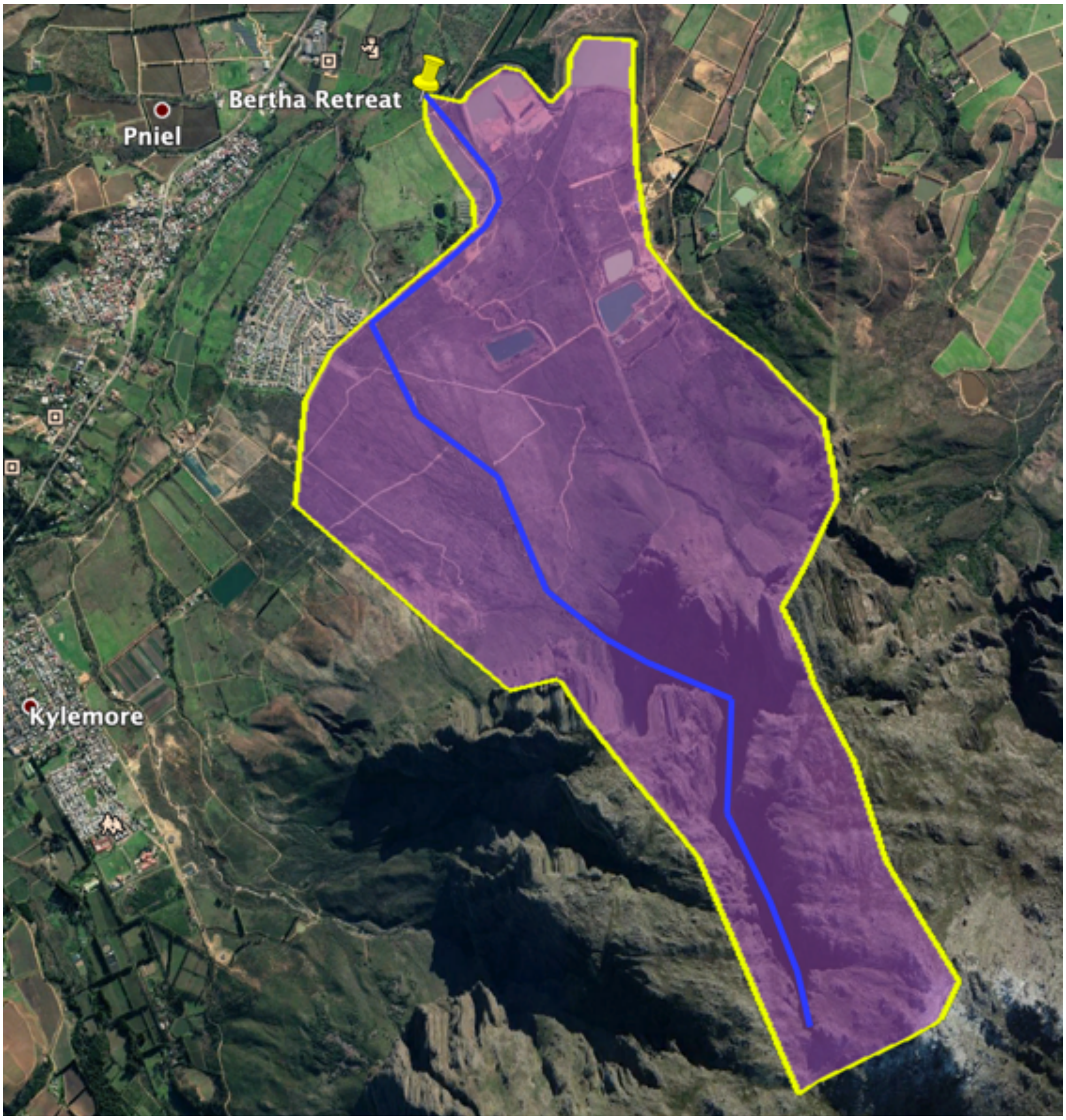



# Boschendal Estate: Proposed Bertha Retreat Development on Portion 11 of Farm 1674, Paarl

# Flood Line Study: Revised

**Mark Obree**
Consulting
Engineer
ECSA 810323

1 Goede Gift Road
Simon's Town

021 786 1494 (h/o)
083 267 0982 (cell)
mrobree@gmail.com

**25 February 2021**

# Contents



## Annexures

# 1 Introduction

Boschendal Estate propose re-developing existing accommodation to be known as Bertha Retreat on Portion 11 of Farm 1674, Paarl. Flood lines are required by Stellenbosch Municipality in terms of the requirements of the National Water Act (No 36 of 1988) prior to building plan approval. Mark Obree Consulting Engineer was appointed by Boschendal Estate to investigate flood risks and determine the extent of flooding on the site.

