Hydrological Assessment for the proposed abstraction of water from the uMhlathuzana River

Prepared for Clover SA (Pty) Ltd

Final Report

GroundTruth

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GroundTruth
PO Box 916
9 Quarry Road, Hilton
KwaZulu-Natal, South Africa
+27 33 343 2229
admin@groundtruth.co.za

www.groundtruth.co.za

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Prepared by	GroundTruth environment & engineering
Author	Keanu Singh
Document Reviewer	Trevor Pike (Pr.Eng)
Prepared For	Clover Way Better™

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- That there are no circumstances that may compromise the objectivity of GroundTruth in performing such work;
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- All the particulars furnished by team members in this document are true and correct; and
- GroundTruth is aware that any person(s) is guilty of an offence in terms of Regulation 48 (1) of the EIA Regulations, 2014, if that person provides incorrect or misleading information. A person who is convicted of an offence in terms of sub-regulation 48(1) (a) to (e) is liable to the penalties as contemplated in section 49B(1) of the National Environmental Management Act, 1998 (Act 107 of 1998).

Signed:

Date: 06/06/2025

Trevor Pike

Pr. Eng: 20130774

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1. INTRODUCTION

GroundTruth Environment and Engineering (GroundTruth) were contracted by Clover SA (Pty) Ltd to undertake a hydrological assessment which included hydrological modelling, a conceptual water balance and a floodline delineation as part of a Water-Use License Application (WULA) for the Clover SA facility located on Remainder of Erf 2999, Queensburgh within the eThekwini Municipality, KwaZulu-Natal Natal. A locality map of the Clover property boundary is shown in Figure 1.1.

Figure 1.1 Locality map of the Clover property boundary (background imagery by Google Earth)

1.1 Objectives

The hydrological assessment utilised GIS spatial data and historical rainfall records to perform hydrological modelling of the subcatchments contributing to the uMhlathuzana River within the Clover property boundary. The objectives of this study, were as follows:

- Evaluate the site's hydrological setting (i.e. climate, rainfall, drainage, etc.).
- Determine the 1:2, 1:5, 1:10, 1:50, and 1:100-year peak flows under pre- and post-development scenarios. The pre-development scenario refers the system prior to the design and installation of the proposed development i.e. an abstraction weir, whilst the post development refers to post construction of the weir.
- Undertake a conceptual water balance to assess the likely alterations to the mass balance of water for the site in light of the proposed abstraction weir.

- Undertake a floodline assessment and associated delineation of the 1:2, 1:5, 1:10, 1:50, and 1:100-year peak flows under both pre- and post-development scenarios.

1.2 Study Relevance to the Season in which it was Undertaken

This study was undertaken as a singular assessment, using historical rainfall, evaporation and existing GIS-based datasets. The assessments in this study are not restricted by seasonal variations, and where necessary, monthly averages were estimated and used.

1.3 Scope of Work

The scope of work completed, was as follows:

- Baseline hydrology review which included hydro-meteorological data collection and analysis, catchment delineation and drainage characteristics, and determination of catchment hydraulic and geometric parameters.
- Design rainfall estimation representative of the study site for the 1:2, 1:5, 1:10, 1:50, and 1:100-year return period events.
- Design flood peak estimation under a pre- and post-development scenario.
- Flood line modelling for the 1:2, 1:5, 1:10, 1:50, and 1:100-year peak flows event using HEC-RAS hydraulic software, and analysis of the modelling results.
- Undertake a conceptual water balance.
- Provide a concise report including generalised recommendations and conclusions.

2. METHODOLOGY

2.1 Hydrological Assessment

Hydrological data for the study area were obtained from various sources including rainfall from the South African Weather Services (SAWS), the South African Water Resources Study WR2012 database (Bailey & Pitman, 2015), and the Daily Rainfall Data Extraction Utility (Lynch, 2004). Datasets from these sources were used to determine the Mean Annual Precipitation (MAP), Mean Annual Runoff (MAR), and Mean Annual Evaporation (MAE) of the study site as well as the design rainfall used to calculate peak flow volumes used to model flood lines.

2.2 Catchment Delineation

Flow accumulation, flow direction, a topographic convergence index raster layer, contour lines and in-situ knowledge of stormwater systems were used to delineate the effective catchment areas for the assessment area. Figure 2.1 below shows the delineated catchment boundaries feeding the system. The focus catchments are located in the Mvoti-uMzimkhulu water management area, in the U60F quaternary catchment. The general area is drained by the uMhlathuzana River and is largely urbanised. Catchment characteristics are shown in Table 2.1.

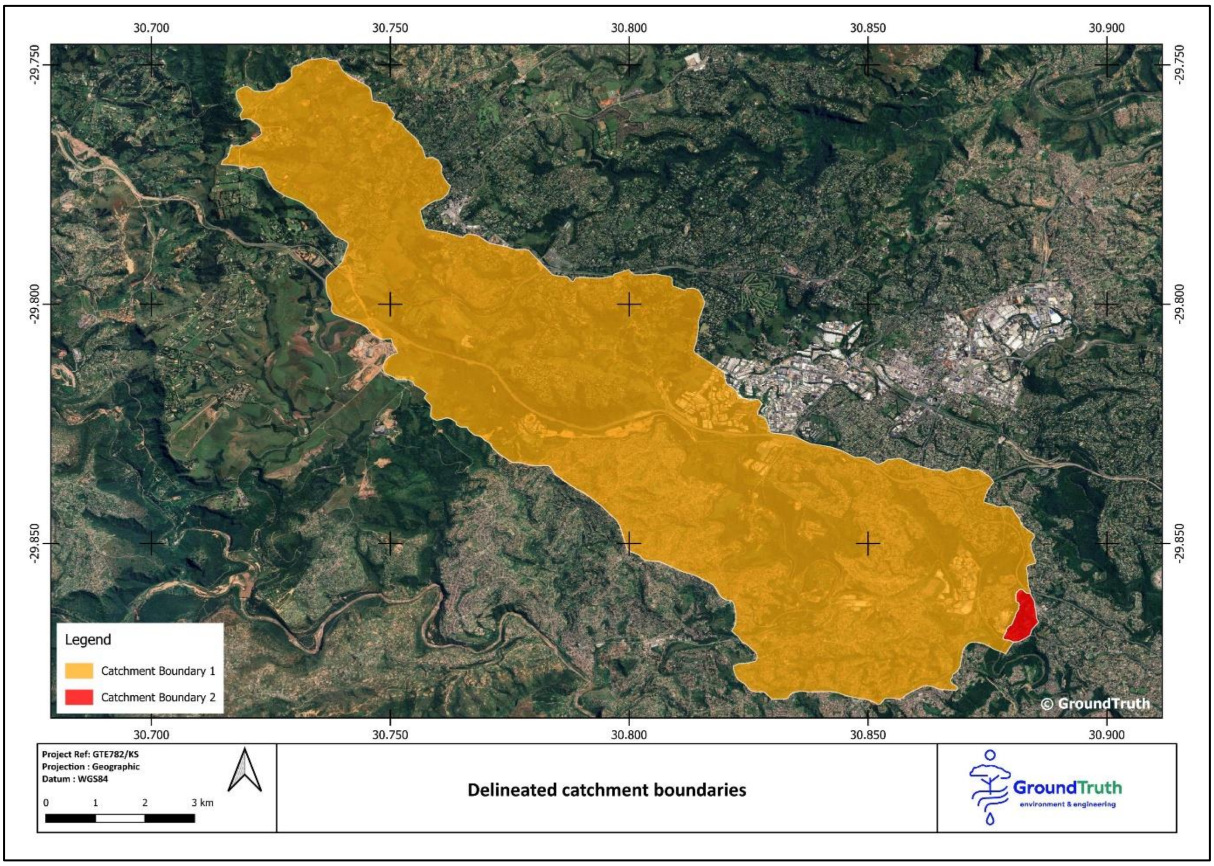

Figure 2.1 Delineated catchment boundaries

Table 2.1 Catchment characteristics for hydrological modelling

Catchment ID	Catchment Area (km²)	Mean Annual Precipitation (mm)	Mean Altitude (m)	Length of longest watercourse (km)	Time of concentration (hr)
1	79.1	786	583	32.9	5.0
2	0.5	1018	95	1.3	0.6

