Figure H-5 Locations for TSS and turbidity measurements (Google; AfriGIS Ltd, 2011)](image)

H.3 Maintenance
Regular maintenance has to be performed on the structure to guarantee that the structure can cope with the next high water and would not fail. In order to do, the depth of possible scour holes at the inlet and outlet structure, the amount of sandbags and the state of the wooden structure have to be checked.

When scour holes are getting to deep they must be filled with sandbags to prevent the scour hole from getting deeper. When a scour hole gets too deep, a sandbag next to it will slip down into the hole as shown in Figure H-6-B. When the bottom protection is not repaired the scour hole will erode even more. After a while the whole structure may become unstable and will slip down into the hole as shown in Figure H-6-C. To prevent collapsing of the structure the depth of the channel on the upstream and downstream side of the weir must be measured after a high water event. When the hole is deeper than the height of two or three sandbags and sandbags start to slide down into the hole new sandbags have to be placed in the hole to counteract the scour and erosion.

![Figure H-6 Instability of the bottom protection; A: scour hole at downstream end. B: Erosion of the bottom protection. C: Collapse of the structure (Schiereck)](image)
Also the sandbag covers at the narrowing part of the new Back Channel in front of the weir and at the stilling basin have to be monitored. When sandbags are removed due to high flows and turbulence, new sandbags have to be placed. This must be done to be sure that the bottom protection can withstand a following high water event.

Besides sandbags and scour holes also the state of the wooden weir should be checked regularly. If some planks on the weir have disappeared after a high water, these have to be reinstalled. The weir also has to be checked every 2 months on the state of the wood in the structure. Due to the aggressive salty/brackish environment in which it is located, wood will decay. Varying water levels contribute even more to the decay of the wood. When the wood has rotten too much, these beams or planks have to be replaced to guarantee the strength of the weir.
H.4 Decision flow chart (monitoring and maintenance)

When a flood peak is measured upstream of the Mfolozi, the system manager has to make a decision about how to respond to it. Three factors are important for his decision namely, the water level at the floodplains, the measured peak flow of the flood peak and the state of the berm. If the berm is open nothing will happen. The flood will pass the berm without leading to damage to the sugarcane farmlands. But if the berm is closed water levels will rise at the floodplains. If the volume of the flood is bigger than the storage capacity, flooding of the lower sugarcane farms could occur. To prevent this, the storage capacity must be bigger than the volume of the flood. With use of the hypsometric curve (Appendix ) and the modelled triangular flood peaks (Appendix B.3) several runs of the SOBEK model have been made to check whether the storage capacity is bigger than the volume of the flood to be stored. The output of the SOBEK calculations is presented in Error! Reference source not found.. The maximum flood peak which can be stored depends on the water level at the floodplains. So if a flood peak has been measured upstream, the manager should check the water level at the floodplains and the maximum flow of the flood peak. Based on these values the manager is able to make the decision whether to breach or not.

Figure H-7 Flow decision chart for reaching
I. MODEL RESULTS

I.1 Water balance model
This appendix contains the results of the water balance model used in Appendix Error! Reference source not found.. The numbers of the figures match with the numbers in Table F-9 of the aforementioned appendix. The flood peaks used for the modelling are also added. Lastly, figures have been added displaying the flow rate at different water levels in the channels.
Null variant and improved Back Channel

Figure I-1 Null variant and improved Back Channel, Peak 99 m³/s, Berm height 2.00 m + GMSL

Figure I-2 Null variant and improved Back Channel, Peak 121 m³/s, Berm height 1.50 m + GMSL
Figure I-3 Null variant and improved Back Channel, Peak 121 m$^3$/s, Berm height 2.00 m + GMSL

Figure I-4 Null variant and improved Back Channel, Peak 138 m$^3$/s, Berm height 2.00 m + GMSL
Figure I-5 Null variant and improved Back Channel, Peak 183 $m^3/s$, Berm height 2.00 m + GMSL

Figure I-6 Null variant and improved Back Channel, Peak 223 $m^3/s$, Berm height 2.00 m + GMSL
Figure I-7 Null variant and improved Back Channel, Peak 565 m³/s, Berm height 2.00 m + GMSL
Beach Channel

Figure I-8 Beach Channel, Peak 99 m³/s, Berm height 2.00 m + GMSL

Figure I-9 Beach Channel, Peak 121 m³/s, Berm height 1.50 m + GMSL
Figure I-10 Beach Channel, Peak 121 m³/s, Berm height 2.00 m + GMSL

Figure I-11 Beach Channel, Peak 138 m³/s, Berm height 2.00 m + GMSL
Figure I-12 Beach Channel, Peak 183 m$^3$/s, Berm height 2.00 m + GMSL

Figure I-13 Beach Channel, Peak 223 m$^3$/s, Berm height 2.00 m + GMSL
Figure I-14 Beach Channel, Peak 565 m³/s, Berm height 2.00 m + GMSL
Link Channel

Figure I-15 Link Channel, Peak 99 m$^3$/s, Berm height 2.00 m + GMSL

Figure I-16 Link Channel, Peak 121 m$^3$/s, Berm height 1.50 m + GMSL
Figure I-17 Link Channel, Peak 121 m$^3$/s, Berm height 2.00 m + GMSL

Figure I-18 Link Channel, Peak 138 m$^3$/s, Berm height 2.00 m + GMSL
Figure I-19 Link Channel, Peak 183 m³/s, Berm height 2.00 m + GMSL

Figure I-20 Link Channel, Peak 223 m³/s, Berm height 2.00 m + GMSL
Figure I-21 Link Channel, Peak 565 m³/s, Berm height 2.00 m + GMSL
Old Main Channel

Figure I-22 Old Main Channel, Peak 99 m$^3$/s, Berm height 2.00 m + GMSL

Figure I-23 Old Main Channel, Peak 121 m$^3$/s, Berm height 1.50 m + GMSL
Figure I-24 Old Main Channel, Peak 121 m³/s, Berm height 2.00 m + GMSL

Figure I-25 Old Main Channel, Peak 138 m³/s, Berm height 2.00 m + GMSL
Figure I-26 Old Main Channel, Peak 183 m$^3$/s, Berm height 2.00 m + GMSL

Figure I-27 Old Main Channel, Peak 223 m$^3$/s, Berm height 2.00 m + GMSL
Figure I-28 Old Main Channel, Peak 565 m$^3$/s, Berm height 2.00 m + GMSL
Flood peak graphs

Figure I-29 Flood peak of 99 m³/s (Apr. 2006)

Figure I-30 Flood peak of 121 m³/s (Nov. 2005)
Figure I-31 Flood peak of 138 m$^3$/s (Feb. 2009)

Figure I-32 Flood peak of 183 m$^3$/s (Okt. 2007)
Figure I-33 Flood peak of 223 m$^3$/s (Feb. 2002)

Figure I-34 Flood peak of 565 m$^3$/s (May 1995)
Flow rate plotted against water levels in the channels

Figure I-35 Null variant and Back Channel

Figure I-36 Beach Channel
Figure I-37 Link Channel

Figure I-38 Old Main Channel
I.3 Sediment trap efficiency model

An objection against reconnecting the Mfolozi with the St Lucia estuary with combined mouths is the large amount of fine sediment which is expected to be transported by the flow of water into the St Lucia estuary. Coarse sediment settles before reaching the estuary and does not cause problems. Due to the low flow conditions or even the no-flow conditions in St Lucia fine and very fine sediment will only settle in the Narrows causing silting up of the region. An indirect connection of the Mfolozi with the St Lucia estuary via the Back Channel will lower the amount of sediment flowing into St Lucia. The fine and very fine sediment will settle under normal flow conditions in the retention basin behind the berm when the Mfolozi mouth is closed. In the following subsections the sediment trap efficiency of the Mfolozi estuary is determined with the empirical relationships of Brune (1953) and Churchill (1948). Not much is known on the exact flow conditions within the Mfolozi estuary. The results of both calculations are discussed in section I.3.3.

I.3.1 Reservoir sediment trap efficiency

The performance of a reservoir in trapping sediment without spilling it to lower regions can be expressed with the reservoir sediment trap efficiency. The sediment trap efficiency is depending on a number of factors. According to Brune (1953) these factors are among others:

- the age of the reservoir;
- the shape of the reservoir;
- the detention time of the water within the reservoir;
- the velocity of the water within the reservoir;
- the ratio between inflow and capacity;
- the gradation of the transported sediment;
- the behaviour of finer sediments under varying conditions, and
- the type of outlets and methods of operation.