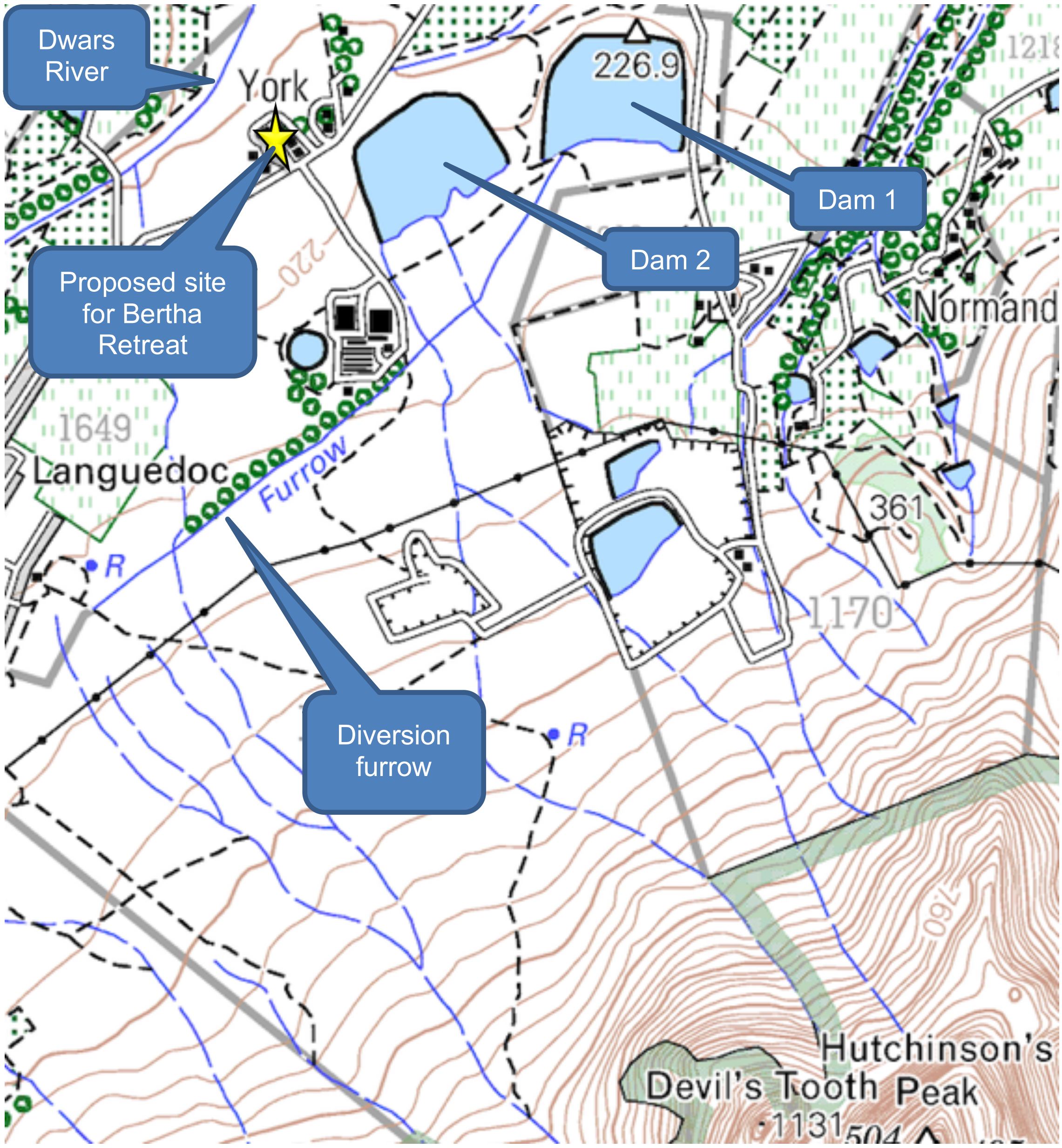
The initial flood line analysis (and report dated 13 January 2021) was based on peak flows determined using the Rational Method. As is common practice when considering runoff from small rural catchments, it was conservatively assumed that the water storage dams in the catchment were likely to be full at the time of a peak storm event, and that their ability to attenuate flows by temporarily storing the peak runoff would be small. The effect of attenuation in the dams was therefore ignored in that analysis and the peak, unattenuated, 100-year runoff was estimated to be 61m$^3$/s.