2.3 Design Rainfall Estimation

Mean Annual Precipitation (MAP) and design rainfall depths for the various return periods and durations at each catchment centroid have been estimated using the Regional L-Moment Algorithm and Scale Invariance (RLMA&SI) method (Smithers and Schulze, 2003) as incorporated in the Design Rainfall Estimation tool developed by Smithers and Schulze (2003). The RLMA&SI method is a regional index-flood type approach to frequency analysis (Smithers and Schulze, 2003). The RLMA&SI method was used to estimate short (Smithers and Schulze, 2000a) and long duration (Smithers and Schulze, 2000b) design rainfall depths at different confidence intervals at any location in South Africa (Smithers and Schulze, 2003). A Generalised Extreme Value (GEV) probability distribution was used to analyse 1789 rainfall stations with a minimum of 40 years of rainfall data. Based on Linear-Moments, a regionalisation process was performed and 78 homogenous regions in South Africa was developed. Quantile growth curves for each region were then developed and with index storm values, design rainfall depths at 3946 rainfall stations were estimated.

A gridded system of design rainfall depths over South Africa was further developed by applying regression equations developed by using mean the 1-day Annual Maximum Series of observed rainfall. The gridded system comprises of 429 701 points which form a 1'x 1' of a degree latitude and longitude grid (Smithers and Schulze, 2003). Design rainfall durations for greater than 1 day may be calculated from the product 1 day growth curves and an estimate of the mean of the annual maximum series (index value) for the duration to be assessed (Smithers and Schulze, 2003). A Java based computer program for this method, called Design Rainfall Estimation in South Africa, was developed and may be used to estimate design rainfall for return periods of 2 to 100 years from 5 minutes to 7 days anywhere in South Africa (Smithers and Schulze, 2003).

The following assumptions are present in the study regarding the Design Rainfall Estimate (DRE):

- The design rainfall estimates at the median confidence interval were selected in this study.
- Any uncertainties within the DRE have not been accounted for.

The design rainfall depth and MAP that were extracted for the catchment for the corresponding time of concentration are shown in Table 2.2**.**

Table 2.2 Design rainfall depth (mm) at the catchment centroid for the estimated time of concentration

Catchment ID	Recurrence Interval					
	2-Year	5-Year	10-Year	20-Year	50-Year	100-Year
1	55.03	80.14	101.16	125.37	163.94	199.26
2	28.91	43.37	54.53	66.82	85.22	101.09

2.4 Design floods

The Rational and SCS-SA methods were selected based on the total effective catchment areas to estimate design flood peaks and hydrographs for the delineated catchments under the pre- and post-development scenarios. The SCS-SA and Rational methods are prescribed for use in South Africa by the South African Nation Road Agency SOC Ltd (SANRAL) (2013), and by the Department of Water and Sanitation (Van der Spuy and Rademeyer, 2021). A brief description of these methods is provided below.

2.4.1 *Rational*

The Rational method (RM), developed by Mulvaney (1850), is a rainfall-based design flood peak estimation model. The RM is widely used throughout the world in small rural and urban catchments (Pilgram and Cordery, 1993), and is recommended for use in South Africa (SANRAL, 2013; Van der Spuy and Rademeyer, 2021). Pegram and Parak (2004) mentions that the RM is often used in industry as a “check”. The RM is based on the law of the conservation of mass and is represented in Equation 2.1. The estimated design peak discharge is directly proportional to the effective catchment area, rainfall intensity and a user selected runoff coefficient.

$$Q_p = 0.278\,(C \times I \times A) \qquad (2.1)$$

Where:

C = run-off coefficient (dimensionless),

I = average rainfall intensity over catchment (mm/hour), and

A = Area (km^2).

The RM assumes the following (Haarhoff and Cassa, 2009):

- a constant rainfall intensity throughout the rainfall event.
- rainfall is uniformly distributed across the entire catchment.
- peak discharge occurs when the duration of the rainfall event equals the time of concentration.
- runoff coefficients do not change for different rainfall durations.
- the return period of the design rainfall event is equal to the return period of the design flood.

2.4.2 *SCS-SA*

The SCS model is an event-based model used globally for DFE (Hawkins *et al.*, 2014; Aichele and Andresen, 2013). The SCS-SA model was created as an adaptation to the original SCS model by Schulze and Schmidt (1987) to better represent South African soil types. The primary outputs from of the SCS-SA model are surface runoff volumes and peak discharges. Stormflow depths and peak discharges are calculated using Equation 2.2, Equation 2.3, Equation 2.4 and Equation 2.5, which uses design rainfall depths and a catchment's runoff response index referred to as the Curve Number (CN). The CN is a dimensionless parameter that ranges between 0 to 100 and which accounts for three physiographic catchment characteristics, *i.e.* land cover, land treatment and soils (Savvidou *et al.*, 2016). A rainfall event must satisfy initial abstractions which consist of depression storage, interception and infiltration into the soil prior to the commencement of runoff. Once the runoff commences, the quantity of actual infiltration increases with increasing rainfall up to the potential maximum soil water retention.

$$Q = \frac{(P-I_a)^2}{(P-I_a+S_w)} \qquad (2.2)$$

Where:

Q = stormflow depth (mm),

P = daily rainfall depth (mm),

I_a = initial losses (mm), and

S_w = potential maximum soil water retention (mm).

$$S_w = \frac{25400}{CN} - 254 \qquad (2.3)$$

Where:

CN = curve number (dimensionless),

$$\Delta Q_p = \frac{0.2083 \times A \times \Delta Q}{\frac{\Delta D}{2}+T_l} \qquad (2.4)$$

Where:

ΔQ_p = peak discharge of incremental unit hydrograph ($m^3.s^{-1}$),

A = catchment area (km^2),

ΔQ = incremental stormflow depth (mm)

ΔD = unit duration of time (hours), and

T_l = catchment lag (mm),

$$T_l = \frac{A^{0.35} \times MAP^{1.1}}{41.67 \times y^{0.3} \times (I_{30})^{0.87}} \qquad (2.5)$$

Where:

MAP = mean annual precipitation (mm),

y = average catchment slope (%), and

I_{30} = 2-year return period 30-minute rainfall intensity (mm.hr^{-1}).

2.5 Floodline modelling

2.5.1 *Methods and Input Data*

The Hydraulic Engineering Centre of the US army corps of engineers – River Analysis Software (HEC-RAS) (USACE, 2019) model was used to determine the flooded extents of the primary watercourses within the Clover Property boundary. The HEC-RAS model simulates the total energy of water by applying principles of mass, continuity and momentum as well as roughness factors between all cells. Input requirements of the 2D HEC-RAS model includes, *inter alia*, design flood hydrographs, dimensions and levels of hydraulic structures, terrain elevation data and hydraulic roughness coefficients. Design flood peak discharges and hydrographs estimated using the conservative results as mentioned in Section 3.3 were inputted to HEC-RAS. Details of hydraulic structures were obtained from a site survey undertaken. A description of the inputted terrain elevation data and hydraulic coefficients are described below. Inlet boundary conditions were set from the design flow hydrograph modelled for each subcatchment with the boundaries longitudinal slope for distribution of flow. The outlet boundary condition was set based on a normal depth determined by the longitudinal hydraulic slope measured off the LiDAR data. Based on the above, a pre-development (without the abstraction weir) and a post development (with the abstraction weir) were modelled. The topographical data and Manning's roughness co-efficient as inputted to the HEC-RAS models are provided in the sections below.