The empirical methods of Churchill (1948) and Brune (1953) are discussed below. These methods are used to compute sediment trap efficiency for annual runoff and peak runoff.

Churchill (1948)

Over time, several attempts have been made at correlating one or a combination of the factors listed in the introduction this paragraph with the trap efficiency of a reservoir. Churchill (1948) linked the period of retention with the velocity of the water within the reservoir and correlated this with the trap efficiency. He developed a “sedimentation index”, which is the period of retention divided by the velocity. Churchill (1948) plotted a curve through a set of “sedimentation index” points retrieved from several reservoirs of the Tennessee Valley Authority (TVA) (Figure I-39). Brune (1953) states that although the curve seems to be fairly accurate, it cannot readily be used for other reservoirs because for most reservoirs data on retention time and flow velocity is not available.

Brune (1953)

Brune (1953) related the ratio of capacity over annual inflow (C/I-ratio) with the trapping efficiency of a reservoir. To do so, he plotted the trapping efficiency of 44 reservoirs against the C/I ratio. Of these 44 reservoirs, 40 were normal ponded reservoirs, 2 were semi-dry reservoirs (not containing water all the time) and 2 were desilting basins. Dredging, venting and sluicing were taken into account for all basins. Brune (1953) concluded that the use of the C/I-ratio gives a much better index of trapping efficiency than the use of the ratio of the capacity over the watershed (C/W-ratio), which was widely used before. He also concluded that although the C/I-ratio of reservoirs may indicate that the trapping efficiency might be low,
it can be much higher because of an trap efficient shape of the reservoir, for example in desilting basins.

---

**Figure I-39** Percent of incoming silt passing through reservoir as related to sedimentation index, TVA reservoirs, (Churchill, 1948) in (Brune, 1953)

---

**Figure I-40** Trap efficiency as related to capacity-watershed ratio, (Brune, 1953)

---

**Annual runoff**

Murthy (1980) developed techniques which allows one to make a comparison between the methods of Churchill (1948) and Brune (1953) (Figure I-42). According to Yang (2006) the general guideline is that one should use the Brune (1953) method for large storage or normal ponded reservoirs. The method of Churchill (1948) should be used for small reservoirs, settling basins, semi dry reservoirs, continually sluiced reservoirs or flood retarding structures. No guidelines are given to subdivide a certain reservoir in one of these groups. For this reason both methods will be used for calculating the trap efficiency, although the Mfolozi estuary will probably be a small reservoir. According to Huizinga & Van Niekerk (2005) the mean annual runoff of the Mfolozi is $920 \cdot 10^6$ m$^3$. Using the DEM of Chrystal & Stretch the
storage of the Mfolozi estuary is $12.03 \times 10^6$ m$^3$ for a berm height of 2.0 m. The C/I-ratio then becomes 0.013 which leads to a trap efficiency of approximately 52% (Figure I-42).

Figure I-41 Trap efficiency as related to capacity-inflow ratio, type of reservoir and method of operation, (Brune, 1953)

\[
K = SI \times \frac{g}{(\text{gravitational acceleration})}
\]

Figure I-42 Trap efficiency curves, (Churchill, 1948) and (Brune, 1953)

**Peak runoff**

The methods of Churchill (1948) and Brune (1953) are meant for reservoirs in front of dams or power plants and do not take regular breaching into account. Therefore the trap efficiency will also be checked for peak runoffs. To do so, the volume of several peak runoff datasets will be calculated assuming a triangular peak hydrograph as described in section B.3. The berm height is set at 2.0 m, giving a maximum retention basin capacity of $12.03 \times 10^6$ m$^3$. For this calculation, the $T_c$ (time between start of the flood and the flood peak) has been set at 15 hours. A flood peak has a total duration of $3 \cdot T_c = 45$ hours. The maximum volume which can be retained without breaching the berm (starting at a water level of 1.20 m + GMSL) is $9.8 \times 10^6$ m$^3$. This corresponds with a flood peak of 121 m$^3$/s. Also some higher peaks have
been added to make a good comparison. Results are given in Table I-1. What stands forward from these results is the high trapping efficiencies for all calculated peak flow rates, much higher than for the annual runoff (52%). Another remark is the fact that there is no trap efficiency of 100%. This is due to the always present re-suspension of sediment in a normal pond. The results of Table I-1 will be discussed in section I.3.3.

Table I-1 Trapping efficiency for several peak flow rates

<table>
<thead>
<tr>
<th>Peak flow rate (m³/s)</th>
<th>50</th>
<th>75</th>
<th>100</th>
<th>125</th>
<th>150</th>
<th>175</th>
<th>200</th>
<th>300</th>
</tr>
</thead>
<tbody>
<tr>
<td>Volume (-10⁶ m³)</td>
<td>4.05</td>
<td>6.08</td>
<td>8.10</td>
<td>10.13</td>
<td>12.15</td>
<td>14.75</td>
<td>16.20</td>
<td>24.30</td>
</tr>
<tr>
<td>C/I-ratio (-)</td>
<td>2.97</td>
<td>1.98</td>
<td>1.49</td>
<td>1.19</td>
<td>1.00</td>
<td>0.82</td>
<td>0.74</td>
<td>0.50</td>
</tr>
<tr>
<td>Trapping efficiency (%) (Churchill, 1948)</td>
<td>92</td>
<td>91</td>
<td>90</td>
<td>89</td>
<td>88</td>
<td>87</td>
<td>86</td>
<td>85</td>
</tr>
<tr>
<td>Trapping efficiency (%) (Brune, 1953)</td>
<td>98</td>
<td>98</td>
<td>98</td>
<td>98</td>
<td>98</td>
<td>98</td>
<td>98</td>
<td>97</td>
</tr>
</tbody>
</table>

I.3.2 Modelling plug flow

A simple representation of the flow through the Mfolozi estuary is by modelling it as a plug flow. This means that water enters a schematized basin from one side and flows evenly divided over space towards the other end of the basin. The sediment, which enters the basin together with the water, follows this flow and at the same time slowly settles. The setup, assumptions and the result of the model are discussed in the following subsections. It should be noticed that the results of this model only give a very rough estimate of what the sediment trap efficiency can be. The real flow conditions in the retention basin will not follow the plug flow conditions.

Model set-up

For this model, the Mfolozi estuary has been modelled as rectangular basin with a water depth $h$, a length $L$ and a width $W$. The flow through the basin can be modelled with a horizontal or a vertical gradient. It is assumed that the sediment concentration is uniform over the depth when it enters the basin. This sediment can settle as long as the water, with which it entered, is within the basin. The time the water is within the basin is called the retention time of the basin and it assumes there is no short circuiting of water through the basin. Another assumption is that the inflow is the same as the outflow. This means the volume of the basin does not change over time. It is further assumed that during the retention time the settling is not hindered by turbulence.

Horizontal gradient

When the flow is modelled with a horizontal gradient, this means the water enters the basin at the left side of Figure I-43 (assuming it is evenly spread over the depth of the basin), flows towards the right and exits the basin at that side. The retention time of the water can be calculated with the following formula:

$$T_r = \frac{V}{Q} = \frac{LW h}{Q}$$

[1]
In which:

\[ T_r = \text{retention time [s]} \]
\[ V = \text{volume of the basin \([m^3]\)} \]
\[ Q = \text{flow rate through the basin \([m^3/s]\)} \]
\[ L = \text{length of the basin \([m]\)} \]
\[ W = \text{width of the basin \([m]\)} \]
\[ h = \text{water level within the basin \([m]\)} \]

In order to provide sediment free water near the exit of the basin all sediment must settle in the basin before the water leaves the basin. This means the time required to settle the particles present at the water column, \( T_s \), should be shorter than the retention time of the basin, \( T_r \). In terms of formulas:

\[
T_r > T_s \rightarrow \frac{LW h}{Q} > \frac{h}{w_s} \rightarrow \frac{w_s LW}{Q} > 1 \rightarrow \frac{w_s A}{Q} > 1
\]  \[2\]

In which:

\[ A = \text{area of the basin \([m^2] \) (L-W)} \]

From formula [2] it follows that whether the basin is large enough for all the sediment to settle is independent of the water depth and only depends on the area. This is only the case for theoretical situations in which the basin is box-shaped. In reality the area of the Mfolozi basin is depth dependent. For the calculations in this section the area has been set at a single value.

