WELMOED DEVELOPMENT

PORTION 28 OF FARM 468, LYNEDOCH, WITHIN THE STELLENBOSCH MUNICIPAL AREA, WESTERN CAPE

STORMWATER MANAGEMENT PLAN

REVISION 00

APRIL 2024

Uniqon Developers (Pty) Ltd.

17 Catherine Road

Shere Agricultural Holdings

Pretoria East

0081

UDS Africa

9 Elektron Road

Techno Park

Stellenbosch, 7600

Contents

STORMWATER MANAGEMENT PLAN FOR THE WELMOED DEVELOPMENT ON PORTION 28 OF FARM 468, LYNEDOCH, WITHIN THE STELLENBOSCH MUNICIPAL AREA, WESTERN CAPE

1. Background

UDS Africa was appointed by Mr Etienne Coetzer to compile a stormwater management plan in support of the development application for the proposed Welmoed development.

The scope of our appointment includes the compilation of a stormwater management plan to support the development application. Our mandate includes compliance with the latest Stellenbosch Municipal by-laws in order to meet the requirements regarding the quality and quantity treatment volumes of the post development stormwater run-off generated on Farm 468 Portion 28.

The site covers an area of 45.48 ha with and is currently used for agricultural purposes.

2. Locality

The site is located in Lynedoch, within the Stellenbosch Municipal area. The site is bordered to the north, east and west agricultural land and to the south by Lynedoch Road. Below is a locality plan for reference (outlined in red, area highlighted in yellow).

Figure 1: Locality plan of Portion 28 of Farm 468

3. The development

The proposed development will be large residential in nature, with a clubhouse, commercial and school component. Please refer to **Annexure A** for a copy of the site development plan prepared by Urban Studio Architects and Urban Designers. Below is a summary of the land uses to from part of the proposed development:

USE	AREA	UNITS
School	1.78 Ha	
Commercial	0.50 Ha	
Clubhouse	0.18Ha	
Mixed use/ Residential (80/Ha)	6.58 Ha	515
Residential (40/Ha)	8.74 Ha	355
Residential (allotment villas)	10.31 Ha	14

Table 1: Summary of land uses

4. Mean annual precipitation

For further hydrological calculations a MAP of 667mm is used.

Rainfall intensities for the 0.5 year RI; 24 h storm event and 1 year RI; 24 h storm event was calculated using the following formula:

$$i = \frac{(3.4 + 0.023\ MAP)\ R^{0.3}}{(0.2 + td)^{0.75}}$$

Where i = rainfall intensity; R = Recurrence Interval; td = storm duration in hours

5. Hydrology

5.1 Calculation method

The catchment area is small and the Rational Method is deemed to be a suitable method to calculate the flood peaks.

Currently stormwater from the higher lying farm 489 and 468 portion 22 is cut-off by means of a berm and diverted along the eastern and western boundary respectively.

The site itself has watershed resulting in a northern catchment area of 21.72Ha draining towards the north eastern corner of the site and a southern catchment area of 23.76Ha draining towards the south eastern corner of the site.

5.2 Current Developed Level

(for 1:50 year recurrence interval calculations)

The following catchment characteristics are applicable to the northern catchment area:

- Catchment area = 0.2172km²
- Average slope = 10.60%
- t_c = 33.94 minutes
- C = 0.43
- I = 74.04 mm/h (for 1:50 year)

Peak flow with a recurrence interval of 50 years is calculated as follows:

= 0.95 (CIA)/3.6

= 0.95 (0.43 x 74.04 x 0.2172) /3.6 = **1.825m³/s**

The following catchment characteristics are applicable to the southern catchment area:

- Catchment area = 0.2376km²
- Average slope = 12.17%
- t_c = 32.22 minutes
- C = 0.43
- I = 76.19 mm/h (for 1:50 year)

Peak flow with a recurrence interval of 50 years is calculated as follows:

= 0.95(CIA)/3.6

= 0.95(0.43 x 76.19 x 0.2376)/3.6 = **2.054m³/s**

5.3 Pre-developed Level

(for 1:10 year recurrence interval calculations)

The following catchment characteristics are applicable to the northern catchment area:

- Catchment area = 0.2172km²
- Average slope = 10.60%
- t_c = 33.94 minutes
- C = 0.43
- I = 45.69 mm/h (for 1:10 year)

Peak flow with a recurrence interval of 10 years is calculated as follows:

= 0.85(CIA)/3.6

= 0.85(0.43 x 45.69 x 0.2172)/3.6 = **1.007m³/s**

The following catchment characteristics are applicable to the southern catchment area:

- Catchment area = 0.2376km²
- Average slope = 12.17%
- t_c = 32.22 minutes
- C = 0.43
- I = 47.01 mm/h (for 1:10 year)

Peak flow with a recurrence interval of 10 years is calculated as follows:

= 0.85(CIA)/3.6

= 0.85(0.43 x 47.01 x 0.2376)/3.6 = **1.134m³/s**

5.4 Post development runoff

The catchment characteristics of the northern catchment area will change as follows once developed:

- t_c = 18.67 minutes
- C = 0.61

The flood peaks for the post-development scenario are calculated as follows:

Post Development run-off							
Recurrence Interval	0.5 year: 24 hr event	1 year: 24 hr event	2 Year	5 Year	10 Year	20 Year	50 Year
Rainfall Intensity (mm/h)	1.395	1.718	38.16	50.24	61.85	76.14	100.23
Peak run-off (m^3/ s)	0.038	0.047	1.048	1.471	1.925	2.509	3.486

Table 2: Post development run-off for northern catchment

The catchment characteristics of the southern catchment area will change as follows once developed:

- t_c = 17.82 minutes
- C = 0.61

The flood peaks for the post-development scenario are calculated as follows:

Post Development run-off							
Recurrence Interval	0.5 year: 24 hr event	1 year: 24 hr event	2 Year	5 Year	10 Year	20 Year	50 Year
Rainfall Intensity (mm/h)	1.395	1.718	38.98	51.32	63.17	77.77	102.38
Peak run-off (m^3/ s)	0.042	0.052	1.171	1.644	2.150	2.803	3.895

Table 3: Post development run-off for southern catchment

5.5 Flood lines

A flood line study was completed by AED in June 2023 for the Eerste River located further east of the site. Flood lines were found not to affect Farm 468 Portion 28. Refer to **Annexure B** for a copy of the report.

6. Best management practice

In order to reduce impacts of urban stormwater systems on receiving waters, all stormwater management systems shall be planned and designed in accordance with best practice criteria and guidelines laid down by council, to support water sensitive urban design principles and the following specific sustainable urban drainage system objectives:

- Improve the quality of the runoff;
- Control the quantity and rate of runoff, and
- Encourage natural groundwater recharge

The BMP (Best Management Practice) proposed to improve the quality and control of the quantity of the stormwater runoff by means of providing an attenuation pond. The attenuation pond has the abilities to reduce the suspended solids (80%) and total phosphorous (45%) to the levels required by the Urban Stormwater Impact Policy and to provide storage capacity to reduce the rate of runoff.

6.1 Protect Downstream Properties

The downstream properties must be protected against fairly frequent nuisance floods by reducing the post-developed peak flows with recurrence interval up to 10 years to the undeveloped levels.

The storage volume required for the northern catchment is calculated by the Abt and Grigg Method, and is calculated as follow:

$V = Q_p . t_c (1-\alpha)^2$, where

V = storage volume in (m^3)

Q_p = the post-development peak discharge in (m^3/s)

t_c = time of concentration in (seconds)

α = pre-development / post-development discharge ratio

$V = 1.925 \times 1120 \times (1-0.523)^2 =$ **$489.71\ m^3$**

The storage volume required for the southern catchment is calculated by the Abt and Grigg Method, and is calculated as follow:

$V = Q_p.t_c(1-\alpha)^2$, where

V = storage volume in (m^3)

Q_p = the post-development peak discharge in (m^3/s)

t_c = time of concentration in (seconds)

α = pre-development / post-development discharge ratio

$V = 2.15 \times 1069 \times (1-0.527)^2$ = **513.63 m³**

6.2 Protect Floodplain developments

Floodplain developments are to be protected by limiting the post-development run-offs to that of the existing development run-offs for storms with a recurrence interval of up to 50 years.

The storage volume required for the northern catchment is calculated by the Abt and Grigg Method, and is calculated as follow:

$V = Q_p.t_c(1-\alpha)^2$, where

V = storage volume in (m^3)

Q_p = the post-development peak discharge in (m^3/s)

t_c = time of concentration in (seconds)

α = pre-development / post-development discharge ratio

$V = 3.486 \times 1120 \times (1-0.218)^2$ = **887.02 m³**

The storage volume required for the southern catchment is calculated by the Abt and Grigg Method, and is calculated as follow:

$V = Q_p.t_c(1-\alpha)^2$, where

V = storage volume in (m^3)

Q_p = the post-development peak discharge in (m^3/s)

t_c = time of concentration in (seconds)

α = pre-development / post-development discharge ratio

$V = 3.895 \times 1069 \times (1-0.527)^2 =$ **930.35 m³**

The storage capacity must accommodate the largest value between the protection of downstream properties and protection of floodplain developments. Thus the storage capacity required in terms of quantity control for the northern catchment is **887.02 m³** and **930.35m³** for the southern catchment.

6.3 Water Quality

The water quality volume is calculated by using the following formula as recommended in The Neighborhood Planning and Design Guide, Section L4.1.2:

Northern catchment:

WQV = P x C x A x 10

WQV = water quality volume in (m^3)

P = total rainfall depth in (mm)

C = the runoff coefficient

A = drainage area in (ha)

V = 25 x 0.61 x 21.72 x 10 = **3 312.30m³**

Southern catchment:

WQV = P x C x A x 10

WQV = water quality volume in (m^3)

P = total rainfall depth in (mm)

C = the runoff coefficient

A = drainage area in (ha)

V = 25 x 0.61 x 23.76 x 10 = **3 623.40m³**

Therefor a total treatment volume (quantity and quality) of 4 199.32m^3 will be required for the northern catchment area and a total treatment volume of 4 553.75m^3 for the southern catchment area.

6.4 Attenuation facilities

For the northern catchment area, stormwater will collect in a series of stormwater swales parallel to the roads, before discharging into a swale to the west and south of B1, from where it will discharge into a detention facility which is located on the north eastern corner of the site.

For the southern catchment area, stormwater will collect in a series of stormwater swales parallel to the roads, from where it will discharge into detention facilities located north of B11 and south of B15. These detention facilities will accommodate approximately 20% of the required treatment volume for the southern catchment area.

It is proposed that two further detention facilities are constructed between Baden Powell Drive and Lynedoch road. As part of the proposal it is proposed that the culvert beneath Lynedoch Road is upgraded to an 1800 x 900 rectangular culvert. Refer to **Annexure A** for the site development plan showing the positions of the detention facilities.

The development will be phased. Refer to **Annexure C** for the phasing plan. The required stormwater infrastructure for each phase of the development will be determined once the site development plan for each phase becomes available.

7. Proposed infrastructure

It is proposed that swales are created adjacent to the roads to collect stormwater during small rainfall events. Inlets and a piped stormwater system will be provided below the swale in order to accommodate larger rainfall events.

Due to the steepness of the site, energy breakers will be constructed within the swales and backdrop manholes will be utilised in order to limit the flows within the stormwater pipes.