2.5.2 *Topographical data*

For any 2D surface flow modelling, a digital elevation model (DEM) of the terrain is required. Two datasets were assessed to be used in the study i.e. a). eThekwini LiDAR (used to generate a 0.25m DTM) and b). DEMSA2 L3 2m DTM. The DEMSA2 L3 (DTM) was created by using GEOBIA on a combination of aerial imagery, Sentinel 2 satellite imagery, and the L1 DSM to automatically identify surface features, derive the elevation below the features, and then remove them from the DSM to create the DTM. To further inform and assess the accuracy of the elevation datasets, a site visit was carried out where hydraulic cross sections of the uMhlathuzana River and contributing tributary were surveyed, as well as basic hydraulic structures such as culverts and bridges. Figure 2.2 shows the location of the surveyed cross sections.

The following were noticed upon investigating the datasets for potential application:

- The current profile of the uMhlathuzana River is significantly different from that of the eThekwini and DEMSA2 datasets, both of which were capture in 2016. This is probably as a result of the 2022 flood events. The river profile is now much wider, with the inundation extent being greater. There are also significant sediment deposits throughout the watercourse.
- After a thorough analysis of the DEMSA2 product, primarily by comparison to the insitu cross sections, the product was considered inaccurate, unrepresentative and lacked hydraulic definition which would result in irrelevant floodline extents. A methodology to artificially create an elevation model by editing the DEMSA2 with the insitu cross sections

was explored. It was concluded that this artificially created elevation model would also lack hydraulic definition throughout the watercourse profile which would also result in an unrepresentative outcome.

- The higher resolution and detailed eThekwini dataset was used in the modelling. Even though outdated, the eThekwini LiDAR dataset is a far superior product and provides adequate hydraulic definition for modelling purposes.

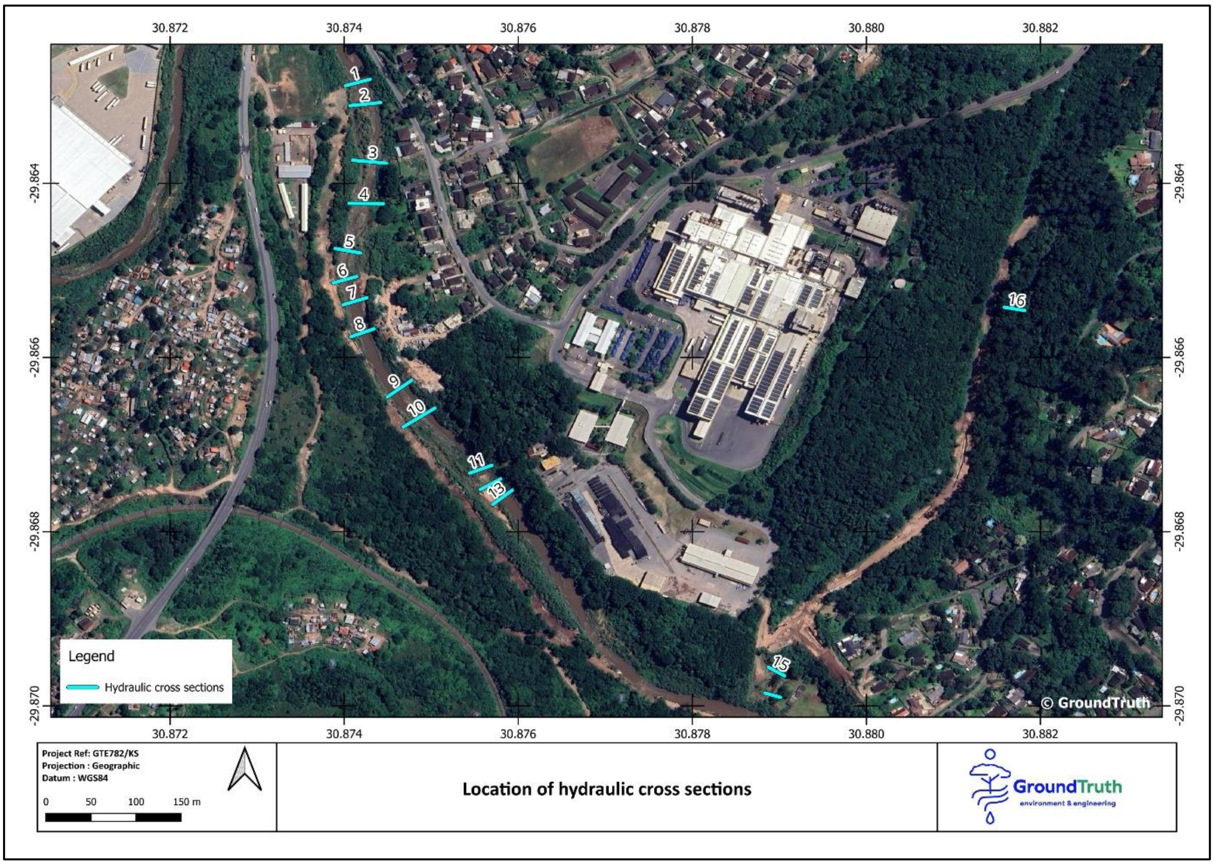

Figure 2.2 Location of the surveyed hydraulic cross sections

2.5.3 *Mannings roughness coefficients*

The Manning's n roughness coefficients were determined by assessing the onsite vegetation and land use characteristics. The values that were used are given in Table 2.3 (Chow *et al*, 1988).

Table 2.3 Selected Manning's n values per land use

Description	Manning's n values
Dense vegetation (Forestry/sugarcane)	0.100
Bare soil	0.020
Grassland	0.030
Watercourse	0.035
Urban (residential)	5.000

2.6 Water Balance

The water balance was compiled for the proposed river abstraction with a view to being used as a decision-making tool for the Department of Water and Sanitation (DWS) in terms the submission of a Water Use Licence Application (WULA). The primary focus of this water balance, and most considerable of the water uses on the site, is the main processing facility. The facility comprises of an approximate hardened area of 109 954m^2 (higher runoff potential) and a pervious area of 20 588m^2 (lower runoff potential). Runoff generated by precipitation on the facility footprint is directed through a series of drains into the adjacent natural catchment.

The proposed operation of the abstraction and associated treatment is as follows, as well as a schematic as shown in Figure 2.3, as provided by the client:

- The feed water from the river will be pumped through basket strainers, which will remove larger suspended solids from the feed and filter the water down to 2 mm.
- From the strainers, the feed will be dosed with a suitable oxidant and proceed to the settling tank, which also allows for settling of heavy suspended solids like sand. This is especially important during flooding conditions in the river.
- The settling tank overflow will be dosed with coagulant to destabilize and neutralize the charged particles in the water, allowing for floc formation, which will aid in achieving more effective settling. The coagulated feed will then proceed to the rapid mixing tank, to ensure adequate chemical mixing. The overflow from the rapid mixing tank will be dosed with flocculant which will ensure that the flocs formed by coagulation, will agglomerate. This further enhances the settling efficiency of the suspended solids. The coagulated and flocculated feed will then proceed to the slow mixing tank, which will allow for enough retention time to ensure proper floc formation and distribution, as well as gentle agitation as to not break the forming flocs.
- From the mixing tanks the feed will proceed to the Dissolved Air Flotation (DAF) for the removal of mainly algae, and some particulate COD and suspended solids.
- The DAF overflow will proceed to the clarified water tank, from where the water will be pumped to the media filters, for final removal of residual suspended solids.
- At this point the water will be close to potable standard, but due to the detectable amounts of untreated sewage in the water, there is a risk of micro pollutants in the water that can have adverse effects on human health. These micro pollutants (also known as Compounds of Emerging Concern, CECs) include hormones, pharmaceuticals, pesticides, herbicides, etc. and are notoriously difficult to remove from water. A UV-Hypochlorite Advanced Oxidation Reactors (AORs) will be specifically designed for treatment.
- The Advance Oxidation Reactors (AOR) final product will be stored in the product tank and be continuously disinfected by the in-tank UV, for further distribution and use.
- All plant waste, including backwash, DAF Sludge and DAF scum will be sent to Clover's effluent treatment plant.
- All process pumps will be on standby for redundancy and in order to limit plant down-time. Allowance was made for automated backwash and will be timer-based or triggered via a pressure differential. Allowance was also made for automated desludging of the DAF, which will be timer-based.