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**HORIZONTAL FLOW**

*Figure I-43 Plug flow - Horizontal gradient*

**Vertical gradient**

When the flow is modelled with a vertical gradient water entering the basin at the lowest point of the basin of Figure I-44 flows towards the top and exits the basin. When the water enters it is assumed that it is evenly spread over the bottom. For the sediment to settle its settling velocity should be larger than the vertical flow velocity of the water. In terms of formulas:

\[
w_s > \frac{Q}{LW} \rightarrow \frac{Q}{A} \rightarrow \frac{w_s A}{Q} > 1
\]  \[3\]

When the inequality of [3] is compared with the inequality of [2], it can be seen there is no difference in modelling the flow with a horizontal or a vertical gradient.

For calculations the area of the estuary has been set at a water level of 1.75 m + GMSL, which is the average of the berm height used in the design (2 m + GMSL) and the berm height at which it nowadays will be breached under request of the sugar cane farmers (1.5 m + GMSL). With this assumption it is possible to calculate which settling velocity and corresponding median diameter is required needed for total settlement of sediment. The flow rate \( Q \) must be interpreted as a flow rate which is constant over the retention time of the water at that exact flow rate \( T_r = V/Q \) in which \( V \) is the basin volume at a water level of 1.75 m + GMSL). Calculations have been performed with Matlab.
Sediment composition

The sediment transport by the Mfolozi is composed of bed-load, suspended load and wash-load. The bed-load and suspended sediment are mainly composed of granular, non-cohesive materials ($\rho_s = 2650$ kg/m$^3$) and are suspected to settle fully when entering the Mfolozi estuary under normal flow conditions. The wash-load is composed of fine to very fine silt and clayey material. The settling velocities of these particles are very small and they will stay in suspension for a long time. Research performed by Maine (2011) shows that most of the fine and very fine sediments flocculate when they enter the Mfolozi estuary because of the higher salinity levels there. Tidal action and waves will cause water from the sea to get in the estuary during low berm levels. The sea water will heighten the salinity levels. When the berm grows over time less seawater will enter the estuary. Dilution of the estuary water with river runoff will lower the salinity levels, though salinity levels are suspected to stay high enough for the fine and very fine sediment to effectively flocculate. The effective average specific gravity of the flocs formed in the estuary is 1200 kg/m$^3$. The median floc diameter is in the order of 8 to 10 microns or 0.008 to 0.01 mm (Maine, 2011). These data have been used as input for the model.

Settling velocity

When the minimal required settling velocity is calculated with the inequalities of [2] and [3] the corresponding particle diameter can be calculated with a settling velocity formula. The settling velocity formulas of Van Rijn (1993) and Soulsby (1997) are widely used but are both mainly applicable for granular sediment ($\rho_s=2650$ kg/m$^3$). To be able to use these formulas a crude assumption is made: flocs, with a specific gravity of 1200 kg/m$^3$, will settle on the same manner as granular sediment. Hindered settlement, which is normally observed for muddy flocs (Winterwerp, 2002), will also be ignored. The settling velocity for natural granular sediment is expressed by Van Rijn (1993) as:

$$w_s = \frac{\Delta g D_{50}^2}{18 \nu}$$

for $1 < D_{50} \leq 100 \mu m$ [4a]

$$w_s = \frac{10 \nu}{D_{50}} \left( \sqrt{1 + \frac{0.01 \Delta g D_{50}^2}{\nu^2}} - 1 \right)$$

for $100 < D_{50} \leq 1000 \mu m$ [4b]

$$w_s = 0.92 \sqrt{\Delta g D_{50}}$$

for $D_{50} > 1000 \mu m$ [4c]

In which:

- $\rho_w$ = specific gravity of water (1000 kg/m$^3$)
- $\rho_s$ = specific gravity of sediment (sand: 2650 kg/m$^3$, flocs: 1200 kg/m$^3$)
- $\Delta$ = submerged specific gravity of grains ($(\rho_s - \rho) / \rho$) [-]
- $g$ = gravitational acceleration (9.81 m/s$^2$)
- $D_{50}$ = median nominal particle diameter [m]
- $\nu$ = kinematic viscosity ($10^{-6}$ m$^2$/s)

The formula developed by Soulsby (1997) is especially meant for sediments in a marine environment and reads:
\[ D_x = \left( \frac{g \Delta \rho}{\rho g} \right)^{1/3} D_{50} \]  \hspace{1cm} [5a]  
\[ w_x = \frac{\nu}{D_{50}} \left( \sqrt{10.36^2 + 1.049 D_x^2} - 10.36 \right) \]  \hspace{1cm} [5b]

In Figure I-45 both formulas are compared for both granular material (left) with a specific gravity of 2650 kg/m\(^3\) and flocs (right) with a specific gravity of 1200 kg/m\(^3\). For small particle diameters there is almost no significant difference between the two formulas. Taking into account the crudeness of the model and the fact Soulsby (1997) uses in principle only one formula, formula [5] has been implemented in the Matlab model.

Figure I-45 Comparison of settling velocity formulas for granular materials (left) and flocs (right)

Results

The minimal required settling velocities are calculated for sand and flocs with the aid of inequality [2] and [3]. The calculated settling velocities have been plotted against the flow rate Q. The same result is found for both granular materials and flocs.

Figure I-46 Minimal required settling velocity plotted against the flow rate for granular materials (left) and flocs (right)

By calculating (formula [5]) the minimal required particle diameter from the settling velocities the differences can be seen (Figure I-47). The minimal required (effective) diameters are
smaller for grains than for flocs due to the larger specific gravity of granular non-cohesive particles.

![Graph showing minimal required median nominal (effective) diameter against the flow rate for granular materials (left) and flocs (right)](image)

Figure I-47 Minimal required median nominal (effective) diameter against the flow rate for granular materials (left) and flocs (right)

In the left graph of Figure I-47 it can be seen that the assumption on the settling efficiency of granular material is plausible. According to the American Geophysical Union silt (finest granular non-cohesive material) has nominal diameters between 0.004 and 0.062 mm. This range lies above the minimal required nominal diameter in the left graph of Figure I-47. This means the settling efficiency is 100% for flow rates between 0 and 200 m$^3$/s. According to Maine (2011) the average effective floc diameters lie within the range of 0.008 to 0.010 mm. This means that the settling efficiency will be less than 100% for flow rates above 110 m$^3$/s, which is far more than normal flow rates (in the order of 0-20 m$^3$/s) (Grenfell & Ellery, Hydrology, sediment transport dynamics and geomorphology of a variable flow river: the Mfolozi River, South Africa, 2009).

I.3.3 Discussion

**Empirical methods**

Both empirical methods used in the calculations do not mention what kind of sediment the relationship is meant for. The rivers and reservoirs used in the development of the methods are suspected to have contained fine sediments. Therefore the results of the methods can be used as an indication of the trap efficiency of the Mfolozi estuary.

When the results of the empirical methods for annual and peak runoff are compared it becomes clear that the trap efficiency for peak runoffs is far larger than that for annual runoff. This is because the methods of Brune (1953) and Churchill (1948) are originally only meant for annual runoff. When water flows through the old Back Channel under normal flow conditions in the Mfolozi the water entering St Lucia is virtually sediment-free (Taylor, The St Lucia-Mfolozi connection: A historical perspective, 2011b). The results obtained for lower peak runoffs (<75 m$^3$/s) could therefore be fairly accurate. During large flood events, the amount of sediment transported by the Mfolozi is far larger (Grenfell & Ellery, Hydrology, sediment transport dynamics and geomorphology of a variable flow river: the Mfolozi River, South Africa, 2009). Under these circumstances the amount of sediment flowing through the Back Channel and entering St Lucia is large. It is suspected that during peak runoffs the water short circuits through the Mfolozi basin. During a relative short period a large mass of water with a high momentum enters the basin: the retention time is much lower than calculated with the assumption of plug flow.
The calculated trapping efficiency for annual runoff is approximately 50%. This is, taking into account the peak runoffs, a reasonable estimate. It is recommended that this result should be checked by continuous measurements, see appendix G.11.

**Modelling plug flow**

Before discussing the results it is important to mention the following large assumptions have been made in order to get results:

- Inflow is equal to the outflow: surface area is constant;
- No turbulence, no re-suspension and no hindered settling;
- Constant salinity levels: constant forming of flocs;
- No difference between the settling formulas of Van Rijn (1993) and Soulsby (1997) for fine sediments;
- No short circuiting of water through the basin;
- Box-shaped basin, and
- Surface area set at that of 1.75 m + GMSL.

Taking into account the limited knowledge on the flow patterns through the basin, modelling the flow as plug flow is justified at this stage. There is too little knowledge to develop a sophisticated at this moment.

Results show all flocs will settle when the inflow of the basin is lower than 110 m$^3$/s, which seems to be too large. Though the inflow is not constant for a long period as is assumed in the model and the outflow is also not that large. So a large part of flocs will settle but certainly not all of it due to turbulence, short circuiting, re-suspension etc.