8. Maintenance

The home owners association will be responsible for the following maintenance activities:

Description of infrastructure	Description of activities to be carried out	Timeline for implementation
Inlet structures and manholes	• Clean and remove litter • Clean and remove sand and silt • Inspect for damages and implement remedial action	6 month basis (at least once before winter)
Pipework	• Clean and remove litter • Clean and remove sand and silt • Inspect for damages and implement remedial action	6 month basis (at least once before winter)
Head wall and gabion structures	• Clean and remove litter • Clean and remove sand and silt • Inspect for damages and implement remedial action	6 month basis (at least once before winter)
Vegetated channels	• Clean and remove litter • Maintain grass to a height of below 50mm • Remove sediment from channel as required	Continuous, but not less than once per month
Embankments of vegetated channels, swales and attenuation ponds	• Inspect embankment for signs of erosion • Implement remedial action as required	6 month basis
Plant species	• Inspect plant species for successful establishment. If unsuccessful plant new/ different species.	6 month basis

Table 4: Maintenance Schedule

9. Conclusion

It has been determined that the provision of attenuation facilities with a combined capacity of **8 753.07m^3** will satisfy the stormwater quantity and quality requirements for the proposed Welmoed Development on Farm 468 Portion 28.

Please do not hesitate to contact the undersigned should you require any additional information.

Yours faithfully,

Compiled by:

Ruaan Siebrits (Pr Tech Eng)

Attachments:

Annexure A – Site Development Plan
Annexure B – Flood line study
Annexure C – Phasing plan

ANNEXURE A

PORTION 28 LAND USE TABLE			
	Site area	**45.48 Ha**	
	Land use	**Area**	**Units**
	School Component (A2)	1.78 Ha	
	Commercial (A1,B5)	0.50 Ha	
	Clubhouse Component(B5)	0.18 Ha	
	Mixed Use Component (B1-4)	0.7 Ha	515 units
	Residential (@80 du/ha)	5.88 Ha	
	Residential (@40 du/ha)	8.74 Ha	355 units
	Alottment Villas	10.31 Ha	14 units
	Total (excluding areas below)	**28.09 Ha**	**884 units**
	Detention & SW area	1.15 Ha	
	Indigenous slopes	6.43 Ha	
	Roads & squares	4.86 Ha	
	Private open space	4.04 Ha	

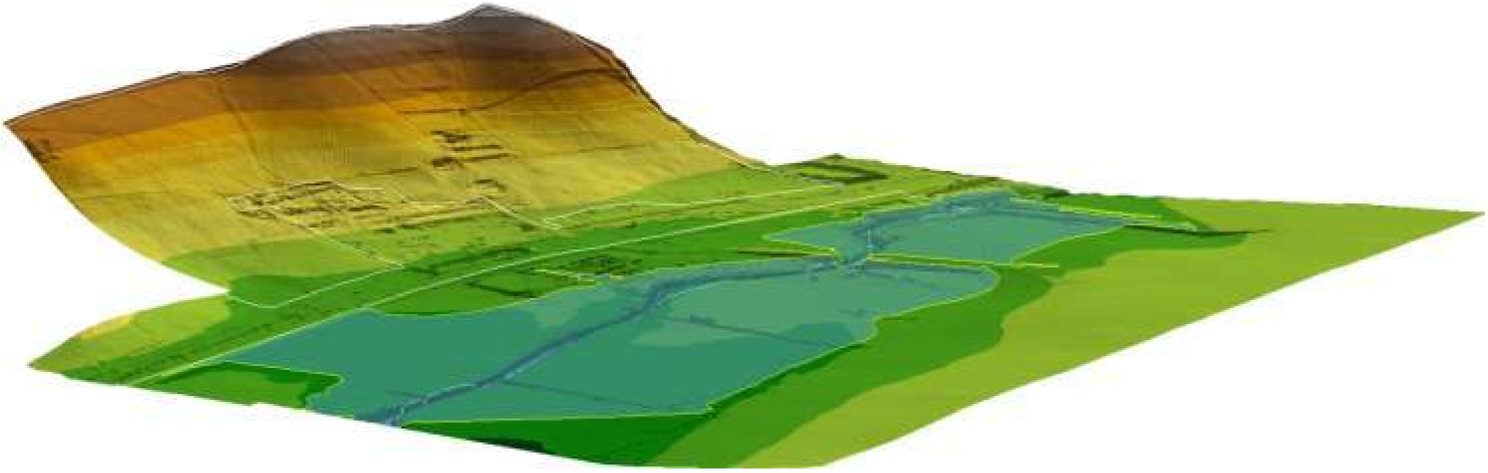
Block Plan (PORTION 28) REV 05.2 | 1:2000 (@A1) | 28.03.2024

WELMOED

URBANSTUDIO architects & urban designers

ANNEXURE B

100-YEAR FLOOD LINES FOR THE EERSTE RIVER AT PORTIONS 27 & 28 OF THE FARM WELMOED ESTATE NO 468, WESTERN CAPE PROVINCE, RSA



Compiled by

Garfield Krige
Pr.Sci.Nat. Aquatic Science (Reg. No. 400068/10)

and

Al Zylstra
Pr.Eng. (Reg. No. 730439)

of

African Environmental Development
129 Malmani Rd
Sterkfontein Country Estates 1739

Tel: 083 657 0560
E-mail: info@aed.co.za

Report Number AED0450/2023
Date of Report: 02/06/2023

Revision Number: 00

This page is intentionally left blank (for back-to-back printing purposes)

<u>This report may be cited as follows:</u>

Krige W. G. & Zylstra A. A. (2023): <u>100-YEAR FLOOD LINES FOR THE EERSTE RIVER AT PORTIONS 27 & 28 OF THE FARM WELMOED ESTATE NO 468, WESTERN CAPE PROVINCE, RSA</u>. Report No. **AED0450 Revision 00**, African Environmental Development

<u>Cover Image</u>: A 3-D DTM of the terrain surrounding the study area, showing the 100-year floodplain of the Eerste River

<u>Prepared for:</u>

UDS Africa

Time Square Building
9 Electron St, Techno Park
Stellenbosch 7600
PO Box 50487, V&A Waterfront
Cape Town 8002
Tel: 021 880 0443 / 072 6519568
Email: ruaan@udsafrica.co.za

<u>Prepared by:</u>

African Environmental Development

129 Malmani Rd,
Sterkfontein Country Estates,
Krugersdorp, RSA 1739
Tel: +27 83 657 0560
Email: info@aed.co.za
garfield@aed.co.za
Website: www.aed.co.za

Table of Contents

List of Acronyms and Abbreviations

Item	Description
~	Approximately (e.g. ~10 m wide = "approximately 10 m wide")
>	Greater than
<	Less than
3-D or 3D	Three dimensional
AED	African Environmental Development
℄	Centreline
CAD	Computer Aided Drawing
DTM	Digital Terrain Model
DEM	Digital Elevation Model
GIS	Geographic Information Service
HRU	Hydrological Research Unit (a division of the Civil Engineering Department of the University of the Witwatersrand in South Africa)
mamsl	Metres above mean sea level (elevation)
MAP	Mean annual precipitation (rainfall, measured in millimetres)
MAR	Mean annual run-off (the amount of the annual rainfall that reaches a watercourse, measured in millimetres)
N	North
P_e	Probability
RI	Return interval (pertaining to floods, e.g. RI of 100-years)
RIs	Plural of RI (Return Intervals)
S	South
SG	Surveyor General (e.g. SG Diagram)
TC	Time of Concentration (time for a storm to reach its maximum rate of rainfall)

African Environmental Development
No 129 Malmani Road
Sterkfontein Country Estates
Krugersdorp

African Environmental Development
PO Box 1588
Rant-en-Dal 1751
Tel: - 083 657 0560
Fax:- 086 670 5102
E-mail: - garfield@aed.co.za
http://www.aed.co.za

100-YEAR FLOOD LINES FOR THE EERSTE RIVER AT PORTIONS 27 & 28 OF THE FARM WELMOED ESTATE NO 468, WESTERN CAPE PROVINCE, RSA

1 Introduction and Background

African Environmental Development (AED) was commissioned by ***Mr Ruaan Siebrits*** of ***UDS Africa***, to develop the 100-year flood lines for the Eerste River at Portions 27 and 28 of the farm Welmoed Estate No 468. The project locates in the Eerste River catchment in the Stellenbosch Municipality, Western Cape Province. The goal of this study was to model the 100-year flood lines for this river in proximity to Portions 27 and 28, hereafter referred to as the "*study area*", as the 100-year floodplain would likely affect Portion 27. This report is a brief description of the methodology used, the assumptions made and a discussion of the resultant 100-year flood lines of this watercourse.

1.1 Study Area

The study area locates in the area of jurisdiction of the Stellenbosch Municipality in the Western Cape Province. The Eerste River drains a mostly rural area with initial steep mountainous surfaces with very high rainfall in the upper reaches of the catchment, and flat surfaces in the downstream parts of the catchment with much lower rainfall.

The Eerste River at the study area drains a surface area of 330.366 Km² to the north, northeast and east of the study area. In comparison to the area at the study area, which receives a Mean Annual Precipitation (MAP) in the vicinity of low to mid 600s mm/a, the eastern part of the catchment receives a very high MAP. In particular the upper reaches of the Eerste River (where the Jonkershoek River rises) receives a MAP in the high 2000s mm/a, even exceeding 3000 mm/a in places and to a lesser extent, the upper reaches of its tributary, the Klippies River, which receives a MAP of over 2000 mm/a in places. Refer to Figure 2 for details of the catchment's rainfall.

Although the Eerste River rises in a mountainous region with steep slopes, at the study area, the river flows onto a wide floodplain with very low slopes. The velocity of flowing water in the river would diminish once this flat area is reached, which implies that the floodplain would be exceptionally wide.

There is a bridge at the study area where Annandale Road passes over the river. It was thus also necessary to include the backwater, produced by this bridge, in the flood model of the river, as it would be highly likely that this bridge would increase the elevation of the 100-year floodplain upstream from this bridge. Please refer to ***Photos 1***, ***2*** and ***3*** for details of this bridge. The surveyor, Johan Louw of *Neil Woodin Surveys*, took these photos on 09/05/2023 during a detailed survey of the bridge.

Photo 1: The road surface of the Annandale Road Bridge, viewing from north to south

Photo 2: The right-hand abutment of the Annandale Rd Bridge across the Eerste River, viewing the intake side of the bridge

Photo 3: The left-hand abutment of the Annandale Rd Bridge, viewing from the intake side of the bridge. (Note: Left and right banks of rivers are always referred to as viewing from up- to downstream)

1.2 The Catchment of the Eerste River

The catchment of the Eerste River is 330.366 Km².

The catchment upstream from the study area comprises of 2 full and one partial quaternary catchment, quaternary catchments G22F, G22G (full catchments) and G22H (partial catchment - ending at the study area just upstream from the confluence of the Vlaeberg River with the Eerste River). Refer to ***Figures 1*** and ***2*** for the catchment of the Eerste River and the general area at, and around the study area.