- Apart from the apparent benefit of 24/7 monitoring and response, the system will also be able to provide Clover with routine automated reports on water and chemical usage of the system.

Figure 2.3 Location of plant and *proposed abstraction point (provided by the client)

*Please note that the abstraction point is proposed by the client and will need to be finalised by a team of engineers and associated environmental specialists.

To simplify the above process, and for the purposes of this conceptual water balance, water use by the processing plant was lumped into the following categories:

- rainfall onto the system;
- river abstraction;
- environmental losses and stormwater runoff;
- water consumption for the generation of product, domestic use and sewage; and
- generated domestic waste, sewage and by-product from the processing plant.

3. RESULTS

3.1 Site Overview and Baseline Hydrology

3.1.1 *Landcover*

Landcover was assessed using the South African National Landcover Database (SANLC), 2020 (DEA and GTI. 2020). and visual inspection using Google Earth Imagery. The predominant landcovers are residential, and a natural forestry and sugarcane vegetation type.

3.1.2 *Climate*

Climate, amongst other factors, influences soil-water processes and peak flows. The most influential climatic parameter on peak flow estimates is rainfall. Rainfall intensity, duration, evaporative demand, and runoff were considered in this study to indicate rainfall partitioning within the project area. Monthly historically averaged rainfall and evaporation factors as used in the WR2012 study for the U60F quaternary are shown in Table 3.1.

Table 3.1 Average historical monthly rainfall, evaporation and evaporation conversion factors for the U60F quaternary

Month	Average Rainfall totals (mm)	Evaporation (mm)	SPan Factor	APan Factor
October	102	113	0.8	0.61
November	121	116	1.0	0.76
December	126	127	1.0	0.77
January	131	126	1.0	0.77
February	119	113	1.0	0.76
March	119	112	1.0	0.76
April	64	91	1.0	0.73
May	42	77	1.0	0.71
June	25	67	1.0	0.68
July	27	71	0.8	0.55
August	35	87	0.8	0.58
September	65	102	0.8	0.60

3.2 Design Rainfall Estimation

The estimated gridded short (<=1day) and long (>1day) duration design rainfall (Smithers and Schulze, 2003) events are shown in Table 3.2 and Table 3.3 for Catchment 1 and Catchment 2, respectively. These design events were used in the design flood estimation.

Table 3.2 Gridded short and long duration design rainfall for Catchment 1

Duration	2-year	5-year	10-year	20-year	50-year	100-year	200-year
5 m	14.2	20.7	26.1	32.4	42.3	51.4	62.2
10 m	17.6	25.7	32.4	40.2	52.5	63.8	77.2
15 m	20.0	29.1	36.8	45.6	59.6	72.4	87.7
30 m	26.0	37.8	47.8	59.2	77.4	94.1	113.8
45 m	30.3	44.1	55.6	69.0	90.2	109.6	132.6
1 h	33.7	49.1	62.0	76.9	100.5	122.2	147.8
1.5 h	39.3	57.3	72.3	89.6	117.1	142.3	172.2
2 h	43.8	63.8	80.5	99.8	130.5	158.6	191.9
4 h	52.2	76.0	95.9	118.9	155.5	189.0	228.6
6 h	57.8	84.2	106.3	131.7	172.2	209.3	253.3
8 h	62.2	90.5	114.3	141.6	185.2	225.1	272.4
10 h	65.8	95.8	120.9	149.9	195.9	238.1	288.1
12 h	68.9	100.3	126.6	156.9	205.2	249.4	301.7
16 h	74.1	107.9	136.1	168.7	220.6	268.1	324.4
20 h	78.4	114.1	144	178.5	233.4	283.7	343.2
24 h	82.0	119.5	150.8	186.9	244.4	297.0	359.4
1 d	69.6	101.4	127.9	158.6	207.3	252.0	304.9
2 d	88.4	128.8	162.5	201.4	263.4	320.1	387.3
3 d	101.7	148.1	186.9	231.7	302.9	368.2	445.5
4 d	109.1	158.9	200.6	248.6	325.1	395.1	478.1
5 d	115.3	167.9	211.9	262.6	343.4	417.4	505.0
6 d	120.6	175.6	221.6	274.7	359.2	436.5	528.2
7 d	125.2	182.4	230.2	285.3	373	453.4	548.6

Table 3.3 Gridded short and long duration design rainfall for Catchment 2

Duration	2-year	5-year	10-year	20-year	50-year	100-year	200-year
5 m	9.6	14.3	18.1	22.1	28.2	33.5	39.4
10 m	14.9	22.3	28.1	34.4	43.8	52.0	61.2
15 m	19.2	28.8	36.3	44.5	56.7	67.3	79.2
30 m	26.0	39.0	49.1	60.1	76.6	90.9	107.0
45 m	31.0	46.5	58.5	71.7	91.4	108.4	127.6
1 h	35.1	52.6	66.3	81.2	103.6	122.9	144.6
1.5 h	41.9	62.8	79.1	96.9	123.5	146.5	172.4
2 h	47.5	71.1	89.6	109.8	139.9	166.0	195.3
4 h	58.4	87.5	110.2	135.0	172.2	204.2	240.3
6 h	65.9	98.8	124.4	152.4	194.3	230.6	271.2
8 h	71.8	107.6	135.6	166.1	211.8	251.3	295.6
10 h	76.8	115.1	144.9	177.6	226.4	268.6	316.0
12 h	81.1	121.5	153.1	187.5	239.1	283.6	333.7
16 h	88.3	132.4	166.8	204.4	260.5	309.1	363.6
20 h	94.4	141.5	178.3	218.5	278.5	330.4	388.7
24 h	99.7	149.5	188.3	230.7	294.1	348.9	410.5
1 d	83.0	124.4	156.7	192.0	244.7	290.3	341.6
2 d	104.5	156.6	197.3	241.7	308.2	365.6	430.1
3 d	119.6	179.2	225.8	276.6	352.6	418.3	492.2
4 d	125.9	188.7	237.7	291.2	371.2	440.4	518.1
5 d	131.0	196.3	247.4	303.0	386.4	458.3	539.2
6 d	135.3	202.9	255.6	313.1	399.2	473.5	557.1
7 d	139.1	208.5	262.7	321.8	410.3	486.8	572.7

3.3 Design Flood Peak Estimation

The 2-, 5-, 10-, 20-, 50-, 100-year RP design flood peak events were estimated for the delineated catchment upstream of the outlet of the study site, as shown in Figure 2.1. The influence of the proposed development i.e. the abstraction weir on the estimated design flood events under the post development scenario, is considered to be insignificant. As such a single set of flood estimates are provided which is representative of both pre- and post-development scenarios, as shown in Table 3.4.

Table 3.4 Estimated design peak flows using SCS-SA and the Rational Method which are representative for both pre- and post-development scenarios

Method	Design Peak Flows (m³/s)					
	2-Year	5-Year	10-Year	20-Year	50-Year	100-Year
SCS-SA	20.8	63.2	114.1	182.8	303.3	427.7
Rational Method	90.2	134.5	173.7	222.0	310.6	403.7

The difference in estimated peak events between the two models can be attributed to varied degrees of antecedent soil moisture accounting and water budgeting. The larger return periods, i.e. the 50 and 100 year RP, are particularly important for this study, of which the model results are comparable. Given the nature of the project, a conservative approach was adopted i.e., the larger estimated event from the Rational and SCS-SA model was used for the floodline delineation.