It is questionable whether the surface area of the basin at a water level of 1.75 m + GMSL represents the average effective area. Until a water level of 1.5 m + GMSL the surface area stays approximately constant. The hydrograph shows that as soon as the water level rises above 1.5 m + GMSL, areas start to flood. Due to flooding the surface area of the basin becomes larger. These flooded areas are very shallow, even when the water level reaches 2.0 m + GMSL. The flow in the shallow areas will be small. It is therefore better to use the surface area at a water level of 1.50 m + GMSL in the plug flow model. When the water level is lowered to 1.50 m + GMSL the required (effective) nominal particle diameters become larger (Figure I-48). The flow rate at which 100 % trap efficiency occurs is now lowered to a flow rate of 50 m$^3$/s, which seems more reasonable. Taking into account ability of the basin to store such a runoff peak this seems even more probable (G.2). One should keep in mind a trap efficiency of 100 % is virtually impossible in a system like that of the Mfolozi and St Lucia. Also during very low discharges of the Mfolozi (mouth closed) turbulence and resuspension will bring sediment into St Lucia. Such a trap efficiency could only be reached in the hypothetical case of an extremely deep or wide basin.
To improve the model one could add the influence of the changing inflow and outflow over time. The model should be divided in several timesteps and the change of the effective area per time step should be calculated. The outflow of the basin changes with the water depth, so in this case the retention time is depending on the water depth. During a time step the inflow and outflow are assumed to be constant. This kind of modelling is the same as a water balance model. A further improvement of the model would be to add stratification of the flow through the basin and the corresponding turbulence. This way of modelling goes beyond the scope of this project and will not be further elaborated.

**Methods compared**

From both models it can be concluded that not all sediment will be retained within the basin during flood peaks. During these peak runoffs a lot of sediment is transported, much more than during normal flow periods. Before the Mfolozi mouth breaches a certain amount of sediment will flow into St Lucia due to short circuiting of water through the Mfolozi basin. To get a good insight in the amount of sediment flowing into the Mfolozi estuary and into St Lucia an extensive monitoring plan should be set up. This should include a continuous measurement of the water and sediment inflow. With this data correct statements on the sediment trap efficiency of the system can be made. The data from these measurements could also be used to further develop the plug flow model. It is also advised to do research on the flow patterns of the water through the Mfolozi basin during normal and peak runoff situations. When enough data is gathered, it is recommended to develop a 3D flow model of the Mfolozi estuary to get a good insight on the behaviour of the flows through the estuary.
I.4 SOBEK model

Building the model is done in several phases. Each phase is as small as possible in order to ease the calibration after each phase of the model. In this way the real world is modelled as good as possible. In the first phase, described in I.4.3, the original single Back Channel has been modelled. The main channel has been modelled based on cross sections measured by M.Sc. student Clinton Chrystal. With use of measured water levels upstream and a flow rate downstream a calibration has been performed. After calibration a storage node was added in the model. This storage node represents the topography of the low lying area at the Mfolozi mouth. When the water level rises, the flooded area increases and more water can be stored. This is described in I.4.4. The third phase is adding the existing new Back Channel dug on the 10th of September 2011. With use of measured flow and water levels during a field trip the extended model has been calibrated again. A flood peak came down the river during the field trip. This flood peak has been modelled as an inflow at the upper boundary of the model. This has been done in phase four, described in I.4.6. After the fourth phase, the basic discharge calculations, done in G.2 have been checked with the model in I.4.7. Several model runs with different configurations of weir width and channel width have been performed. Based on the basic discharge formula’s and the model output a decision has been made about the dimensions of the new channel and weir.

I.4.1 The 1D flow model SOBEK

To model the Back Channel system the 1D flow model SOBEK is used. The hydraulic research organization Deltares located in Delft allowed us to use a version of SOBEK during the project. This program works with the Saint Venant Equations including transient flow phenomena and backwater profiles. All kind of cross sections can be modelled in the program, even closed cross sections like pipelines and sewer systems. In these cross sections a distinction between parts of the cross section can be made with different roughness elements. In this way it is possible to model a main channel with a low roughness and a floodplain covered with trees with a high roughness. The program can also work with drying and flooding. This means that it is assumed that the segment is dry if the water level is lower than 5mm above the bed level at the upstream edge of a reach segment. The discharge through the channel is then equal to zero. When the water level is rising again, the water has to be 10mm higher than the bed level before water will flow through the channel. This also counts for structures like weirs. When the upstream water level is too low, the weir becomes dry and the discharge is set to zero. The program is able to model all kind of structures like pumps, weirs, gates, culverts, sluices, and bridges. It is even possible to partly block a culvert when it is blocked by sediments or the area between bridge piers in case trees are stuck in between. The program is also able to calculate with steep channels and supercritical flow and even moving hydraulic jumps. The program can deal with them as easy as canals with mild slopes and subcritical flow. The program is also able to perform an unsteady calculation. Furthermore the program can calculate sediment transport capacity and it can be shown in the network. For running the model upper and lower boundary conditions must be set. These boundary conditions can be modelled as a steady water level, a steady discharge, a function of time of the water level, a function of time of the discharge and a water level versus discharge function over time.

The hydrodynamics can be calculated in different modules; the SOBEK-Rural 1DFLOW module, SOBEK-Urban 1DFLOW, and SOBEK-River 1DFLOW. The SOBEK-Rural 1DFLOW is designed for the simulation of one-dimensional flow in irrigation and drainage systems and small scale canal/river systems. It can be used for solving problems for regional water management like irrigation construction, drainage, canal systems, dredging and flood protection. The SOBEK-Urban 1DFLOW is designed for the simulation of one dimensional flow in wastewater and storm water systems. With use of this module a drainage and sewer
system can be designed. SOBEK-River 1DFLOW is a module used for one-dimensional water flow in river systems and estuaries. It can be used for solving problems like flood protection, flood-risk assessment, real-time forecasting, dam break analysis, navigation and dredging. These modules can be used as a stand-alone version but they can also be used in combination with Urban and River. In the weeks of preparation Deltares was visited to consult Erik Mosselman about the capability of SOBEK and how to use it in our design. He advised to use the Rural module because the model is a small scale canal/river system with major links to drainage systems.

I.4.2 GIS maps
Schematizing of the environment in SOBEK-Rural requires a geographical background map as under layer. A 1:2000 GIS map of the St Lucia & Mfolozi area has been obtained from the geosystem and land surveying department of the UKZN. In addition Google Earth data has been transformed from WGS 84 projection to Universal Transverse Mercator Projection using GDAL (GDAL, 2011). The transformation was required due to import limitations in SOBEK. The maps available (WGS 84) were scaled with the unit decimal degree, and SOBEK can only work with the unit meters. First the channels were drawn in SOBEK on the decimal degrees map, but during running the model errors turned up. In order to model the system the GIS data and the WGS 84 projection of Google Earth were converted into a metric system. Both maps are used to provide an accurate under layer (Figure I-49) on which the model is schematized.

![Figure I-49 Basic under layer imported in SOBEK (Google earth)](image)

I.4.3 Phase 1
The first phase consists of modelling and calibrating the existing Back Channel and natural weir. The channel is modelled in SOBEK as a 1D flow channel with unsteady flow. During this phase the inflow is only dependent of a fixed water level near the inflow point of the Back Channel. The water level in St Lucia estuary is fixed at 0 m GMSL and does not influence the upstream water level.

The cross section of the Back Channel is based on calibrated differential GPS height graphs as measured by Chrystal (2009). Chrystal measured several points along a straight line in order to get accurate cross sections. The data is the most accurate available and can be
considered as reasonable accurate. The yellow marked dots represent the cross sections shown in in Figure I-50.

Figure I-50 A few of the cross sections and their location as used in SOBEK

The model is set up with respect to the Geodetic Mean Sea Level (GMSL). In order to prevent outflow of water near the edges of the cross section the height of the embankment of each channel has been set at +3m GMSL. In a 1D flow model outflow at places other than boundary nodes results in a negative volume balance. In addition the fixed height of the embankments eases the interpolation process between the defined cross sections.

The goal of this model run is to calibrate the existing Back Channel. After calibrating the model should represent the existing situation. Once the channel is calibrated it can be expanded with the new Back Channel and hypsometric curve during phase two and three.
Calibrating phase one

Calibration of the SOBEK model is very important. Without proper calibration results obtained by the modelling might be totally different compared to the real situation. Calibration can be done by using in-situ measured data like inflow and water levels as input. The results obtained by the model should be equal to the real values.