As stated above, the catchment of the Eerste River up to the study area falls across three quaternary catchments, Quaternary Catchments G22F, G22G and G22H. The three catchments have mean annual precipitation (MAP) values of 1 464.50 mm/a, 753.64,mm/a and 669.23 mm/a respectively (*Bailey & Pitman, 2012*). They fall within Ve*ld Zone 2*, the "*Sclerophylious Bush*" in terms of the Midgley-classification (*Midgley, 1972*). However, as there is a huge difference in the amount of annual precipitation each one of these catchments receive and also within each of the catchments, and as the MAP plays a very important role in the flood model, it was also essential to divide the catchment into smaller sub-catchments and then to determine the surface area of each of the sub-catchments to produce an area-weighted MAP for the total catchment together with the MAP of each of the sub-catchments. Thus, the 1-minute x 1-minute map, shown in ***Figure 2***, was produced. The data that was used to produce ***Figure 2*** was obtained from the Water Research Commission WR2012 document (*Bailey & Pitman, 2012*).

Thus, ***Figure 2*** uses degrees and minutes to create a set of tiles, each comprising of a 1-minute x 1-minute tile (we refer to "tiles" as they are not squares, even though they appear to be square), with rows and columns much like a spreadsheet. The X-axis (columns) in ***Figure 2*** shows the coordinates (degrees and minutes) of the columns while the first row of the Y-axis shows the coordinates in Deg/Min and in the rest we have replaced the coordinates with numbers from 1 to 14, to tie in with the row numbers of the MS Excel Workbook where the area-weighted MAP was calculated. This workbook is attached in ***Appendix 2*** of this report.

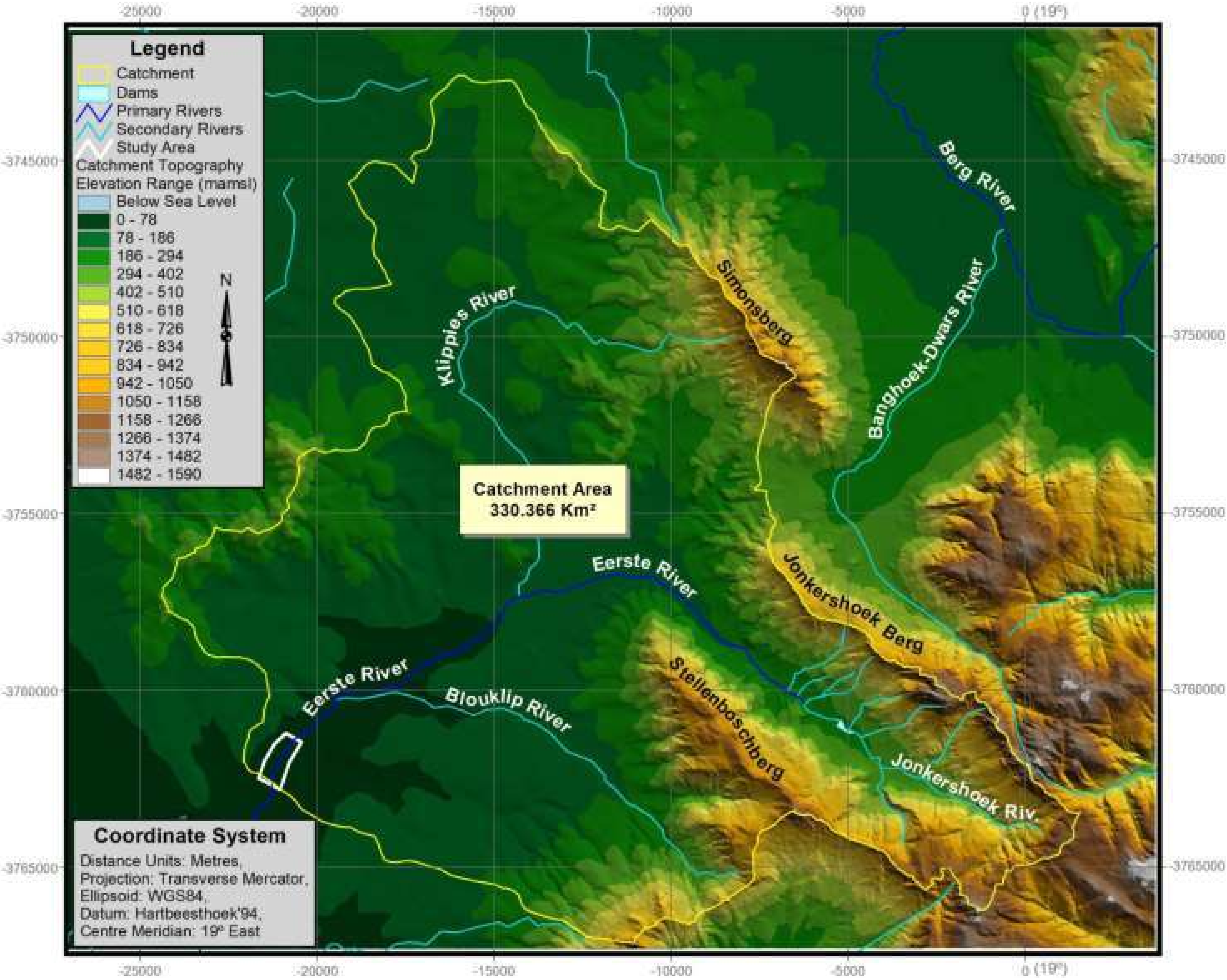


Figure 1: The study area (white polygon), showing the catchment of the Eerste River. Topography produced using the SG 5-m elevation data

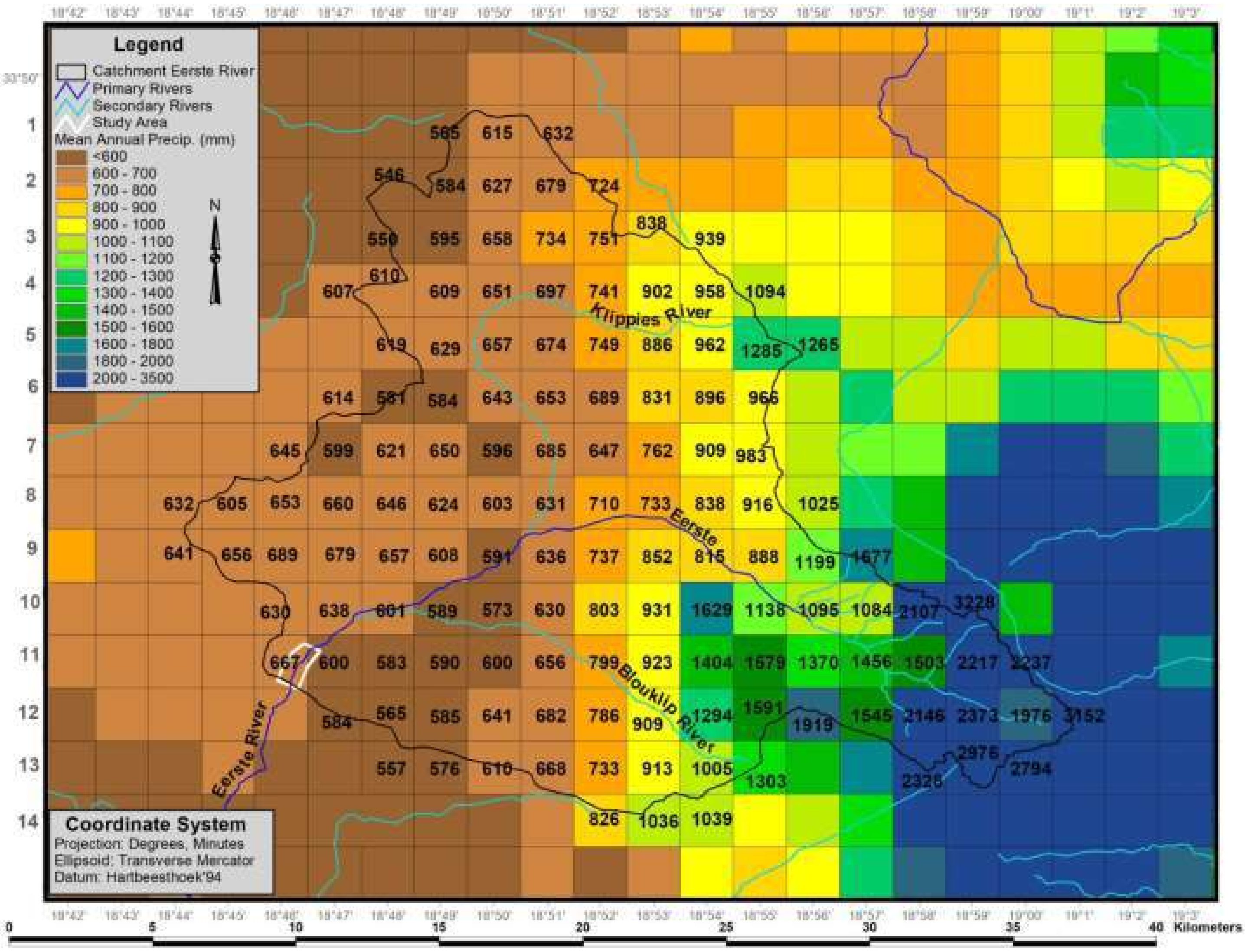


Figure 2: The Eerste River catchment showing 1' x 1' tiles with the MAP written in each of these tiles. The surface area of the tiles around the perimeter of the catchment that were cut into smaller pieces were determined, while the areas of the full ones in the centre of the catchment were added, to calculate a single area-weighted MAP for the catchment. This is done in the workbook in **Appendix 2**

Each tile was allocated its MAP and its surface area and thus it was possible to determine a single area-weighted MAP for the entire catchment up to the study area. This area-weighted MAP was calculated as **921.01-mm/a**. We also calculated the simple average of the MAP of the entire catchment as being 954.39 mm/a, but this value did not take cognisance of the areas of the tiles. Additionally, if the average of the MAP of the three quaternary catchments are determined, it is 962.46 mm/a, again a value not linked to the area of the tiles. So both these values were ignored and only the area-weighted MAP of 921.01 mm/a used in the flood model.

1.3 The shape and coverage of the Eerste River catchment

As can be seen in ***Figure 1***, the catchment of the Eerste River is somewhat triangularly shaped (as opposed to a more elongated shape of most "regular" catchments). This suggests that, rainfall falling along the upper side of the triangle (the side that includes the Jonkershoek Berg and Simonsberg - see ***Figure 1***), drained by the Jonkershoek/Eerste River and the Klippies River, would reach the confluence of these two rivers more-or-less simultaneously. This will give rise to a higher discharge downstream from the confluence than if this catchment had been an elongated catchment. In an elongated catchment, the rainfall falling in the downstream and central parts of the catchment would have left the catchment by the time the rainfall falling in the upper reaches of the catchment reaches its "outlet". In this triangular catchment, however, all rainfall falling along the longest side of the triangular catchment would reach the catchment's "outlet" at the same time, leading to more flashy floods in the downstream parts of this catchment (at the study area), compared to an elongated one. Thus, the hydrographs of triangular (and rounded) catchments would reach a peak discharge (Q in m^3/s) long before the peak in a more elongated catchment of the same size and the same general area would be reached. The time from the beginning of the flood to the end thereof is therefore shorter in a triangularly shaped catchment. Thus, a storm would produce a flood, which is overall shorter in duration, but its peak flow rate (Q) would be higher. In the case of elongated catchments, the flood would have a longer duration, but would not have as high a peak flow as the rounded one.

There is a measure of mitigation to the above in that virtually the entire catchment is rural, particularly its upper reaches where the highest rainfall would occur. This well-vegetated part of the catchment would (to a certain extent) attenuate the floodwaters a little. All this information was incorporated the flood model.