Further analysis has subsequently been carried out to examine the extent of attenuation that the existing farm dams provide, using the PCSWMM software.

This report provides a complete summary of the analysis and recommendations for dealing with the management of the attenuated peak flows in the vicinity of Bertha Retreat.

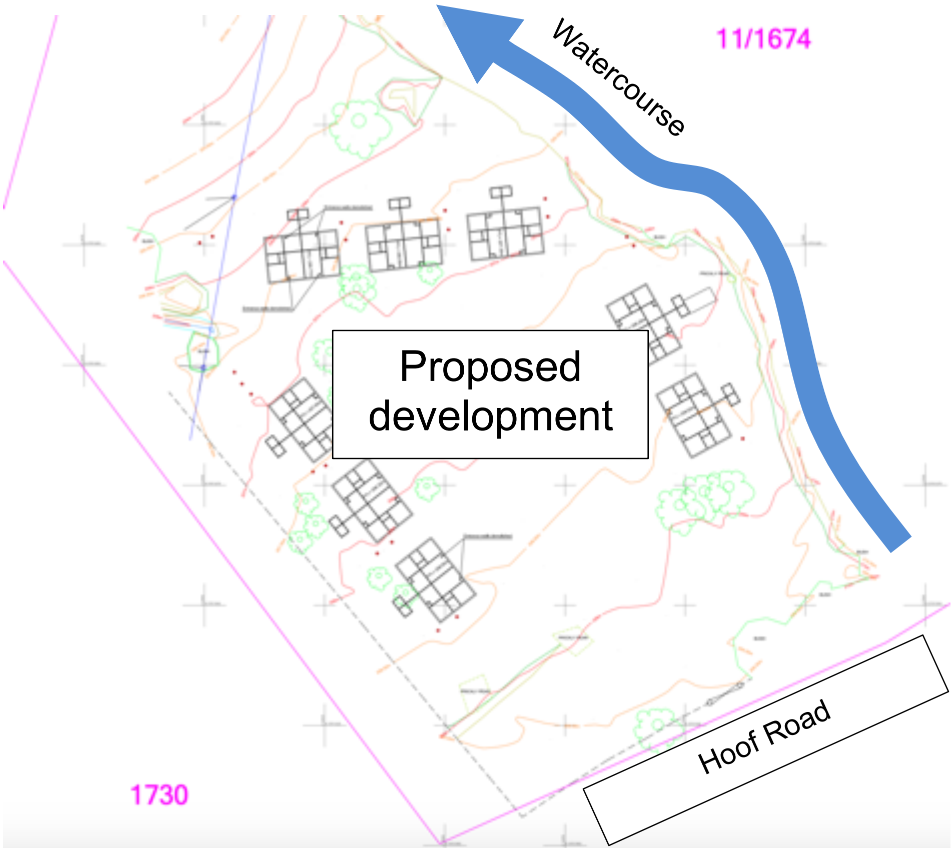
# 2 Location

The site is located on the SE side of the Dwars River, just off the R310. An unnamed stream with a catchment extending up to Hutchinson's Peak on the Hottentots-Holland Mountain range passes the site. This stream is discharges into the Dwars River (which is also known as the Banghoek River) on leaving the site. An extract from the 1:50 000 topographical map (3318DD), showing the location of the site follows.