The dimensions of the channel are fixed during calibration. The only parameter altered during the calibration process is the Manning roughness parameter of the channel and mangrove forest. Although dimensions might not be totally accurate any errors due to wrong dimensions should be forestalled by the Manning parameter.

In this particular case only one datasets is available to calibrate the model. This dataset contains the water level at the UCOSP Coticane gauge (Gerrit de Jager, 2011) and the outflow of the old Back Channel as measured by Ricky Taylor (2011). Input data for the calibration process must be extracted from the datasets of Gerrit de Jager and Ricky Tailor mentioned in paragraph B.2. Only the days on which the outflow of the old Back Channel was estimated can be used as input. Table I-2 lists the available calibration data. A reliable dataset has to be extracted from this data.
Table I-2 Available data for calibration (Gerrit de Jager, 2011)

<table>
<thead>
<tr>
<th>Date</th>
<th>Water level Cotcane Station (msl)</th>
<th>Rate of increase (mm/day)</th>
<th>Weekly Water Level Increase (mm)</th>
<th>Cotcane Drain Level (m³/s)</th>
<th>Umfolozi Flow Rate (m³/s)</th>
<th>Flow into Lake (mm)</th>
<th>Rain Monzi (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3-jun</td>
<td>0,964</td>
<td>10</td>
<td>5,961</td>
<td>1,14</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>11-jun</td>
<td>1,03</td>
<td>17</td>
<td>3,727</td>
<td>2,09</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>14-jun</td>
<td>1,057</td>
<td>9</td>
<td>5,638</td>
<td>1,99</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>16-jun</td>
<td>1,104</td>
<td>6</td>
<td>0,583</td>
<td>7,174</td>
<td>2,63</td>
<td></td>
<td></td>
</tr>
<tr>
<td>25-jun</td>
<td>1,054</td>
<td>-1</td>
<td>5,137</td>
<td>2,09</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2-jul</td>
<td>1,025</td>
<td>-4</td>
<td>4,765</td>
<td>1,67</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9-jul</td>
<td>1,046</td>
<td></td>
<td>2,56</td>
<td>2,04</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>16-jul</td>
<td>1,091</td>
<td>69</td>
<td>2,075</td>
<td>4,8</td>
<td>29</td>
<td></td>
<td></td>
</tr>
<tr>
<td>18-jul</td>
<td>1,21</td>
<td>26</td>
<td>1,033</td>
<td>2,544</td>
<td>5,5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>22-jul</td>
<td>1,18</td>
<td></td>
<td>1,033</td>
<td></td>
<td>4,5</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The UCOSP Cotcane gauge is located upstream the Msunduze near the sugarcane plantations as shown in Figure I-51. The gauge is located in an area where tidal influences are still present when the berm is breached. The water level is measured every 12 minutes. Only the data obtained during periods featuring a closed berm and stable water levels have been used.

![Figure I-51 Overview of measurement stations (Kelbe & Taylor, 2011)](image)

During periods of low runoff and a closed berm the influence of a backwater curve will be very limited. The water level near the Cotcane gauge is a good approximation for the water level near the entrance of the Back Channel. Some considerations must be taken into account:

- Water levels measured by the Cotcane gauge prior to the 26th of September 2011 should be corrected by +0.2 meter in order to obtain levels in GMSL. (surveyed in September 2011)
- The gauge is calibrated to MSL, however the degree of error is unknown.
Flow measurements done by Taylor are estimates and can vary 25% in both directions; they have been measured by hand and with simple methods.

The times at which the Cotcane values were taken do not correspond with the exact times at which the outflows were measured. Both values have been obtained somewhere on the same day leaving some room for error. In order to reduce this error only the values obtained on days with no or a little rate of increase have been used. The data measured on the 16\textsuperscript{th} and the 18\textsuperscript{th} of July do not agree with this statement. The measurements of the 11\textsuperscript{th} and the 14\textsuperscript{th} are of the same order as the measurement made on the 25\textsuperscript{th} of June but contain a larger error. They have not been used for calibration. The measurements made on the 3\textsuperscript{rd} and the 16\textsuperscript{th} of June differ the most compared to the 25\textsuperscript{th} of June whilst the rate of increase is quite low. These values have also been used. Table I-3 lists the dataset which is used for calibration.

<table>
<thead>
<tr>
<th>Date</th>
<th>Corrected water level [m]</th>
<th>Measured flow [m}^3/s]</th>
<th>Modelled flow [m}^3/s]</th>
<th>Difference %</th>
</tr>
</thead>
<tbody>
<tr>
<td>3-jun</td>
<td>1,164</td>
<td>1,14</td>
<td>1,24</td>
<td>8</td>
</tr>
<tr>
<td>16-jun</td>
<td>1,304</td>
<td>2,63</td>
<td>2,75</td>
<td>4</td>
</tr>
<tr>
<td>25-jun</td>
<td>1,254</td>
<td>2,09</td>
<td>2,2</td>
<td>5</td>
</tr>
</tbody>
</table>

Table I-4 contains the results returned by the model after calibrating the Manning roughness parameters. Water level data measured before the 26\textsuperscript{th} of September 2011 has been corrected by +0.2 meter. The maximum difference between the modelled flow and the measured flow is 8%. The calibrated Manning factors are listed in Table I-4.

<table>
<thead>
<tr>
<th>Object</th>
<th>Manning coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>Main channel</td>
<td>0.05</td>
</tr>
<tr>
<td>Floodplains</td>
<td>0.06</td>
</tr>
<tr>
<td>Mangrove forest</td>
<td>0.06</td>
</tr>
</tbody>
</table>

Model sensitivity analysis
A sensitivity analysis is used to study which influence the variation of a parameter has on the output of the model. It is a way to estimate the influence on the model result if a variable deviates from its expected value. In order to investigate this the most important parameters used in the model have been altered by -20%, -10%, 10%, and 20%. The model has been run numerous times whilst changing one parameter at a time. The deviation of the original
outflow with respect to the outflows modeled with changed parameters has been computed and plotted in Table I-6 to Table I-10. The water level at the upstream boundary was assumed to be constant at 1.2m + GMSL.

**Sensitivity of Manning Roughness in main channel**

A lower roughness of the main channel leads to the largest deviation as compared with the initial value. This is in accordance with the Manning formula. An example of its nonlinear curve is shown in Figure I-52. According to Manning the biggest deviations occur at a lower value of \( n \) and the smaller deviations occur at a higher value of \( n \). The results are presented in Table I-6.

### Table I-6 sensitivity of the roughness main channel

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Input data ([s\cdot m^{-1/3}])</th>
<th>Input difference</th>
<th>Model output ([m^3/s])</th>
<th>Difference percentage ([m^3/s])</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roughness headchannel</td>
<td>0.050</td>
<td>0</td>
<td>1.57</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>0.040</td>
<td>-20</td>
<td>1.67</td>
<td>6.4</td>
</tr>
<tr>
<td></td>
<td>0.045</td>
<td>-10</td>
<td>1.62</td>
<td>3.2</td>
</tr>
<tr>
<td></td>
<td>0.055</td>
<td>10</td>
<td>1.55</td>
<td>-1.3</td>
</tr>
<tr>
<td></td>
<td>0.060</td>
<td>20</td>
<td>1.51</td>
<td>-3.8</td>
</tr>
</tbody>
</table>

\[ Q = \frac{1}{n} \cdot A \cdot R^{2/3} \cdot S^{1/2} \]  

In which:

- \( Q \) = Discharge \([m^3/s]\);
- \( A \) = Area of the channel \([m^2]\);
- \( R \) = Hydraulic radius \([m]\);
- \( S \) = Bed slope \([\cdot]\);
- \( n \) = Manning coefficient \([s/m^{1/3}]\).

![Figure I-52 Manning versus Flow (based on A=30m^2 R=3m and S=0.0005)](image)

**Sensitivity of the roughness at the natural control point**

Changing the roughness of the mangrove forests has a larger impact on the results than changing the roughness of the head channel. The larger influence can be explained by the formula of Manning [6]. Deviations are larger for low \( n \) values than for high \( n \) values. The reason for the larger sensibility is the difference in cross section \( A \) and the radius \( R \) between the cross section of the forest channel and the main channel (Figure I-53 and Figure I-54). The area, the wet perimeter and the radius of the channel cross section are:
- \( A = 150 \text{m}^2 \)
- \( L = 70 \text{m} \)
- \( R = 2.1 \text{m} \)

The values of the forest cross section are:
- \( A = 30 \text{m}^2 \)
- \( L = 14 \text{m} \)
- \( R = 2.1 \text{m} \)

If these values are substituted in the formula of Manning and one deletes the \( S \) and the \( n \) from it, because they have the same values for both sections, one gets a ratio between the discharge through the forest and the discharge through the main channel as shown in Table I-8. From this table it can be concluded that the rate of flow in the head channel is 5 times higher than the flow in the forest channel. The ratio between the differences in Table I-6 Table I-7 is around 1:3.2 (original values are 6.4:17.8), which is in line with the ratio of Table I-8.