1.4 Elevation information used

The 0.5-m contour lines used for this project were produced using the survey data supplied by your client. AED was under the impression that the survey was produced using LiDAR surveying techniques, however, after working with the contour lines, it appeared that the survey might have been done using photogrammetry surveying techniques. Generally speaking, AED considers LiDAR a very good technique for surveying elevations at, and around wetlands and watercourses, where it is not always possible for surveyors on foot to gain access. Furthermore, being very narrow laser beams, a LiDAR survey can penetrate all but the densest of vegetation, providing true ground elevations under tree covers. Trees and dense vegetation are often associated with rivers and wetlands.

Photogrammetry cannot penetrate vegetation and is less suitable for our purposes, particularly under trees lining watercourses. The "jumbled" contour lines in the lower part of the Eerste River at the study area gave the impression that the survey might have used photogrammetry and not LiDAR. It must be acknowledged that the accuracy of the flood lines produced by AED is directly related to the accuracy of the contour lines used. AED, however, considers the survey data as being suitable for modelling the flood lines of the Eerste River.

1.5 Background to Flood Lines, Floodplains and Areas of Inundation

Note: Although this section mostly refers to a 100-year flood, it is true for floods with any return interval (RI).

A 100-year flood is a flood event that has a 1% probability of occurring in any given year. The 100-year flood is also referred to as the 1% flood, since its annual exceedance probability is 1%, or as having a return interval of 100-years. The 100-year flood is generally expressed as a flow rate (m³/s). Based on the expected 100-year flood flow rate in a given stream or river, the flood’s water level can be mapped as an area of inundation. The resulting floodplain map is referred to as the 100-year floodplain, which may be very important in how close to the stream buildings or other infrastructure/activities are allowed.

A common misconception exists that a 100-year flood is likely to occur only once every 100 years. In fact, statistically, there is an approximately **63.4 %** chance of one 100-year floods occurring in any given 100-year period. The **Probability** (***P_e***) of one of a specifically sized flood occurring during any return interval, exceeding the specifically sized flood severity, can be expressed as:

African Environmental Development
PO Box 1588
Rant-en-Dal 1751
Tel: - 083 657 0560
Fax:- 086 670 5102
E-mail: - garfield@aed.co.za
http://www.aed.co.za

$$P_e = 1 - \left[1 - \left(\frac{1}{T}\right)\right]^n$$

…where P_e is the probability, T is the return interval of a given storm (e.g. 100-year, 50-year, 20-year, etc.), and n is the number of years. The exceedance probability P_e is also described as the natural, inherent, or hydraulic risk of failure when, e.g. referring to dams, bridges, etc. However, the expected value of the number of 100-year floods occurring in any 100-year period is 1. In other words, 100-year floods have a 1% chance of occurring in any given year (P_e = 0.01), 10-year floods have a 10% chance of occurring in any given year (P_e = 0.1), 50-year floods have a 2% chance of occurring in any given year (P_e = 0.02), etc. The percent chance of an *x-year* flood occurring in a single year can be calculated by dividing 100 by *x*.

1.6 Legal Considerations

In terms of Section 144 of the **National Water Act** of 1998 (Act 36 of 1998), a flood line, representing the highest elevation that would probably be reached during a storm with a return interval of 100 years, must be indicated on all plans for the establishment of townships. The term, "*establishment of townships*" includes the subdivision of stands or farm portions in existing townships, if the 100-year flood lines are not already indicated on these plans, or when the land-use category of a particular portion of land is changed.

The purpose of this section of the act is to inform developers/landowners or residents/occupants/tenants/land-users of the dangers of flooding.

The earlier version of the **National Building Regulations and Building Standards Act** (Act 103 of 1977) referred to the 50-year flood lines. The newer version refers to areas that are "*prone to flooding*". Due to this discrepancy between different legislations, most municipalities now require both the 50- and 100-year flood lines to be indicated on plans, even though the National Water Act supersedes the National Building Regulations and Building Standards Act.

According to the Constitution of South Africa no legislation can supersede another. National, Provincial and Local Legislation (By-Laws) are all on the same level and must be adhered to. Similarly, older and newer legislation pertaining to the same entity are also on the same level.

Being the higher of the two, the 100-year flood lines are obviously the safer of the two flood line sets. AED is confident that there would not be any additional benefit to also model the 50-year flood lines. The 100-year flood lines are appended as DXF and Shape and KMZ files in ***Appendix 1***.

2 Design Storm and Flood Line Modelling

2.1 Methodology Flood Lines

The determination of flood lines is done in two steps, **1)** modelling a succession of "*design storms*", each of them with a specific duration (1 hour, 2 hours, 24-hours, etc.) each producing a particular discharge in m³/s and, **2)** routing the highest discharge produced in Step 1 through cross sections across representative reaches of the rivers/streams at a study area, which then assigns an elevation on each side of the centreline (℄) of the stream to which the floodwaters would rise at that particular cross section. The flood lines are then drawn using these elevations at the cross sections as guides. The flood lines indicate the area that will be inundated during a 100-year flooding event at the study area.

The first part of the process comprises the modelling of a succession of "design storms" with a statistical RI of 100-years and durations ranging from 1 to 24 hours, falling over the catchment of the river at study area during a single rainfall event with a RI of 100-years.

Where a catchment is smaller that approximately 50 Km², design storms are derived using a deterministic approach, as opposed to the purely statistical methods used for larger catchments (i.e. catchments larger than ~100 Km²). This is done as a result of the difficulty of extrapolating the frequency analyses of peak discharges and experience envelope diagrams for small areas, as the range of enveloped values becomes extremely wide as the catchment area decreases. Another reason is the problem of attempting to assign recurrence intervals to these small experience envelopes. Hence, for catchments under ~50 Km², the acceptable procedure is to employ the original Rational Method (Q = CIA), an Empirical Method developed by the HRU, and the Amended Rational Method, as researched, designed and published in Reports No. 1/72, 'Design Flood Determination in South Africa', 1972 and 1/74, 'A Simple Procedure for Synthesizing Direct Runoff Hydrographs' 1974, produced by a joint venture between the CSIR and the Hydrological Research Unit (HRU) (a division of the Department of Civil Engineering at the University of the Witwatersrand). The discharge is then derived using a weighting system linked to the surface area of the catchment.

Where a catchment is larger that ~100 Km², the standard statistical unit-hydrograph HRU model, as described by Bauer & Midgley (Bauer & Midgley, *1974*), is used. Storms with durations of 1 to 24 hours are synthesised, using modelling techniques described in both Reports No 1/72 "Design Flood Determination in South Africa" (*Midgley, 1972*) and No 1/74 "A Simple Procedure for Synthesizing Direct Run-off Hydrographs" *(Bauer & Midgley, 1974)* and using software developed by AED. Using the modelling techniques described by these documents, direct run-off hydrographs are then derived for the different storm durations. The hydrographs that produced the highest discharge is selected.

If the catchment is between 50 and 100 Km², both methods are done and the one producing the greater of the two discharges is selected as the preferred model.

With a catchment of 330.366 Km², the flood discharge in the Eerste River at the study area was derived using the standard statistical unit-hydrograph HRU model (for larger catchments), as its catchment is significantly larger than 100 Km². The highest discharges (produced by a 7-hour storm) for the Eerste River catchment was selected to plot the 100-year flood lines.

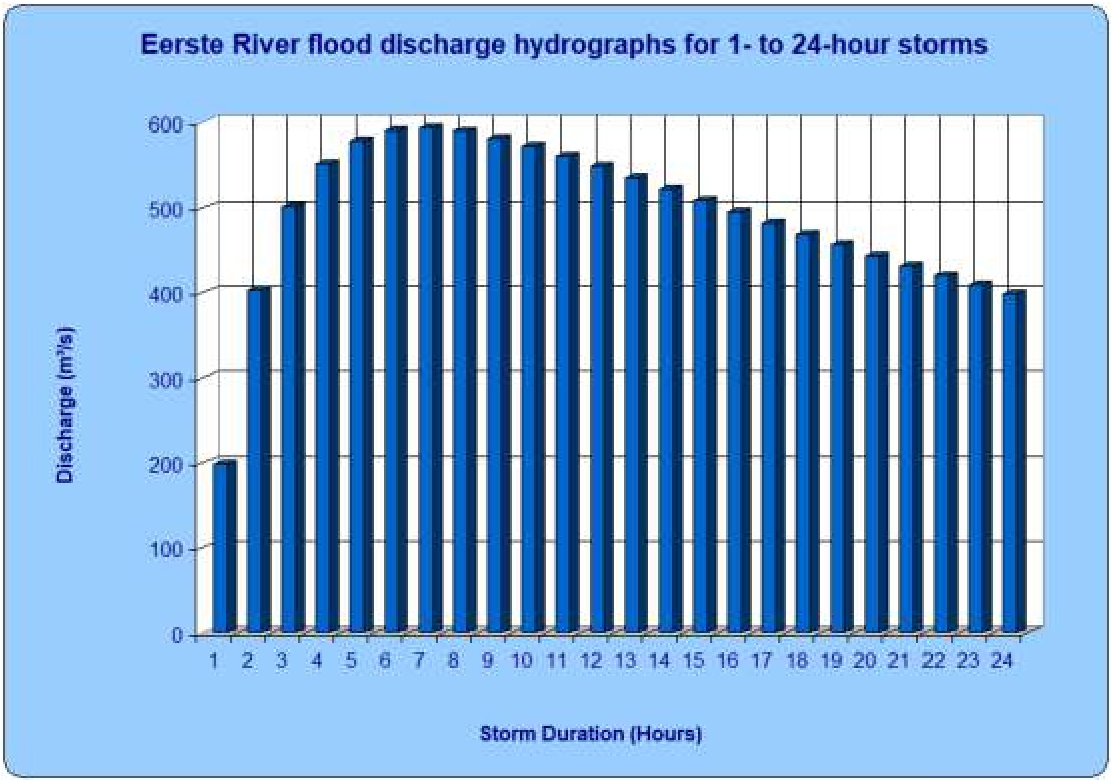
In the second part of the process, the discharges (flow - Q, in m³/s), produced by the design storm that produced the highest discharge, were routed through cross sections across the Eerste River, using Mannings theory for Open Channel Flow (*Chow, 1959*) and using software developed by AED. Ten cross sections were plotted across representative reaches of the Eerste River of which Cross Section 5 was plotted as close as possible to the Annandale Rd Bridge on its upstream side, Cross Section 6 was plotted across the bridge and Cross Section 7 immediately downstream of the bridge. These three cross sections would also be used as Cross Sections 1, 2 and 3 in the Bridge Backwater Model. The Cross Sections are shown in ***Figure 4***.

2.2 Methodology for Bridge Backwater Calculations

AED used the methodology described in the publication, "Hydraulics of Bridge Waterways" (Bradley, 1978) to determine the backwater produced by the Annandale Rd Bridge over the Eerste River.

2.3 Results

Table 1 summarises the discharge of the Eerste River at the study area for the 100-year return interval, using the aforementioned hydrological methodology.



Graph 1: The hydrographs for a 100-year flood in the Eerste River at the study area. A storm of 7 hours in duration will produce the highest peak flow rate (of 592.83 m³/s)

The 100-year flood lines were plotted through the 10 cross sections across the Eerste River, and are shown in ***Figures 3*** to ***7***.