3.4 Floodline Modelling

Following the determination of the peak discharge events, a two-dimensional (2D) surface flow hydraulic model was set up in order to determine the floodline extents using the 2D modelling approach.

3.4.1 *Results*

The flooded extents and depths simulated in HEC-RAS were exported as a raster image and added to available imagery. As an example and as a representation of a larger potential impact, the 1:100 year floodline extent and water depth for the pre-development and post development scenarios are shown in Figure 3.1. Results for the 1:2-, 5-, 10-, and 50-year events under pre- and post-development conditions are shown in the Appendix A. A general measure of model stability and tool used to verify the reliability of the results obtained from a HEC-RAS 2D model is to determine the Courant number. Generally, the Courant number should not exceed the value of 2.0, with it preferably being lower than 1.0. Analysis of the model showed that the Courant number for the model extent was at a maximum of 0.51 for the 1:100year flood, thus confirming the stability of the modelled results.

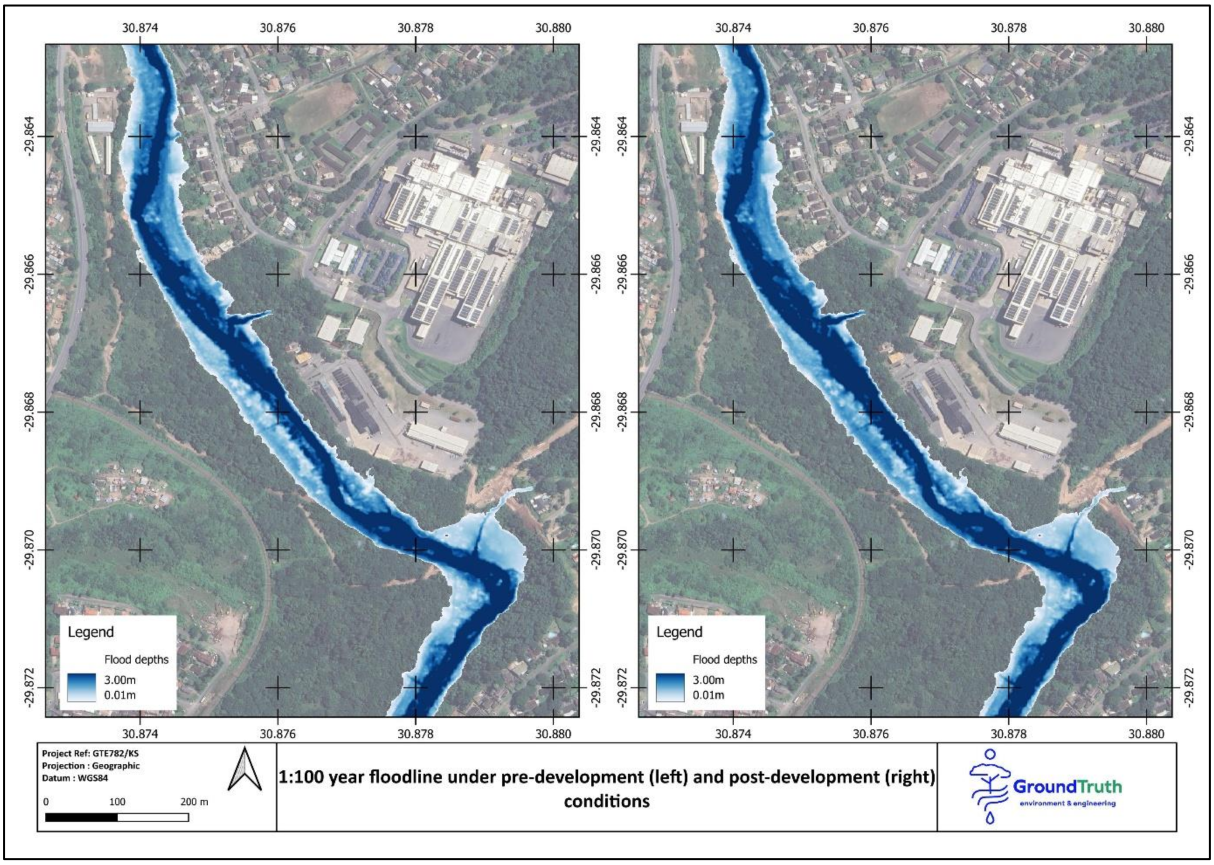

Figure 3.1 1:100 year floodline extent under the pre-development (no abstraction weir) and post-development scenarios (with abstraction weir)

By a visual comparison of the pre- and post-development 1:100 year floodline extents, as shown in Figure 3.1, the presence of the abstraction weir does not result in significant backflooding and does not lead to flood inundation of existing property boundaries. Furthermore, as an indication of satisfactory modelling methodology and results, the estimated 1:100yr pre-development floodline (without the influence of the proposed post-development structure) was compared to floodlines delineated by eThekwini, as shown in Figure 3.2. A visual comparison shows that the estimated 1:100 year floodline was within, and similar to, the bounding extent of eThekwini's indicating that the applied methodology yielded satisfactory results.

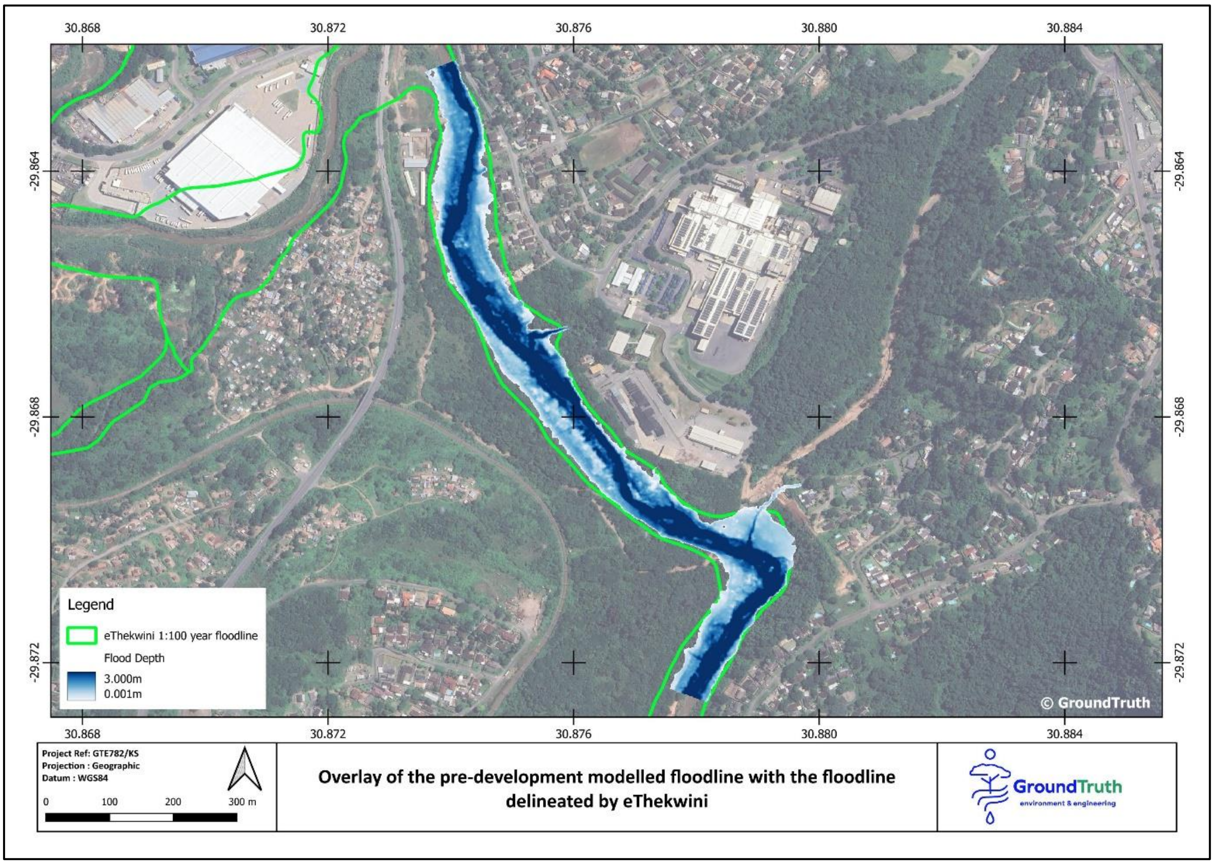

Figure 3.2 Overlay of the pre-development modelled floodline with the floodline delineated by eThekwini