Dwars River
York
Proposed site for Bertha Retreat
226.9
Dam 1
Dam 2
Normand
1649
Languedoc
Furrow
R
361
1170
Diversion furrow
R
Hutchinson's
Devil's Tooth Peak

## 3 Proposed Development

The layout of the proposed development is shown below.




## 4 Mapping Information

The following mapping information has been used for this study:

- Google Earth (Imagery dated 20 July 2020)
- 1 : 50 000 Topographical mapping – Sheet 3318DD (Fifth Edition – 2000), with 20m contours
- 1 : 10 000 Orthophoto Mapping – Sheet 3318DD20 (Fourth Edition 2014), with 5m contours
- Boschendal Estate aerial survey, with 1m contours
- Topo survey of the site and related watercourse, with 0.5m contours

## 5 Site Inspections

Site inspections were carried out on 15 December 2020 and 2 February 2021. The photographs included in Annexure A show some of the features inspected with comments.

Examination of the drainage system indicates that other watercourses have historically been diverted into water storage dams which ultimately discharge into the watercourse that flows alongside the site.

# 6 Hydrology

## 6.1 Catchment Area

The mapping information and site inspection confirm that runoff has been diverted from adjacent catchments towards the dams above the development which then discharge into the watercourse that runs alongside the proposed development. This results in a considerably larger catchment area than would have been the case prior to the diversions taking place.

The catchment area for the stream alongside the proposed development is presently approximately 7.9km$^2$ and is shown on the front cover of this report, while the natural catchment area for the stream was approximately 4.0km$^2$ in extent.

## 6.2 Runoff

The Rational Method was used to determine peak runoff. This method of analysis has been in use since 1851 and is still the most widely used method for determining peak flows from small catchments (up to 15km2).

Rainfall intensity for various storm durations has been obtained from Design Rainfall and Flood Estimation in South Africa (Ref 1).

Peak flows determined by the Rational Method for the present (extended) catchment, **without taking the attenuating effect of the dams** into account, are indicated below.

| Return Period (years) | 2 | 5 | 10 | 20 | 50 | 100 | 200 |
|---|---|---|---|---|---|---|---|
| Peak Flow (m3/sec) | **15.2** | **22.4** | **28.9** | **36.4** | **48.4** | **61.0** | **69.0** |

Peak flows for the original (natural) catchment, are indicated below.

| Return Period (years) | 2 | 5 | 10 | 20 | 50 | 100 | 200 |
|---|---|---|---|---|---|---|---|
| Peak Flow (m3/sec) | **8.4** | **12.4** | **15.9** | **20.1** | **26.7** | **33.4** | **38.1** |

Details of the runoff calculations using the Rational Method are included in Annexures B1 and B2.