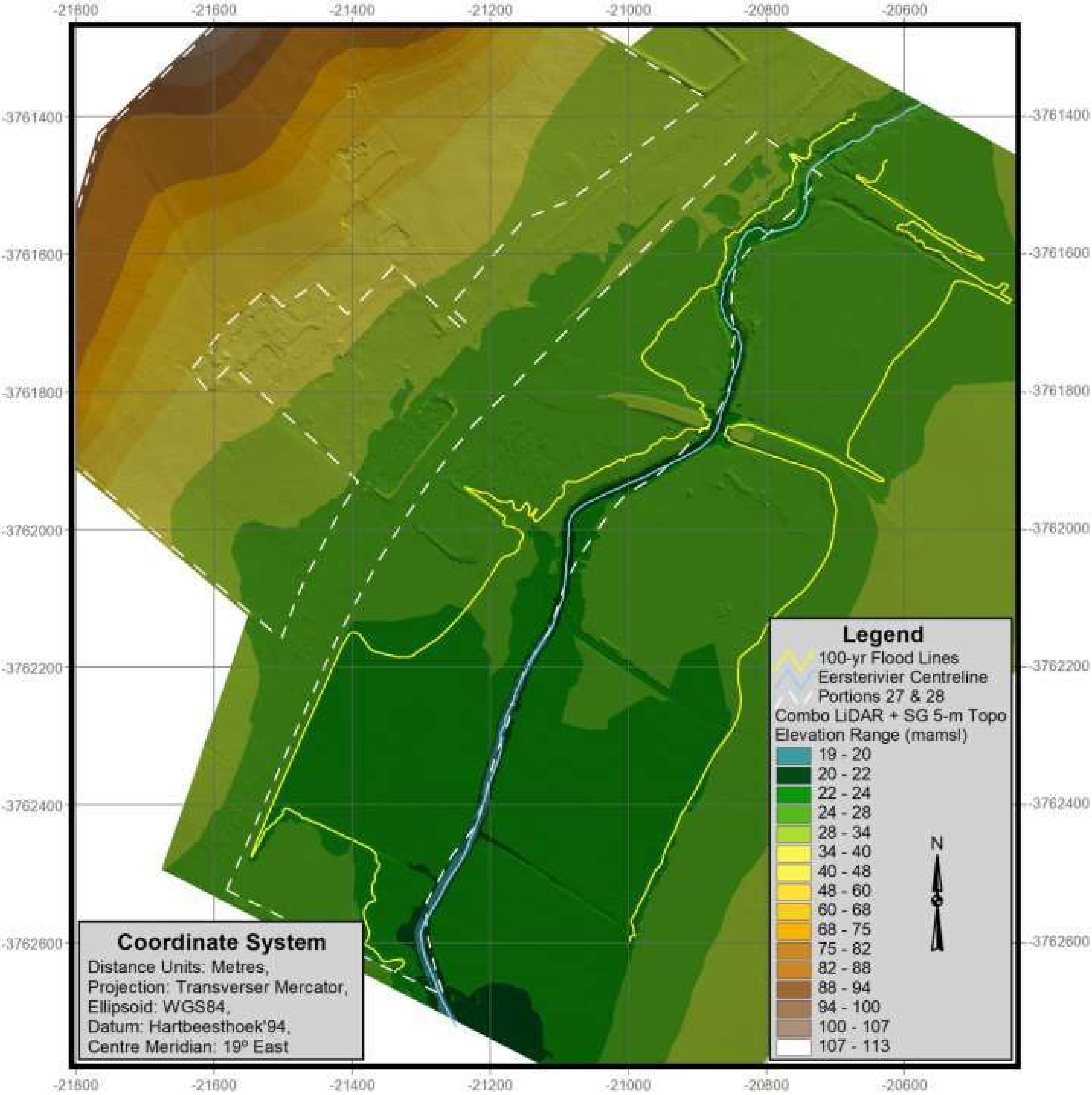


Figure 3: The 100-year flood lines of the Eerste River at the study area on a backdrop of the local topography, indicates a very wide floodplain due to the extremely low gradient of the river at this site

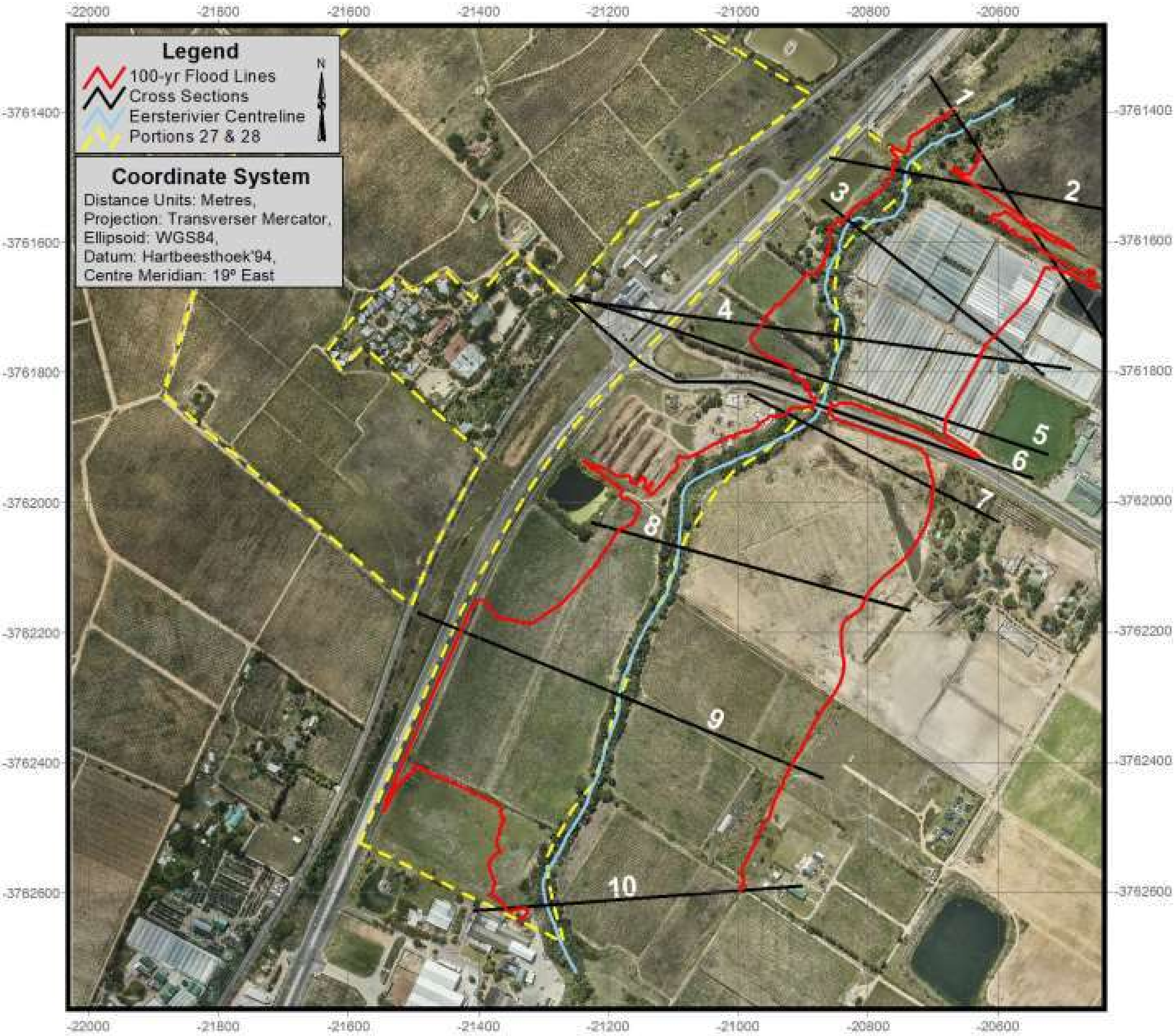


Figure 4: The 100-year flood lines of the Eerste River at the study area on a backdrop an orthophoto recently surveyed by the client's surveyors, indicates a very wide floodplain due to the extremely low gradient of the river at this site. It can be seen from both this figure and Figure 3 that, the Annandale Rd Bridge will not be overtopped

Also shown are the 10 cross sections across the Eerste River used in the hydrological model