3.4.2 *Assumptions and Limitations*

The following assumptions and limitations are presented below regarding the hydraulic modelling:

- The river channel has been significantly modified between the captured date of the LiDAR (2016) and present (2025) primarily after the 2022 floods. Due to lack of bathymetry data and to avoid creating a DEM which would misrepresent reality, the 2016 LiDAR dataset with an "old" channel profile was used. Hydraulically, the present channel profile has an increased capacity as compared to the older channel profile, and as such, the current modelling represents a worst case scenario.
- Mannings Roughness Coefficients have been based off visual inspection from the imagery and on-site observations. These have not been calibrated to the model through field tests;
- The model does not account for geohydrological contributions or groundwater tables, only surface runoff. It is assumed that for design events greater than the 1:2 year, these would be deemed negligible during the time of the event and only become prominent when modelling base flows following the event; and
- When compared, the insitu and eThekwini derived cross sections presents no significant difference in the maximum depth of channel profile. However it should be noted that, as a worst-case scenario, the model was run on top of the existing water surface elevation surveyed by the LiDAR scanner (as LiDAR does not penetrate water), and the terrain was

not manipulated to represent the channel bed. This approach is further supported as the uMhlathuzana Tributary has flow throughout the year and is unlikely to start off as dry during a time of such an event.

3.5 Water Balance

3.5.1 *Hydrological inputs*

The climatic and catchment-related inputs used to calculate the water balance for the development were largely based on hydrological data and coverages obtained from the WR2012 website. More specifically, the pre-configured network model data for the WRSM/Pitman rainfall-runoff model set up for the Mvoti to Umzimkulu WMA was used.

The hydrological data indicates that the development site has a Mean Annual Precipitation (MAP) of 649mm[1], a Mean Annual Evaporation (MAE) of 1192mm and a Mean Annual Runoff (MAR) of 166.7mm. Monthly proportions of rainfall and evaporation values applicable to the development site, i.e., location of the proposed weir, are summarised in Table 3.5.

Table 3.5 Summary of monthly MAP and MAE for the development site

Descriptor	Month											
	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	June	July	Aug	Sep
MAP (mm)	102	121	126	131	119	119	64	42	25	27	35	65
MAE (mm)	113	116	127	126	113	112	91	77	67	71	87	102

3.5.2 *Assumptions*

Several assumptions were made to calculate the water balance for the development. These assumptions and the relevant sources are summarised in Table 3.6.

Table 3.6 Summary of assumptions and data sources

Variable	Value	Comment
Rainfall (mm)	976	Obtained from WR2012 for Quaternary Catchment U60F
Evaporation (mm)	1200	Obtained from WR2012
Property footprint – hardened surfaces (m^2)	20 588	Estimated
Property footprint – pervious surfaces (m^2)	109 954	Estimated
Ratio of rainfall to runoff	0.81	Based on WR2012
Initial rainfall losses to hardened surfaces	10 %	Where 10 % of rainfall is lost to initial catchment abstractions (Schulze et al., 2004)
Average total water consumption for operations	30%	Client provided
Average total water waste from operations	70%	Client provided

3.5.3 *Conceptual water balance*

Based on the above inputs and assumptions, the conceptual water balance for the development, shown diagrammatically with values in average m^3/day is shown in Figure 3.3. The proposed operation of the abstraction and associated treatment is provided in Section 6.1.3. Therefore, as a simplification, water use by the processing plant was lumped into the following categories:

- rainfall onto the system (generated an average daily volume of 349m^3);
- environmental losses and stormwater runoff (an average daily volume of 349m^3);
- water consumption for the generation of product, domestic use and sewage an average daily volume of 1208m^3); and
- generated domestic waste, sewage and by-product from the processing plant (2901m^3).

The conceptualised water balance does result in a net 0 balance, which is the primary objective of the study.

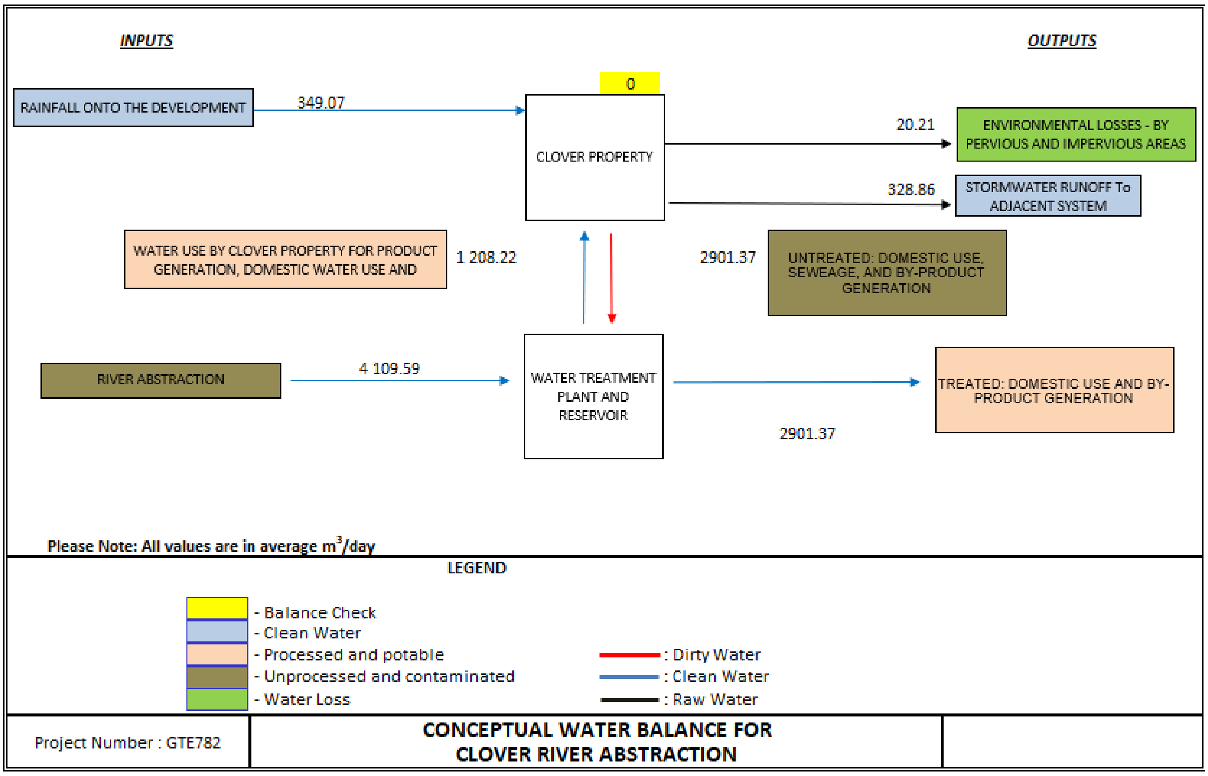

Figure 3.3 Conceptual water balance for the clover river abstraction

4. RECOMMENDATIONS

For the purposes of this project, it is recommended that:

- Water quality of treated water which is to be discarded to the environment be routinely monitored.
- Routine inspection of stormwater systems to ensure there are no blockages within the system which could lead to unnecessary flooding.
- A detailed hydrological and hydraulic modelling analysis of the operation relating to the abstraction of water post construction of the weir be carried out. The primary aim would be to assess the functionality/adequacy of the conceptual design of the weir under realised river flow conditions.
- A detailed ecological water reserve study/hydrological yield modelling to assess the impact of the abstraction on base flow conditions be carried out.
- Should it be required, a detailed LiDAR and bathymetry study of a portion of the uMhlatuzana River could be undertaken to better inform the hydrological and hydraulic modelling.