## 6.3 Peak flow attenuation

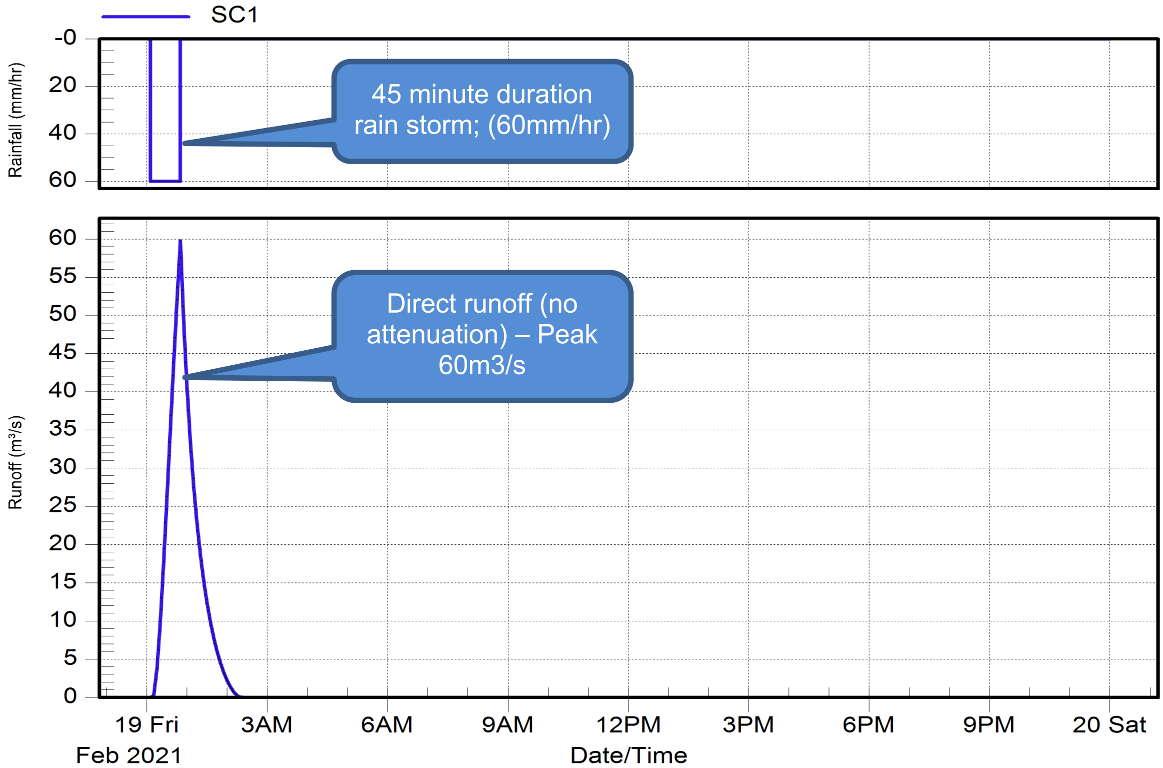
As indicated in the introduction, further analysis has been carried out to examine the extent of attenuation that the storage dams provide, using the PCSWMM software. PCSWMM is based on the United States EPA's Storm Water Management Model (SWMM). It may be described as a dynamic rainfall-runoff simulation model. The routing or hydraulics section of SWMM transports the water through the system of channels, storage ponds and weirs to the outfall. The flow generated within each component of the system is determined in pre-selected time steps throughout the simulation period.

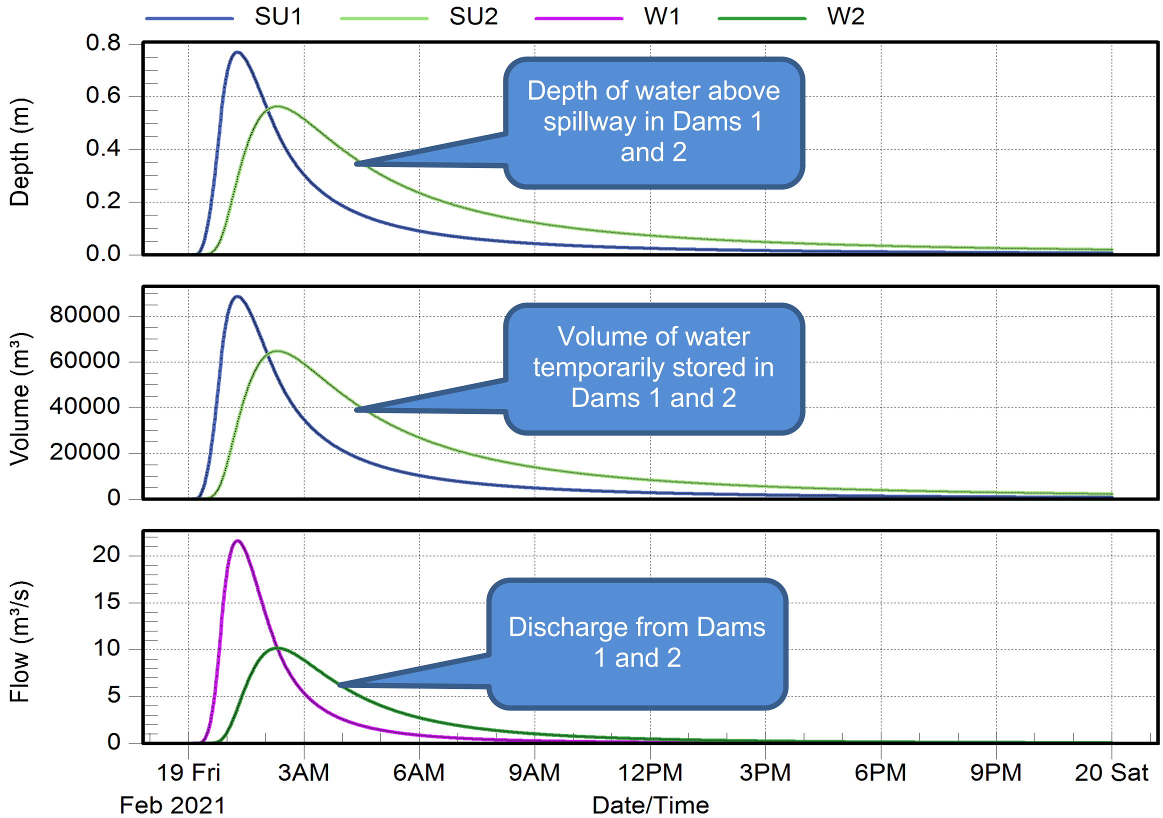
The PCSWMM model uses the runoff peak for the 100-year flood, determined using the Rational Method, and routes this flow through the storage dams No 1 and No 2 to determine the outflow that will be discharged into the stream alongside Bertha's Retreat.