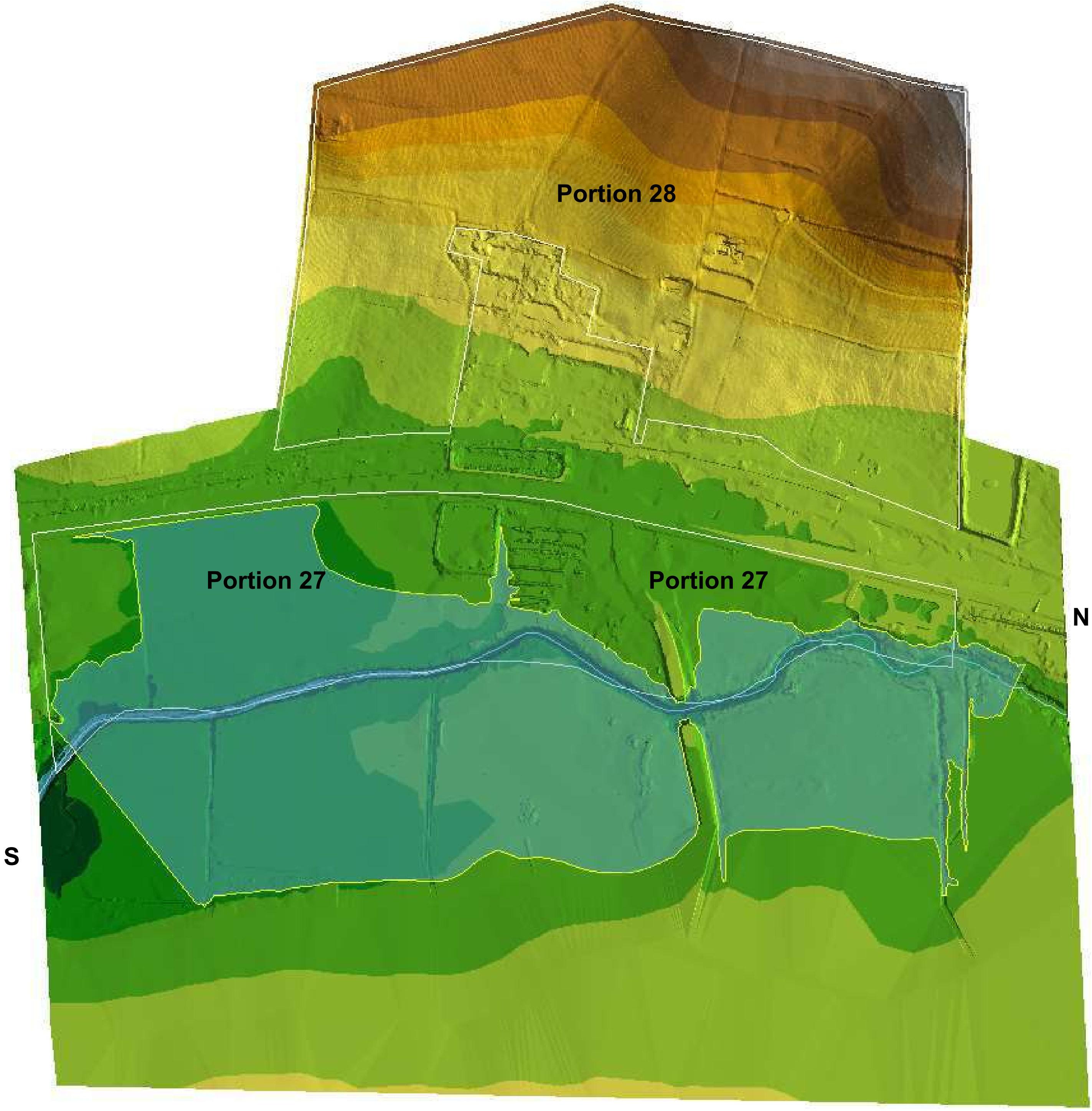


Figure 5: The 100-year floodplain of the Eerste River at the study area

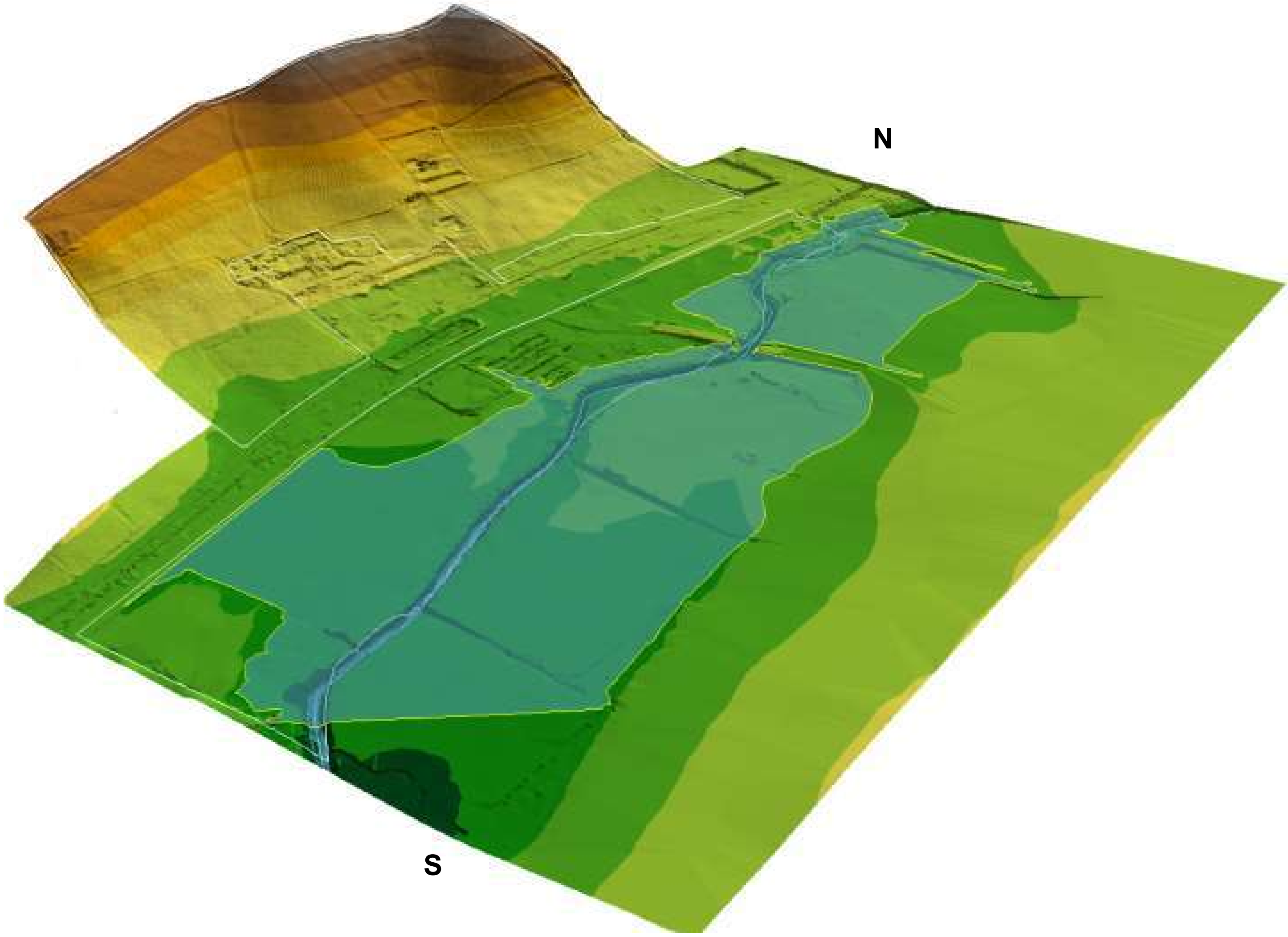


Figure 6: A 3D DTM showing the areas of inundation produced by 100-year storms falling over the catchments of the Eerste River. Please note that a **vertical exaggeration factor of 2.5x** was used to increase the sense of depth, i.e. the topography appears 2½ times deeper or higher.

As can be seen, the bridge at the Annandale Road crossing will not be overtopped during a 100-year flood

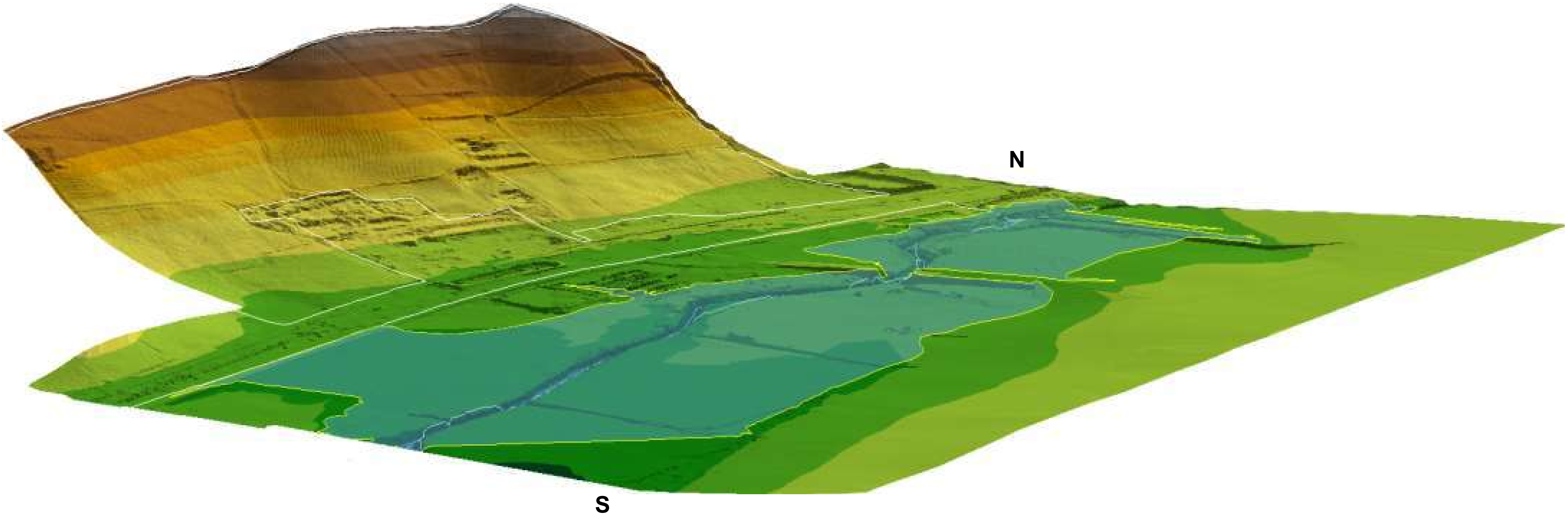


Figure 7: A 3D DTM showing the areas of inundation produced by 100-year storms falling over the catchment of the Eerste River. Please note that a **vertical exaggeration factor of 2.5x** was used to increase the sense of depth, i.e. the topography appears 2½ times deeper or higher. As can be seen, the bridge at the Annandale Road crossing will not be overtopped during a 100-year flood

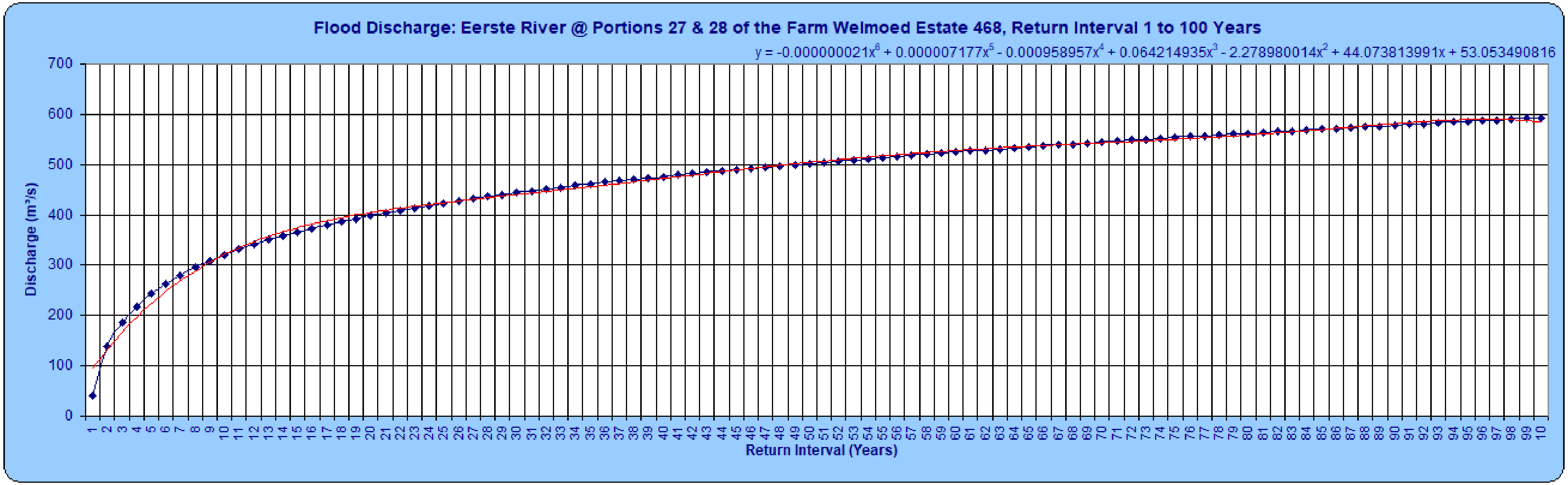
2.4 Discharge off storms with RIs of 1- to 100-years falling over the catchments at the study area

Graph 1 summarises the *RI vs. Discharge* relationship for the Eerste River at the study area. The graph also contain its formula:

Eerste River:

$y = -0.000000021x^6 + 0.000007177x^5 - 0.000958957x^4 + 0.064214935x^3 - 2.278980014x^2 + 44.073813991x + 53.053490816$

The discharge can either be read off the graph directly, or the formula can be used to determine the discharge for a particular RI more accurately. It must further be noted that the graph and formula is specific to this particular catchment and cannot be used anywhere else.



Graph 2: The discharge off storms with RIs ranging from 1- to 100-years, falling over the catchment of the Eerste River.

From the red regression line (trend line) is can be seen that the formula is only accurate after about the 8-year mark. After that the red regression line plots almost perfectly on top of the blue line of the actual graph

2.5 Annandale Road Bridge backwater calculations

There is a bridge across the Eerste River at the study area (bridge No 0618). It was constructed in 2018. It was considered highly likely that this bridge would produce backwater during a 100-year flood, which could increase the extent of the 100-year floodplain at the study area (Portion 27). This bridge is shown in ***Photos 1*** to ***3***.