5. CONCLUSIONS

A hydrological assessment which included hydrological modelling, a conceptual water balance and a floodline delineation as part of a Water-Use License Application (WULA) for the Clover SA facility was undertaken. The baseline catchment hydrology utilised data from the WR2012 study whilst the representative catchment design rainfall was estimated using gridded short and long duration rainfall estimates. The pre- and post-development flood peak estimates were considered to be the same as the abstraction weir would have minimal impact on the estimated peaks. The conceptual water balance was also simplified by lumping up water uses by the processing plant. The floodline assessment was undertaken to delineate the 2-, 5-, 10-, 50- and 100-year design flood peak. Floodlines were generated for both pre- and post-development scenarios i.e. before and after the implementation of the proposed weir. Results indicate that the presence of the abstraction weir does not result in significant backflooding and does not lead to flood inundation of existing property boundaries. It can be concluded from the water balance that the proposed abstraction can meet water requirements from the Clover processing plan.

6. REFERENCES

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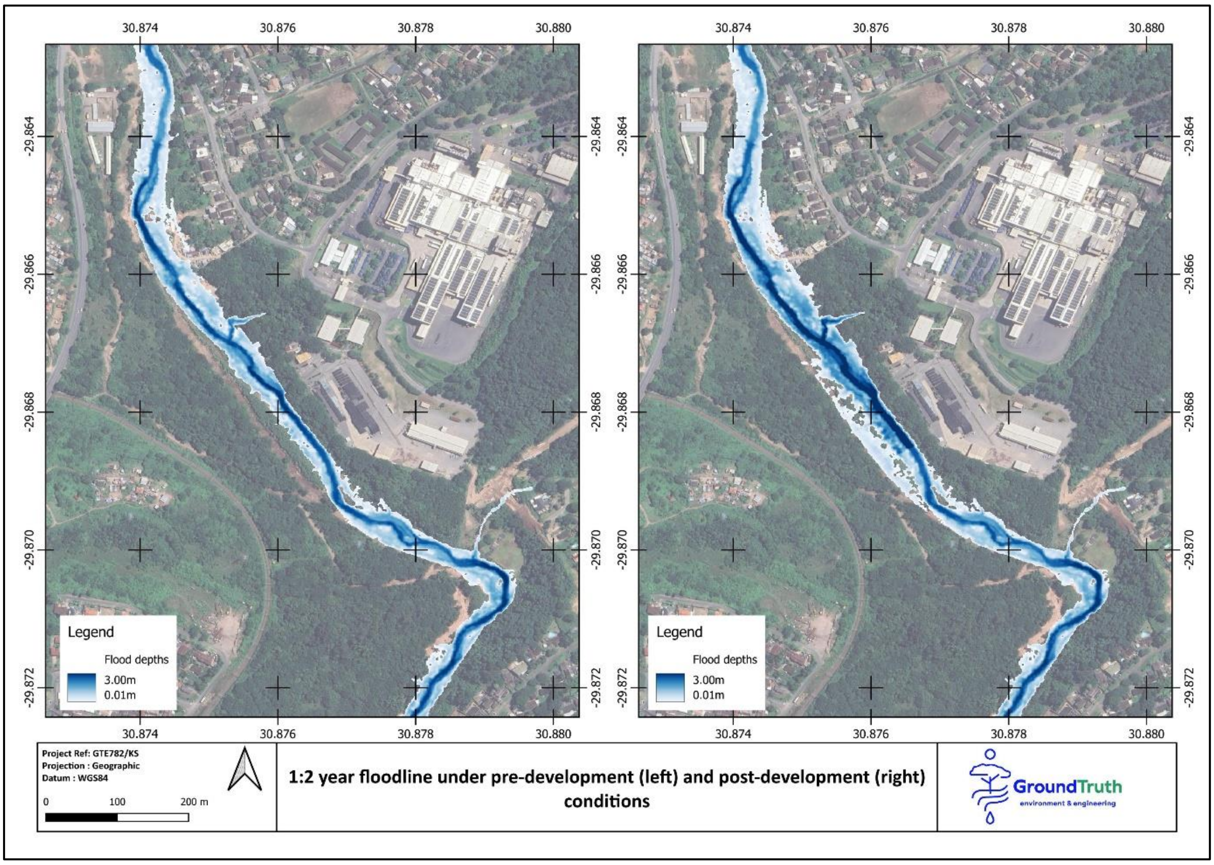
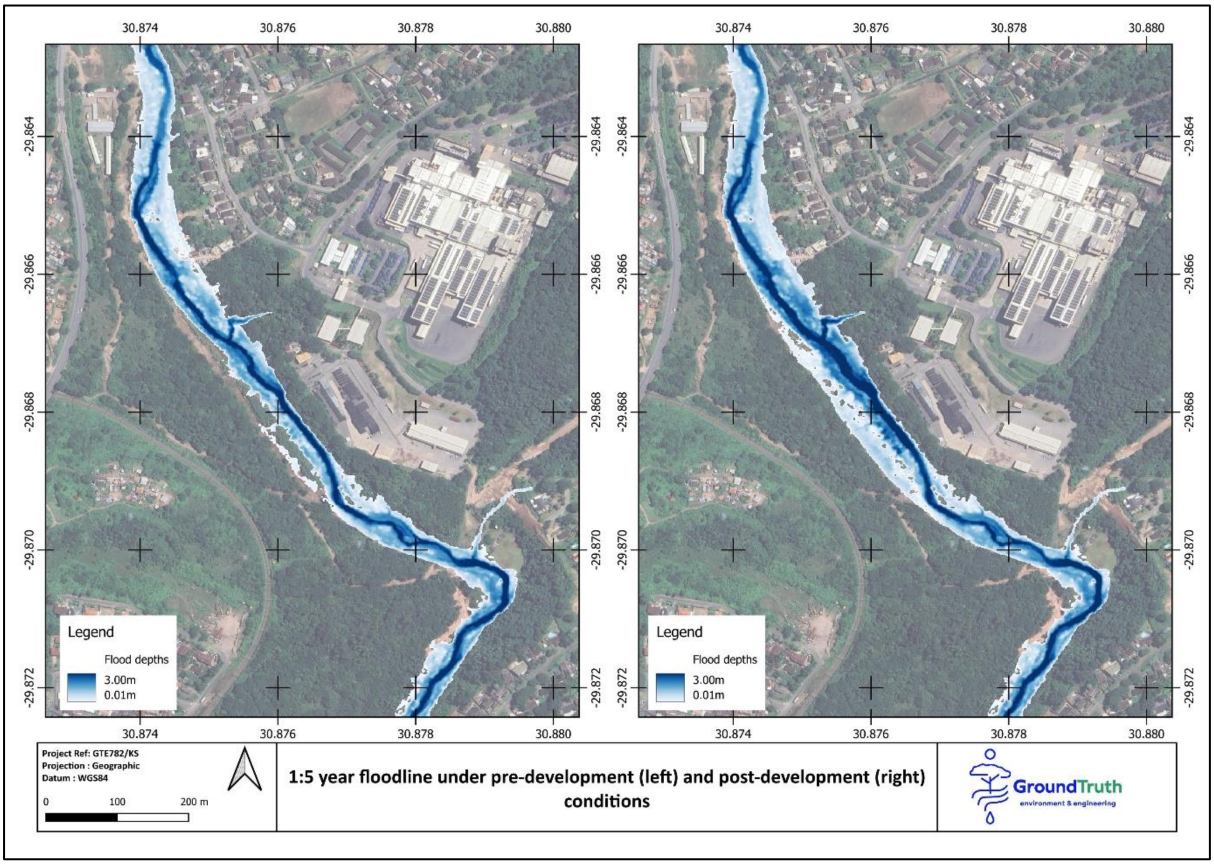
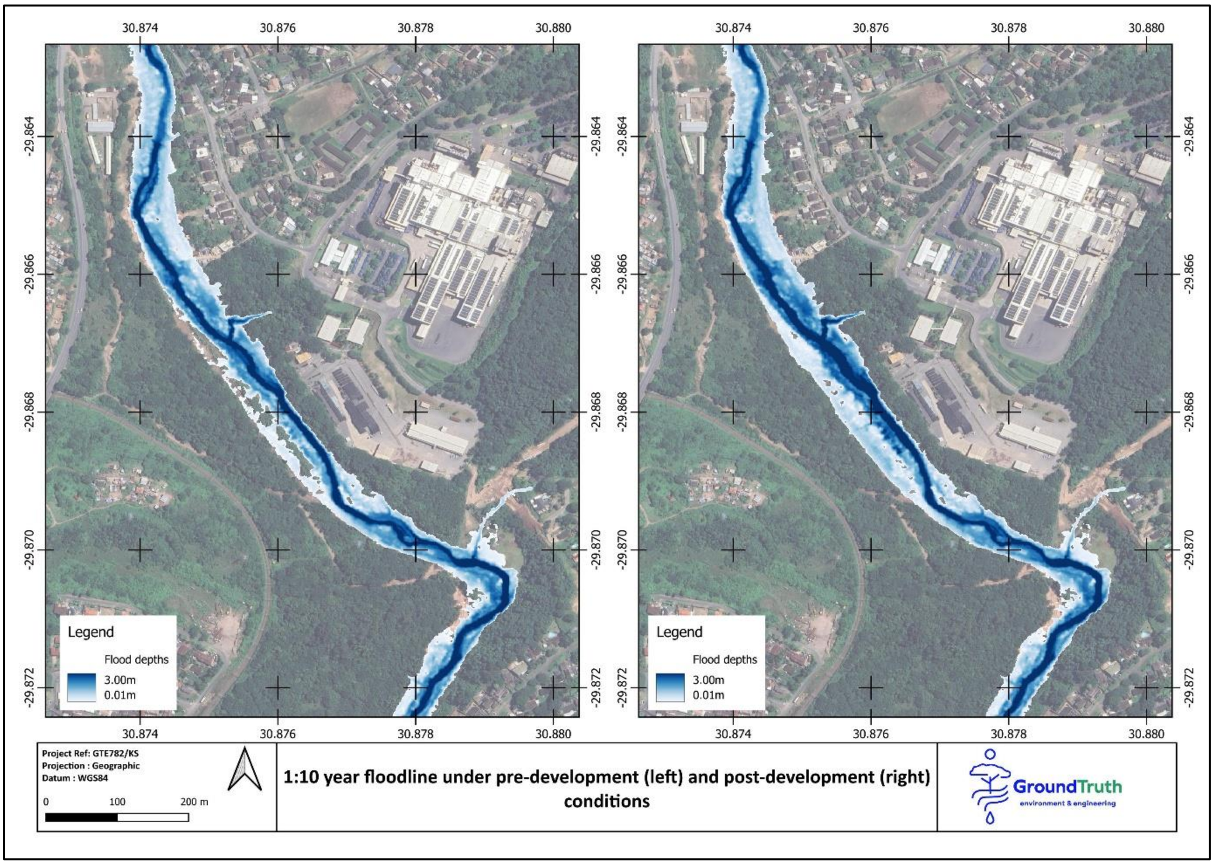
APPENDIX A