### Table I-7 Sensibility of the roughness of the forest

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Input data ([s*m^{-1/3}])</th>
<th>Input difference</th>
<th>Model output ([\text{m}^3/\text{s}])</th>
<th>Difference percentage ([\text{m}^3/\text{s}])</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roughness forest</td>
<td>0.060</td>
<td>0</td>
<td>1.57</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>0.048</td>
<td>-20</td>
<td>1.85</td>
<td>17.8</td>
</tr>
<tr>
<td></td>
<td>0.054</td>
<td>-10</td>
<td>1.7</td>
<td>8.3</td>
</tr>
<tr>
<td></td>
<td>0.066</td>
<td>10</td>
<td>1.47</td>
<td>-6.4</td>
</tr>
<tr>
<td></td>
<td>0.072</td>
<td>20</td>
<td>1.38</td>
<td>-12.1</td>
</tr>
</tbody>
</table>

### Table I-8 ratio forest channel; main channel

- Factor forest channel: 49,86357
- Factor main channel: 249,3179

![Figure I-53 Cross section channel](image-url)
The hydraulic roughness of the mangrove forest lined embankments does not affect the output in the given test case. In this sensitivity test water flows mainly through the central channel because the back channel does not function at its full capacity. When the channel functions at its maximum capacity the roughness of the mangrove forest will influence the results. The influence of the mangrove forest is equal to the influence near the control point. The results are given in Table I-9.

Table I-9 sensibility of the roughness mangroves

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Input data [s*m^-1/3]</th>
<th>Input difference</th>
<th>Model output [m^3/s]</th>
<th>Difference percentage [m^3/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roughness mangroves</td>
<td>0.060</td>
<td>0</td>
<td>1.5774</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>0.048</td>
<td>-20</td>
<td>1.5774</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>0.054</td>
<td>-10</td>
<td>1.5774</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>0.066</td>
<td>10</td>
<td>1.5774</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>0.072</td>
<td>20</td>
<td>1.5774</td>
<td>0</td>
</tr>
</tbody>
</table>

Sensitivity of bottom level of mangrove forest

The bottom level of the mangrove forest can be compared with a natural weir. The height of this “weir” determines the flow capacity through the forest. Results are given in Table I-10. Changing the bottom height results in a larger deviation of the output. Compared with Table I-6, Table I-7 and Table I-9 the bottom height appears to be the most sensitive parameter. Flow responds nonlinear to the head level over a weir [7]. If the hydraulic head over the natural weir lowers by 20% the outflow will double. To compare a standard broad crested weir with a weir in SOBEK formula [7] has been used. The results of the calculation are listed in Table I-11. The model output is almost equal to the values following from the broad crested weir formula. The model is very sensitive to changes in bed level height but follows the theorem of broad crested weirs.

\[
Q = c \times b \times H^{2/3} \quad [7]
\]

\[
c = 1.7 \, m^{1/2}/s
\]
Table I-10 Sensitivity of the bottom height forest

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Input data [m]</th>
<th>Input difference</th>
<th>Model output [m³/s]</th>
<th>Difference percentage [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>bottom height forest</td>
<td>0.900</td>
<td>0</td>
<td>1.57</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>0.720</td>
<td>-20</td>
<td>2.99</td>
<td>90.4</td>
</tr>
<tr>
<td></td>
<td>0.810</td>
<td>-10</td>
<td>2.26</td>
<td>43.9</td>
</tr>
<tr>
<td></td>
<td>0.990</td>
<td>10</td>
<td>0.8</td>
<td>-49.0</td>
</tr>
<tr>
<td></td>
<td>1.080</td>
<td>20</td>
<td>0.24</td>
<td>-84.7</td>
</tr>
</tbody>
</table>

Table I-11 Rate of flow per running meter calculated with the formula for broad crested weirs

<table>
<thead>
<tr>
<th>Input data [m]</th>
<th>c [m^0.5/s]</th>
<th>H [m] (measured in SOBEK)</th>
<th>q [m²/s]</th>
<th>difference [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.9</td>
<td>1.7</td>
<td>0.29</td>
<td>0.3</td>
<td>n/a</td>
</tr>
<tr>
<td>0.72</td>
<td>1.7</td>
<td>0.45</td>
<td>0.5</td>
<td>93</td>
</tr>
</tbody>
</table>

I.4.4 Phase 2

During the second phase the model used in the first phase is expanded with the hypsometric curve as proposed by Chrystal & Stretch (2009). A storage node containing the volume curve as seen in section B.6 is situated at the start of the Back Channel. This storage node schematizes the retention basin behind the sand berm. The storage node and the hypsometric curve are shown in Figure I-55. Breaching of the berm is not yet schematized. As soon as the design breach level is reached the storage node will overflow and the calculations will come to a hold. Both the dimensions of the channel and the roughness factors are equal to those used in paragraph I.4.3. This model will be used to calculate throughput of the channel and weir during various flow conditions. It also acts as a verification of the design height of the weir as proposed in section G.2.3.

The output of the model has been verified with the Cotcane data listed in appendix I.4.3. The output is consistent. The retention basin modelled as storage node works as expected. Water levels are consistent with input and output. In essence this is a water balance model. The system also responds equal to the model in appendix I.1.

I.4.5 Phase 3

During the third modelling phase the New Back Channel is included in the model. Initially the dimensions of the existing New Back Channel are used to obtain a crude estimate of its capacity. Field measurements (paragraph B.4) are used to calibrate the existing New Back Channel. Height profiles have been obtained by handheld differential GPS (Ashtech, 2011) and Real Time Kinetic GPS (Trimble). The base station was located at the KZN Ezemvelo
Office benchmark. Heights have been measured within a 2000 meter range around the base station. The RTK station was used to measure the water levels of the Mfolozi, St Lucia Estuary and Back Channel. In addition the area around the New Back Channel was surveyed. Errors in GPS data have been corrected, Figure I-56 contains the corrected water levels.

Figure I-56 Corrected water levels

Flow velocities measured with an acoustic Doppler (section B.4.1) and by hand were used to check the model. Hand measurements were performed by throwing a stick in the water and measuring the time it took for the stick to travel 15 meters. Ricky Taylor has used this method frequently and the data obtained are listed in appendix Error! Reference source not found.. The measured velocity during our fieldtrip using the “stick method” was 0.35m/s. The accuracy of those measurements is debatable but the data is frequently used in many reports. A. Knox from Stemele-Bosch compared the “stick method” data with more accurate data. He concluded the stick method is accurate enough for basic calculations. The acoustic Doppler tests returned an average flow velocity of 0.3m/s. The velocity profiles are given in paragraph B.4.1.

Calibrating phase 3
The new situation is calibrated with the data gathered during the fieldtrip. The old channel is assumed to be unchanged in relation with phase one and two. The corrected water level at the Mfolozi mouth was +1.4m GMSL and the water level at the St Lucia estuary +0.25m GMSL (Figure B-55). Water levels were kept constant during the calibration process. The inflow constriction of the new Back Channel has been modelled as a shallow and narrow channel with dimensions equal to those measured in the field with the acoustic Doppler. The outflow constriction is modelled as a weir. The inflow and outflow are shown in Figure I-57. The length of the inflow constriction is approximately 25 meters equal to the width of the mangrove friendly floodplain of the Back Channel. These 25 meters correspond to the length between the deepest point of the main channel and the embankment of the Back Channel. The height of the constriction is based on an estimate made by Ricky Taylor (2011) and GPS data by C. Chrystal. The outflow is modelled as a weir because it is the control point of the system. The easiest and most accurate way to model a control point is by using a weir. In addition weirs are easier to calibrate as opposed to channel constrictions.
The model setup with the New and Old Back Channel is shown in Figure I-58. The total length of the New Channel is approximately 100 meters. This includes 25 meter of inflow through the mangrove forest, 40 meter of channel and 35 meter of outflow through mangrove.