For this reason it was necessary to also model the bridge backwater for this bridge under the 100-year flood flow conditions.

From the photos it can be seen that this bridge consist of a single rectangular portal, with no piers under the bridge. The surveyors, *Neil Woodin Surveys*, surveyed the bridge on 09/05/2023. A detailed drawing of this survey (in PDF format) is attached in ***Appendix 3***, the dimensions of which were used in our backwater model. The CAD file of this drawing is also available, if required.

The leading edges of the abutments were finished off in a rectangular format, i.e. presenting the flowing water with a flat surface to get around (rather than a angled surface, channelling the floodwaters into the portal).

The backwater was modelled using two different techniques. The first was done using Bradley's backwater methodology described in the publication, "*Hydraulics of Bridge Waterways*" (*Bradley, 1978*), and using software developed by AED, while the second was done where the portal was modelled as an open channel (this could be done, as there were no piers). Both methods produced results roughly within the same ballpark values, but the Bridge Backwater method, being the more reliable method, produced a higher backwater elevation and these values were preferred.

During a 100-year flood, the maximum elevation of the floodwaters just before the water surface begins to drop away when entering the bridge at L*, as described in Bradley's publication, differed by 42 cm between the two methods (elevations of 25.52- and 25.94-mamsl for the Open Channel and the Bridge Backwater methods respectively). L* is the distance from the leading edge of the bridge to the maximum backwater.

Bradley's calculations indicated that the water level would begin dropping away some 23.287 m before reaching the bridge intake (L* = 23.287 m). The difference between the inlet water elevation of the bridge and its water outlet elevation (Δh) would be 1.378-m (i.e. the drop in water level before and after the bridge).

Interestingly, the water level on the outflow side of the bridge is at a lower elevation as that at Cross Section 7 further downstream. This implies that, due to the higher velocity of the water flowing through the bridge portal (7.598 m/s), a hydraulic jump would occur immediately downstream from the bridge where the water level would abruptly rise from 24.65- to 25.65-mamsl, a 1-m jump of exactly 1 metre. This scenario is shown in ***Figure 8***.

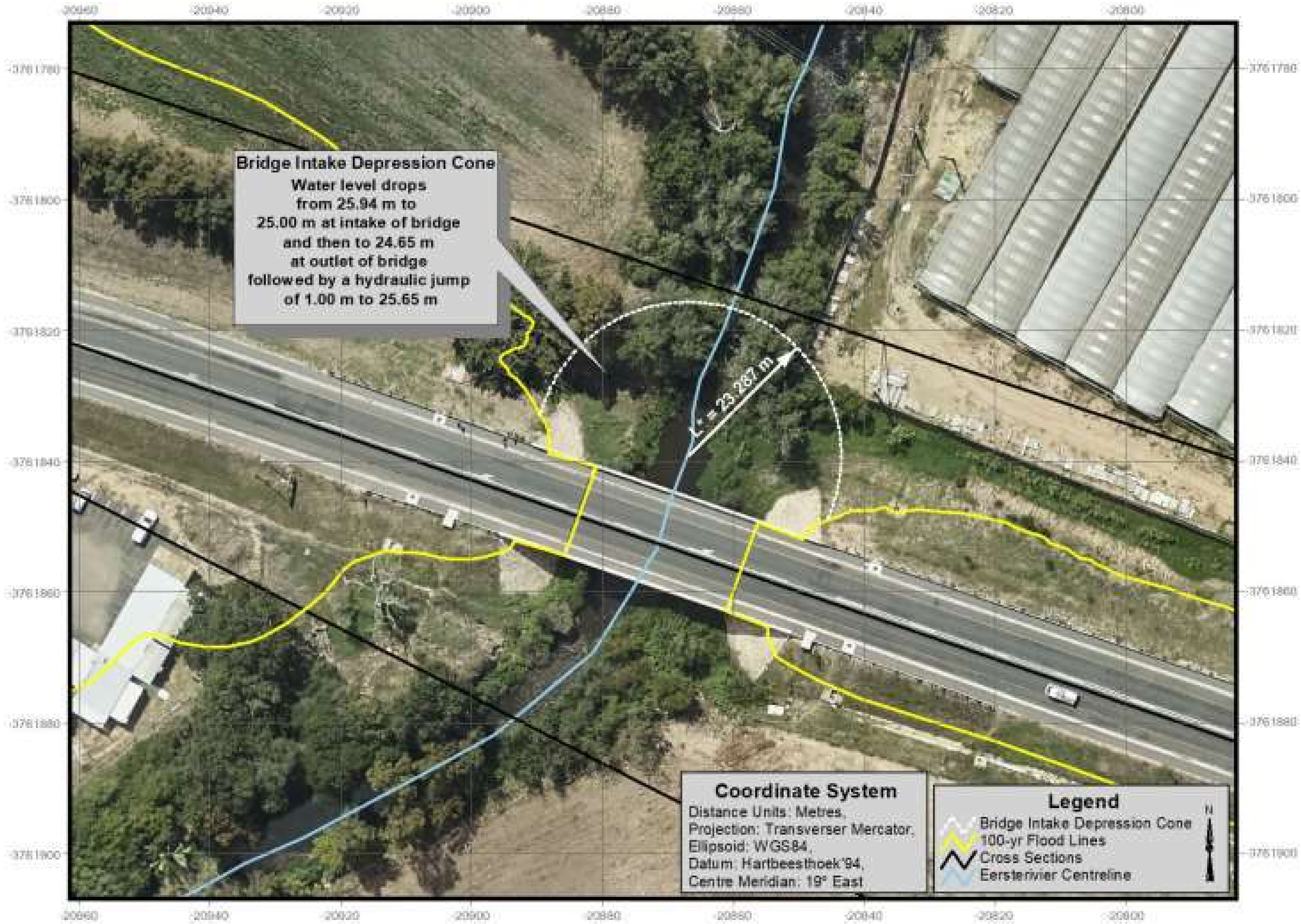


Figure 8: The depression that would form at the intake side of the bridge (dotted line) will begin forming some 23.287 m upstream from the bridge. The distance is referred to as L*, the distance to maximum backwater. At the outlet of the bridge, a 1-m hydraulic jump would occur

A hydraulic jump is a phenomenon, which is frequently observed in open channel flow such as rivers and spillways. When liquid at high velocity discharges into a zone of lower velocity, a rather abrupt rise occurs in the liquid surface. The rapidly flowing liquid is abruptly slowed and increases in height, converting some of the flow's initial kinetic energy into an increase in potential energy, with some energy irreversibly lost through turbulence to heat. In an open channel flow, this manifests as the fast flow is rapidly slowed down and piled up on top of itself, similar to how a shockwave forms. It was first observed and documented by Leonardo da Vinci in the 1500s.

Our calculations confirmed that the bridge would not be overtopped, implying that it was constructed to allow a 100-year flood to pass under it without being overtopped.

2.6 Comments relating to the Flood Lines

2.6.1 Extent of the flood lines at the study areas

The floodplain of the 100-year flood in the Eerste River would impact on Portion 27, but not on Portion 28 at the study area. Due to the very low slope of the river at the study area, the floodplain would spread out very wide on both sides of the stream's centreline.

In most instances, the 100-year flood lines exceed the 32-m buffer, measured from the centre of the stream (as this is a reasonably narrow stream channel under normal flow conditions). In some parts, the 100-year flood line on Portion 27 and the 32-m line almost run on top of each other. Refer to ***Figure 9*** for details of the 32-m buffer (black & white lines) in relation to the 100-year flood lines (yellow lines).

The backwater created by the bridge at Annandale Rd played a role in elevating the 100-year floodplain upstream from the bridge to a point roughly halfway between Cross Sections 4 and 5 (refer to ***Figure 4*** for the cross sections).

2.6.2 Units used in this report

All drawings were produced using the following parameters: **Distance Units**: Metres, **Projection**: Transverse Mercator, **Datum**: Hartbeesthoek'94, **Ellipsoid**: WGS84 and the **Centre Meridian** (Longitude of Origin or "LO"): 19º East (i.e. WG19º).

The GIS program, *ArcView 3.2a* and its extension, *3D Analyst*, were used to produce all the drawings in this document. Please note the **X** and **Y** coordinates in the drawing have been allocated positive values to north and east and negative values to south and west.

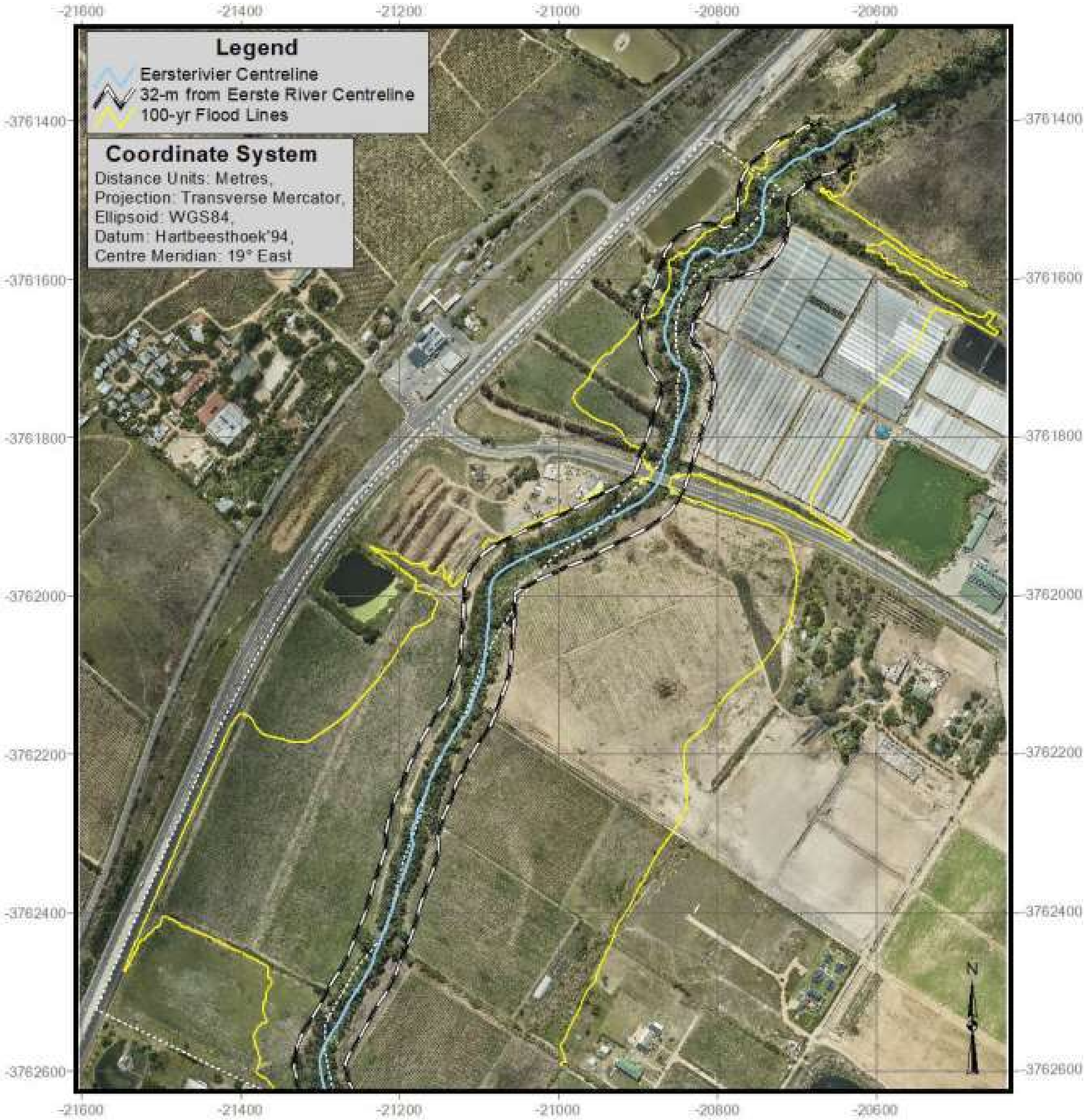


Figure 9: The 32-m buffer area (32-m from the centreline of the Eerste River) in relation to the 100-year flood lines