Appendix 1 1:2 year floodline extent under the pre-development (no abstraction weir) and post-development scenarios (with abstraction weir)

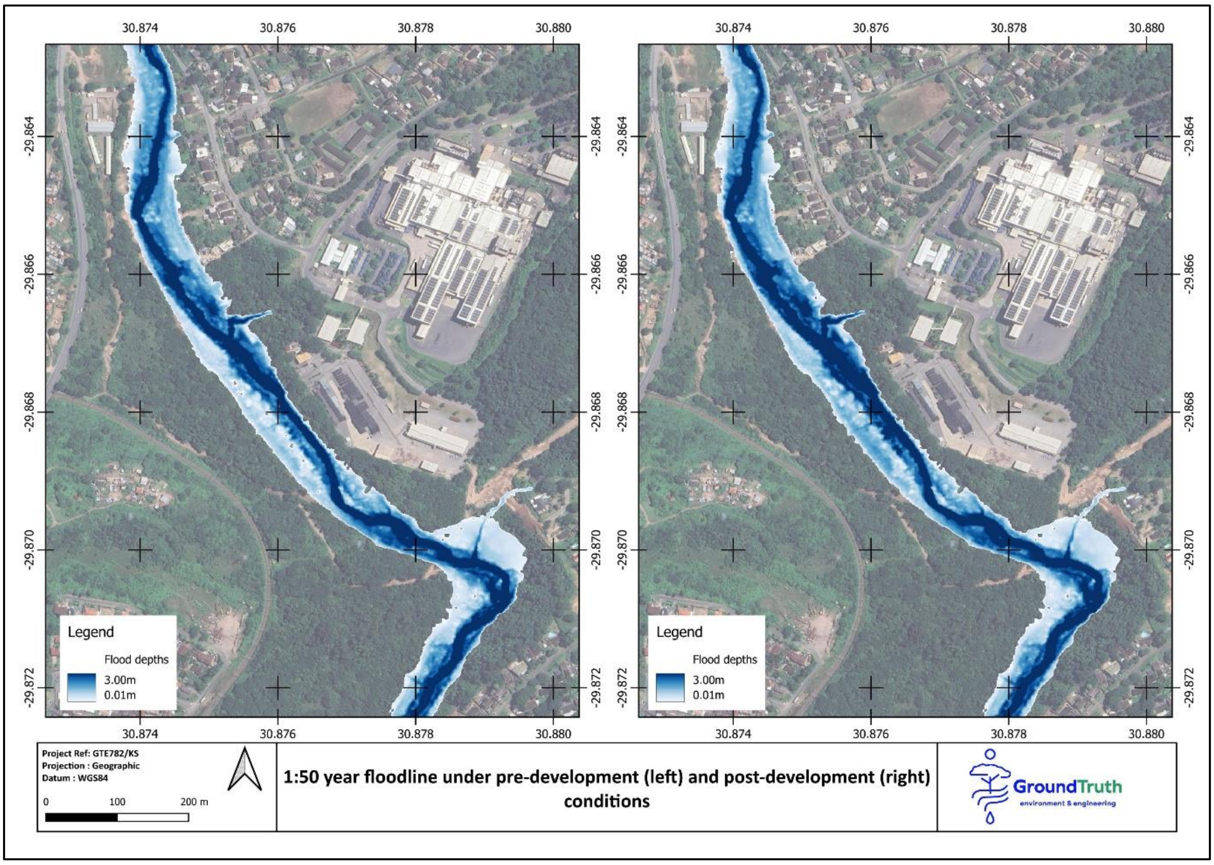


Appendix 2 1:5 year floodline extent under the pre-development (no abstraction weir) and post-development scenarios (with abstraction weir)



1:10 year floodline under pre-development (left) and post-development (right) conditions

Appendix 3 1:10 year floodline extent under the pre-development (no abstraction weir) and post-development scenarios (with abstraction weir)



Appendix 4 1:50 year floodline extent under the pre-development (no abstraction weir) and post-development scenarios (with abstraction weir)