The obtained results are shown in Figure I-59, Figure I-60 and Figure I-61. The discharge in the New Channel is 1.05m³/s whilst the discharge in the ‘old channel’ is 1.65m³/s. The flow velocity in the New Channel is 0.31m/s according to SOBEK and the water level in the storage node remains stable at +1.398m GMSL. The output values of the model are close to the in-situ measured data listed in Table I-12. Based on this one can conclude the model is well calibrated.

<table>
<thead>
<tr>
<th>Table I-12 Calibration data</th>
<th>Measured</th>
<th>Output</th>
<th>error %</th>
</tr>
</thead>
<tbody>
<tr>
<td>h storage/floodplain (m)</td>
<td>1.4</td>
<td>1.398</td>
<td>0.1</td>
</tr>
<tr>
<td>Q new Back Channel (m³/s)</td>
<td>1.0</td>
<td>1.05</td>
<td>4.8</td>
</tr>
<tr>
<td>Q old Back Channel (m³/s)</td>
<td>1.6</td>
<td>1.65</td>
<td>3.0</td>
</tr>
<tr>
<td>v new Back Channel (m/s)</td>
<td>0.3</td>
<td>0.31</td>
<td>3.2</td>
</tr>
</tbody>
</table>
Figure I-59 Discharge New Channel phase 3 (SOBEK)

Figure I-60 Velocity New Channel phase 3 (SOBEK)
I.4.6 Phase 4

A few days prior to the fieldtrip a small flood wave came down the Mfolozi. Water levels associated with this wave has been logged by the W2H032 gauge (paragraph B.1) located near Monzi. The water levels associated with the wave are listed in Figure I-62 and Figure I-63. A rough model of the discharge wave is manufactured and inserted in SOBEK. Figure I-63 shows the modelled wave. The discharge before the wave was assumed to be constant in order to get rid of initial numerical errors. During 26th of September 2011 till the 4th of October 2011 the inflow of the Mfolozi has been set at 1.5m$^3$/s creating a steady equilibrium (inflow equals outflow). Furthermore it is assumed that the rate of flow will decrease after the 10th of October which corresponds with the water levels on the 14th of October by the Cotcane gauge.

Figure I-62 Small flood wave of the 8th of October 2011 as measured by the W2H032 gauge.
The obtained model results do not agree with measured data on the 11th of October 2011. When using the W2H032 flood wave the water levels increased to a higher level than was measured by the Cotcane gauge. In addition water levels rose faster and high water level stroke earlier than measured. The water level at the Mfolozi mouth was +1.4m GMSL on the 11th of October 2011. The computed water level in the storage node, representing the water level behind the berm, was +1.5m GMSL Figure I-64. A water level of +1.4m GMSL had already arrived on the 8th of October 2011 as shown in Figure I-64.

The discrepancy between the model and reality can be caused by various problems, namely:
The W2H032 gauge measures water levels. Discharge graphs are obtained by calculation requiring the cross section of the channel. During low runoff the cross section of the channel is highly dependent on the water level. A flow of 17 m$^3$/s can be considered low given the discharge range of the Mfolozi and the width of the bed. Due to a bad calibration of the gauge and/or poor estimation of the cross section the actual flow might be lower than the flow returned on the DWAF website.

The hypsometric curve as proposed by Chrystal has not been verified. It is the most accurate hypsometric curve available according to D. Stretch (2011); however discrepancies with the reality might occur. If the origin of the hypsometric curve appears to be higher than in reality, water levels will rise faster in the model. By lowering the origin from 0 m GMSL to a lower point, the storage capacity of the basin at a certain water level will increase. Another kind of inaccuracy of the hypsometric curve is the area-volume relationship. Chrystal used a triangular grid to determine the storage capacity as shown in paragraph B.6. Data points on the floodplains are scarce and small deviations might exist.

Extraction of water by farmers might be another reason. Water which is extracted for agricultural use does trespass the Monzi gauge but never reaches the storage node due to evapotranspiration on the floodplains.

Another possible cause of the discrepancy is the delay between the gauge and the berm. The W2H032 gauge is located 12 km inland on the embankment of the Mfolozi. When the wave has been measured in this point it has to travel towards the berm located at the beach. This delay will be shorter than the 3 days difference but it will be part of the discrepancy.

The determination of the true cause of this discrepancy exist is beyond the scope of this project. This discrepancy is of small influence on the structural design of the weir because threshold levels do not change. In addition the model underestimates the capacity of the retention basin. In reality water levels will be lower than modelled causing fewer problems.

I.4.7 Modelling the new channel and weir

Paragraph G.2 describes the requirements for the weir. The weir has to discharge an average 15 m$^3$/s during a flood peak in order to drain the storage node within the given time frame.

With the weir crest set at +0.9 m GMSL, a highest water level of +1.8 m GMSL and a water level in the St Lucia estuary of +0.25 m GMSL the head level difference over the weir equals 1.55 m. The maximum flow over the weir is 2 m$^2$/s based on Figure G-5.

One of the main functions of the weir is to divert water from the floodplain/storage area into St Lucia. During high runoff, water must be diverted as rapid as possible to prevent flooding. In paragraph G.1 is stated that the duration of flooding of the farmlands must be shorter than five days. To fulfil this requirement the weir should to be wide enough to get enough water into St Lucia. At the same time the weir should be the control point. If the weir is too large the main channel becomes the control point.
The optimal width of the weir is assumed to be 10m. Several SOBEK runs are made to verify a weir with a width of 10m does fulfil the requirements whilst remaining the control notch. Runoff waves with a peak discharge of 50m$^3$/s, 75m$^3$/s and 100m$^3$/s are used in the model. Important output parameters are:

- the water levels in the storage node,
- the duration of flooding,
- the discharge through the new Back Channel, and
- the discharge through the old Back Channel.

During simulation the approach channels remain unchanged. The width of the bottom is 15m, the slope of the embankment is 0.5:1 as surveyed in the field. Results are listed in Table I-13, Table I-14 and Table I-15; the output of the different runs is given in I.4.10. A red shading means fail, orange means near fail, green means passes test.

**Table I-13 Water level on the floodplains**

<table>
<thead>
<tr>
<th>Q\width</th>
<th>5</th>
<th>10</th>
<th>15</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>1,75</td>
<td>1,7</td>
<td>1,7</td>
</tr>
<tr>
<td>75</td>
<td>1,9</td>
<td>1,8</td>
<td>1,8</td>
</tr>
<tr>
<td>100</td>
<td>2</td>
<td>1,9</td>
<td>1,9</td>
</tr>
</tbody>
</table>

**Table I-14 Combined discharge trough the old and new Back Channel**

<table>
<thead>
<tr>
<th>Q\width</th>
<th>5</th>
<th>10</th>
<th>15</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>16</td>
<td>20</td>
<td>22</td>
</tr>
<tr>
<td>75</td>
<td>20</td>
<td>26</td>
<td>28</td>
</tr>
<tr>
<td>100</td>
<td>27</td>
<td>34</td>
<td>37</td>
</tr>
</tbody>
</table>

**Table I-15 Duration of flooding above +1.5 GMSL in hours**

<table>
<thead>
<tr>
<th>Q\width</th>
<th>5</th>
<th>10</th>
<th>15</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>130</td>
<td>90</td>
<td>70</td>
</tr>
<tr>
<td>75</td>
<td>190</td>
<td>130</td>
<td>110</td>
</tr>
<tr>
<td>100</td>
<td>230</td>
<td>160</td>
<td>140</td>
</tr>
</tbody>
</table>

From Table I-13 it can be concluded that a flood peak with a peak flow of 100m$^3$/s is beyond the capacity of the storage node. The water levels in the storage node are getting higher than +1,8m GMSL which is not allowed. This water level will be reached with a peak flow of 100m$^3$/s for a weir width of 5 meters, 10 meters and 15 meters. This also counts for the peak flow of 75m$^3$/s and a width of the weir of 5 meters. For this reason they are marked red, which means they do not fulfil. The duration of flooding is also too long. Growing sugar cane can survive a flooding period of 72 hours and ‘mature’ sugar cane a flooding period of between 5 and 7 days (equal to 120 and 168 hours) (Gerrit de Jager, 2011). Sugar cane may suffer 15% to 20% yield loss after 5 days flooding. When the flood lasts longer than 10 days, the yield loss equals 30% to 60% which is unacceptable (www.bses.org.au, 2011). So the maximum duration of flooding must be approximately 120 hours. Flood periods much longer than 120 hours are not reasonable for the farmers so they are also marked red. If they are just a little bigger they are marked orange, in those cases farmers do have to take countermeasures against flooding.