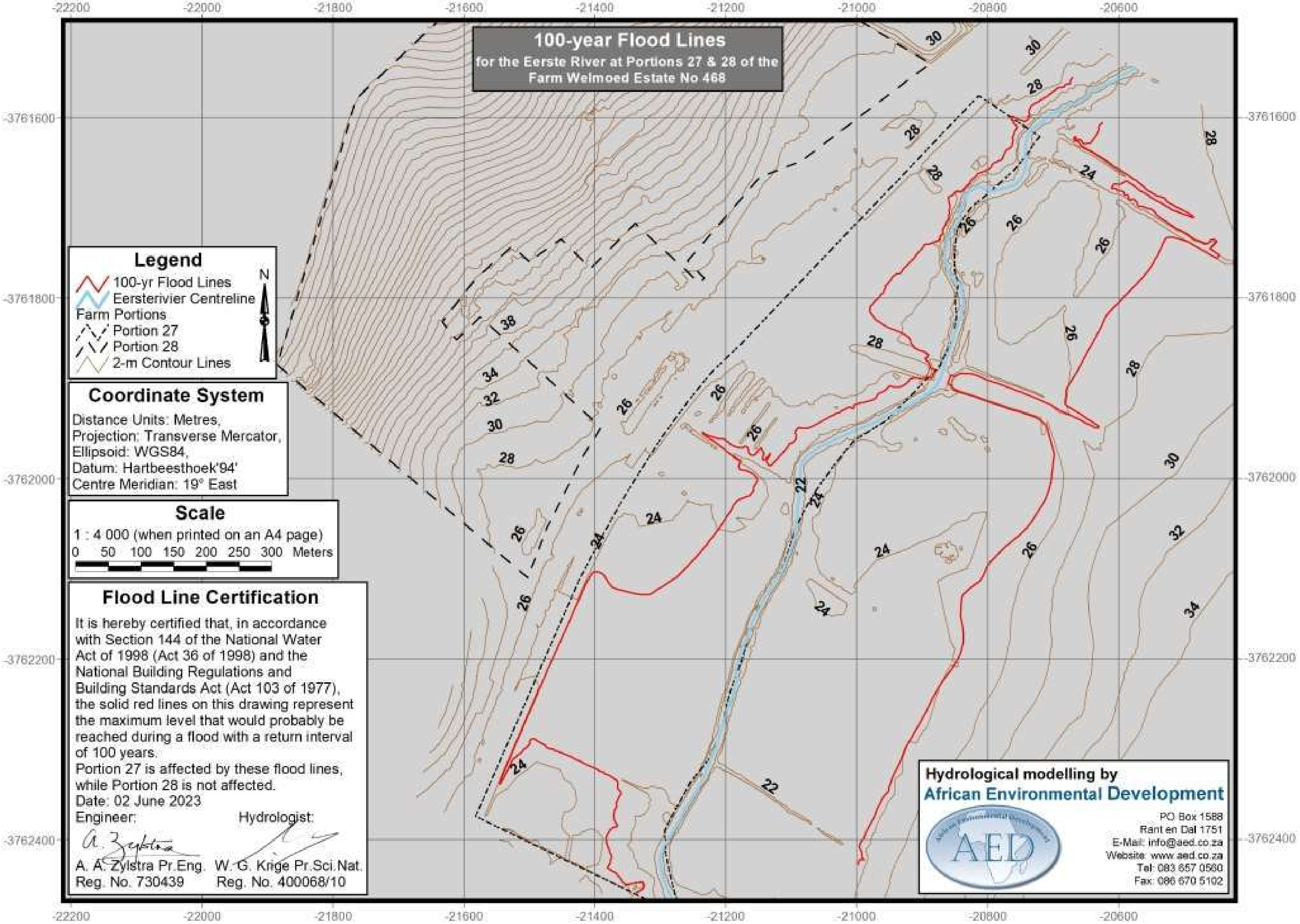


Figure 10: The 100-year flood lines of the Eerste River (red lines).

Although 2-m contour lines are shown in this drawing, AED actually used 0.5-m contour lines to model the flood through the cross sections and through the bridge.

This is a low-resolution copy of the high-resolution certified drawing appended to this document in ***Appendix 1***

African Environmental Development
No 129 Malmani Road
Sterkfontein Country Estates
Krugersdorp

African Environmental Development
PO Box 1588
Rant-en-Dal 1751
Tel: - 083 657 0560
Fax:- 086 670 5102
E-mail: - garfield@aed.co.za
http://www.aed.co.za

3 Certification of Methods Used

100-Year Flood Lines:

The Eerste River at Portions 27 and 28 of the farm Welmoed Estate No 468 District Stellenbosch, Western Cape Province, RSA

05/06/2023

TO WHOM IT MAY CONCERN,

This is to certify that the 100-year flood discharge for the above-mentioned watercourse at the above site were derived using methods described in the *Report No 1/74 "A Simple Procedure for Synthesizing Direct Runoff Hydrographs"* of the *Hydrological Research Unit*, a division of the Department of Civil Engineering of the University of the Witwatersrand, and using software developed by AED.

The 100-year return interval storms were synthesized from direct run-off hydrographs using methods described in *Report No. 1/72 "Design Flood Determination in South Africa"* of the same unit and also using software developed by AED.

Both Reports 1/72 and 1/74 were developed as a joint venture between the CSIR and the University of the Witwatersrand and are considered to be accurate methods, particularly for use under South African rainfall conditions.

The flood flow rates, determined in terms of paragraph 1, were then routed through cross-sections, plotted across the stream, using Mannings theory for open channel flow and using software developed by AED. The design flood through these cross sections produced the flood elevations for the 100-year return interval storm, as indicated in the accompanying CAD files.

The Bridge Backwater for the Annandale Road Bridge was modelled using modelling techniques described in the publication, "*Hydraulics of Bridge Waterways*" by Joseph N Bradley (Bradley 1978) and using software developed by AED.

The 0.5-m contour lines were produced using the elevation information acquired from the client and was surveyed on 10/03/2023. Premier Mapping conducted the survey. The survey produced contour lines that were sufficiently accurate for the modelling of the 100-year flood lines in this stream.

A. A. Zylstra
Pr.Eng. (Reg. No.730439)

W. G. Krige
Pr.Sci.Nat (Reg. No. 40068/10)

4 References

Bailey, A. K. & Pitman W. V.: (2012): Water Resources of South Africa 2012 Study (WR2012) Version 1 Water Research Commission Project No K5/2143/1

Bauer, S. W., Midgley, D. C.; (1974) CSIR/University of the Witwatersrand Report No. 1/74, 'A Simple Procedure for Synthesizing Direct Runoff Hydrographs'

Bradley, Joseph. N.; (1978) 'Hydraulics of Bridge Waterways'

Chow, Ven Te; (1959). 'Open Channel Hydraulics', McGraw Hill Book Company, Re-issue 1998

Midgley, D. C.; Pitman, W, V.; Middleton, B. J. (1994). 'Surface Water Resources of South Africa', Water Research Commission Report No. 298/94

Midgley, D. C.; (1972) CSIR/University of the Witwatersrand Report No. 1/72, 'Design Flood Determination in South Africa'

Middleton, B. J.; Bailey, A. K. (2005): Water Resources of South Africa, 2005 (WR2005) Version 1. Water Research Commission Project No. K5/1491

SANRAL; (2007); 'The South African National Roads Agency Limited Drainage Manual' 5th Edition, 2nd Print, SANRAL Publication

SANRAL; (2013); 'The South African National Roads Agency SOC Limited Drainage Manual Application Guide' 6th Edition, Fully Revised and Extended, SANRAL Publication

---

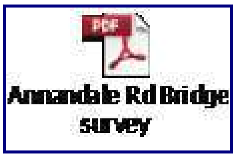
Appendix 1: CAD Files and Certified Drawing

Please double-click on the above icon to open the zipped folder containing the CAD file in DXF and Shape file formats, as well as images of the certified drawings in jpeg file format

Appendix 2: Excel Workbook Area-weighted Catchment Mean Annual Precipitation calculations

Please double-click on the above icon to open the file

Appendix 3: Survey drawing of Annandale Rd Bridge



Please double-click on the above icon to open the file

ANNEXURE C

Subdivision (Erf) reference	Land use	Phase	Comment
1	Private open space, street, place of assembly	1	
2	Educational, school	1	
3	Commercial, business	1	
4	Residential, business, mixed-use	1	
5	Private open space, street	2	
6	Private open space, outdoor sport, stormwater management	6	
7	Residential, business, mixed-use	2	
8	Residential, business, mixed-use	3	
9	Private open space, street	4	
10	Residential, business, mixed-use	4	
11	Place of assembly, business, mixed-use	4	
12	Residential	5	
13	Private open space, outdoor sport	6	
14	Private open space, street	7	
15	Residential	7	
16	Residential	8	
17	Private open space, street	9	
18	Private open space, outdoor sport, stormwater management	9	
19	Residential	10	
20	Residential	10	
21	Residential	11	
22	Private open space, outdoor sport, stormwater management	9	
23	Residential	12	
24	Private open space, outdoor sport	14	
25	Residential	13	
26	Residential	19	
27	Private open space, street	15	
28	Residential	16	
29	Residential	17	
30	Residential	18	
31	Private open space, outdoor sport, stormwater management	15	
32	Private open space, outdoor sport	17	
33	Private open space, outdoor sport	20	
34	Private open space, street	21	
35	Private open space, street	21	
36	Residential	22	
37	Residential	23	
38	Residential	24	
39	Residential	25	
40	Residential	26	
41	Residential	27	
42	Private open space, outdoor sport, street	28	
43	Residential	29	
44	Residential	30	
45	Private open space, outdoor sport, street	31	
46	Residential	32	
47	Private open space, outdoor sport, street	33	
Remainder Road	Public road	1	

PORTION 28 LAND USE TABLE

Site area		45.48 Ha	
Land use		**Area**	**Units**
	School Component (A2)	1.78 Ha	
	Commercial (A1,B5)	0.50 Ha	
	Clubhouse Component(B5)	0.18 Ha	
	Mixed Use Component (B1-4)	0.7 Ha	515 units
	Residential (@80 du/ha)	5.88 Ha	
	Residential (@40 du/ha)	8.74 Ha	355 units
	Alottment Villas	10.31 Ha	14 units
	Total (excluding areas below)	**28.09 Ha**	**884 units**
	Detention & SW area	1.15 Ha	
	Indigenous slopes	6.43 Ha	
	Roads & squares	4.86 Ha	
	Private open space	4.04 Ha	

Block Plan (PORTION 28) REV 05.2 | 1:2000 (@A1) | 28.03.2024

WELMOED

URBANSTUDIO
architects & urban designers