Now a distinction has to be made between the left orange and green boxes in the Table I-13, Table I-14, and Table I-15. The orange option of 50m$^3$/s and a width of 5 meters is not
suitable because the duration of flooding increases enormously when the floods gets higher. Also the floods which can be dealt met are too small. So this is not an option.

The last distinction to be made is between 10 meters and 15 meters width of the weir. When the weir gets 50% wider, the discharge should also increase with the same percentage. But comparing to Figure I-86 and Figure I-88 the difference is only 3m³/s. The increase is only 15%, much lower than the 50%. From this it can be concluded that the weir is not the constricting point in the system. The main Back Channel is governing. This must be avoided because this leads to unwanted side effects like scour in the main Back Channel. Because of this the weir width will be limited at 10 meters.

The next design parameter is the width of the channel before and after the weir. The velocities in these channels are not allowed to be too large. When they are too big, scour can occur in these channels. The peak discharge of the channel is 20m³/s. A reasonable flow velocity for a stiff clay or alluvial silt at a depth of 1.8 meters is 0.7 m/s as shown in Figure B-65.

In order to make a decision about the width of the approach channel of the weir, several model runs have been made. Bottom widths of 5, 10, 15 and 20m have been modelled

<table>
<thead>
<tr>
<th>Bottom width [m]</th>
<th>Area [m²]</th>
<th>Velocity [m/s]</th>
<th>Q [m³/s]</th>
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<td>10</td>
</tr>
<tr>
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<td>15</td>
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</tr>
<tr>
<td>20</td>
<td>36,5</td>
<td>0,6</td>
<td>22</td>
</tr>
</tbody>
</table>

The approach channel is a constriction in the option with a channel width of 5 meters. So this option is not suitable. When the channel is as width as the weir, the flow velocity is higher than the allowed flow velocity, and lower than the critical flow velocity during short term events. To lower the velocity, the channel has to be enlarged. When the width of the channel is bigger than 15 meters, the velocities are lower than 0.8m/s. So a width of 15 and 20 meters can be applied. The difference in discharge between the 15 meter wide channel and a 20 meter wide channel is only 10%. Based on this small difference in capacity it is reasonable to apply a channel width of 15 meters. The gain of a bigger channel is too low compared to the amount of work for construction.

The outlet channel which will be build is hard to model in SOBEK. For determining the dimensions of this channel a simpler method will be used. Based on the formula $A = Q/\nu$ the width of the outlet channel will be designed. The design of this channel is described in G.8.
I.4.8 Overview of the cross sections used in the SOBEK model

Eleven cross sections are used in the SOBEK model. The blue nodes shown in Figure I-65 represent the different cross sections along the Back Channel. The first two cross sections on the beach are approach channels to the storage node and the Back Channel. The sizes of these cross sections are bigger than the cross sections of the Back Channel. In this way the lake in front of the Back Channel is modelled. The cross sections shown in Figure I-66 till Figure I-75 are the cross sections taken from C. Chrystal's data. The cross sections, shown in Figure I-76 and Figure I-77, represent the mangrove forest with the bottom of the channel on 0.9m above GMSL. The dimensions of this cross section are based on a schematized cross section made by C. Chrystal (unfinished report)

I.4.9 Cross sections

![Figure I-65 Overview of the cross sections](image1)

![Figure I-66 Cross section 1](image2)
Figure I-67 Cross section 2

Figure I-68 Cross section 3

Figure I-69 Cross section 4
Figure I-70 Cross section 5

Figure I-71 Cross section 6

Figure I-72 Cross section 7
Figure I-73 Cross section 8

Figure I-74 Cross section 9

Figure I-75 Cross section 10
Figure I-76 Cross section weir 1

Figure I-77 Cross section weir 2
I.4.10 Model runs for determining the weir dimensions

*Flood wave 50 m³/s, width 5 meters*

**Figure I-78** Modelled discharge through the Back Channels (width new Back Channel = 5m)

**Figure I-79** Water level at the floodplains (width new Back Channel = 5m)
Flood wave 50 m$^3$/s, width 10 meters

Figure I-80 Modelled discharge through the Back Channels (width new Back Channel = 10m)

Figure I-81
Water level at the floodplains (width new Back Channel = 10m)
Flood wave 50 m³/s, width 15 meters

Figure I-82 Modelled discharge through the Back Channels (width new Back Channel = 15m)

Figure I-83
Water level at the floodplains (width new Back Channel = 15m)
Flood wave 75 m$^3$/s, width 5 meters

Figure I-84 Modelled discharge through the Back Channels (width new Back Channel = 5m)

Figure I-85 Water level at the floodplains (width new Back Channel = 5m)
Flood wave 75 m³/s, width 10 meters

Figure I-86 Modelled discharge through the Back Channels (width new Back Channel = 10m)

Figure I-87 Water level at the floodplains (width new Back Channel = 10m)
Flood wave 75 m³/s, width 15 meters

**Figure I-88** D Modelled discharge through the Back Channels (width new Back Channel = 15m)

**Figure I-89** Water level at the floodplains (width new Back Channel = 15m)
Flood wave 100 m$^3$/s, width 5 meters

Figure I-90 Modelled discharge through the Back Channels (width new Back Channel = 5m)

Figure I-91 Water level at the floodplains (width new Back Channel = 5m)
Flood wave 100 m³/s, width 10 meters

Figure I-92 Modelled discharge through the Back Channels (width new Back Channel = 10m)

Figure I-93 Water level at the floodplains (width new Back Channel = 10m)
Flood wave 100 m$^3$/s, width 15 meters

Figure I-94 Modelled discharge through the Back Channels (width new Back Channel = 15m)

Figure I-95 Water level at the floodplains (width new Back Channel = 15m)
1.5 Settlement calculation D-Settlement

Report for D-Settlement 0.2
Settlement Calculations
Developed by Deltres

Company: DELTARES
Date of report: 10/20/2011
Time of report: 3:03:34 PM
Date of calculation: 10/15/2011
Time of calculation: 10:04:59 AM
Filename: C:\\Ct_LucaalModelieren\\D-Settlement berekening\Weir sandbags
Project identification: New Back Channel Mflocz

DELTARES                  D-Settlement 0.2

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   2.7 Verticals 4
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2 Echo of the Input

2.1 Layer Boundaries

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<th>Boundary number</th>
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</tr>
<tr>
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<tr>
<td>1 - Y</td>
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</tr>
<tr>
<td>1 - Y</td>
<td>1.000</td>
</tr>
</tbody>
</table>

2.3 General Data

- Soil model: NEN Sijmum
- Consolidation model: Darcy
- Strain model: Linear
- Groundwater level: Initial determined by PL-line number 1
- Unit weight of water: 9.81 [kN/m³]
- Stress distribution
  - Soil: Busmann
  - Loads: None
- End of consolidation: 10000.00 [days]
- No maintain profile
  - Pₚ (initial): Variable parallel to the initial effective stress
  - Pₑ (per step): Automatic increased to the final effective stresses
- Creep rate reference time: 1.000 [days]
- No imaginary surface
- With submerging (only for non uniform loads)
- Iteration stop criterion: 0.13 [m]
- Load column width: 9.00 [m]
- Non-Uniform Loads: 1.00 [m]
- Trapezoidal Loads: 1.00 [m]

2.4 Soil Profiles

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<th>Material name</th>
<th>PL-line top</th>
<th>PL-line bottom</th>
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</thead>
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<td>Clay, heavily sanded</td>
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<td>0</td>
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</table>

2.5 Soil Properties

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<tr>
<th>Layer number</th>
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<th>Unsaturated W</th>
<th>Saturated W</th>
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<td></td>
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<td>Kg/m³</td>
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<table>
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<th>Permeability strain mod.</th>
<th>Initial vertical permeability</th>
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<tr>
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2.7 Verticals

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</tr>
<tr>
<td>6 - 0</td>
<td>1.000 3.750 7.500 6.400</td>
</tr>
</tbody>
</table>

Calculation of cross section at Z = 0.000 m
Discretisation = 100
3 Settlements

3.1 Settlements

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<tr>
<th>Vertical number</th>
<th>X co-ordinate [m]</th>
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<th>Settlement [m]</th>
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End of Report
J. AUTOCAD DRAWING

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Kelbe, B., & Taylor, R. (2011). **Analyses of the hydrological linkage between Mfolozi/Msunduze Estuary and lake St Lucia.** Gezina.


This report contains the final result of a multidisciplinary research project performed by five students of the Delft University of Technology. After two months of hard and skilled labour a solution is given to divert more fresh water into the St Lucia estuary without major impact on the floodplains. The proposed solution counteracts the different complex and dynamic problems encountered in the iSimangaliso Wetland Park.