The surface area of each of the storage dams (determined using Google Earth) is approximately 11.4Ha, and it has been estimated that the spillways are 15m and 8m wide respectively.

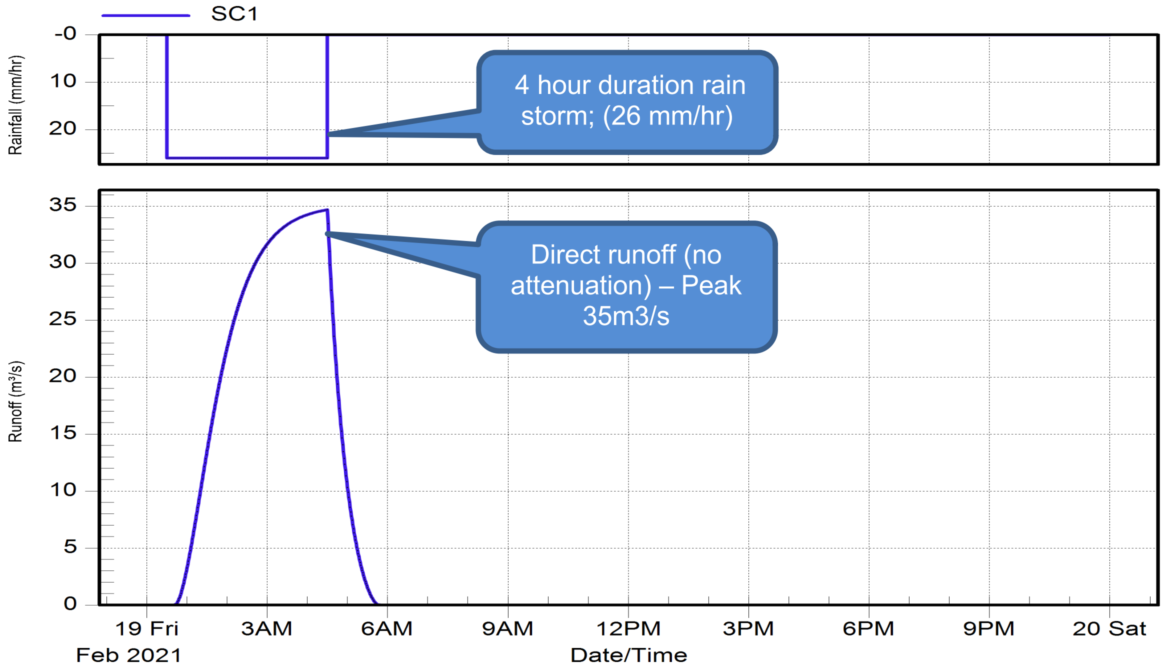
The high intensity rain event generating the highest peak runoff from the catchment, without taking the attenuating effect of the dams into account, is of a relatively short duration of 45minutes. The Rational Method has determined that this peak runoff is in the order of 61m$^3$/s. The attenuating effect of the storage dams on this short duration rainstorm is significant as shown on the pages that follow.

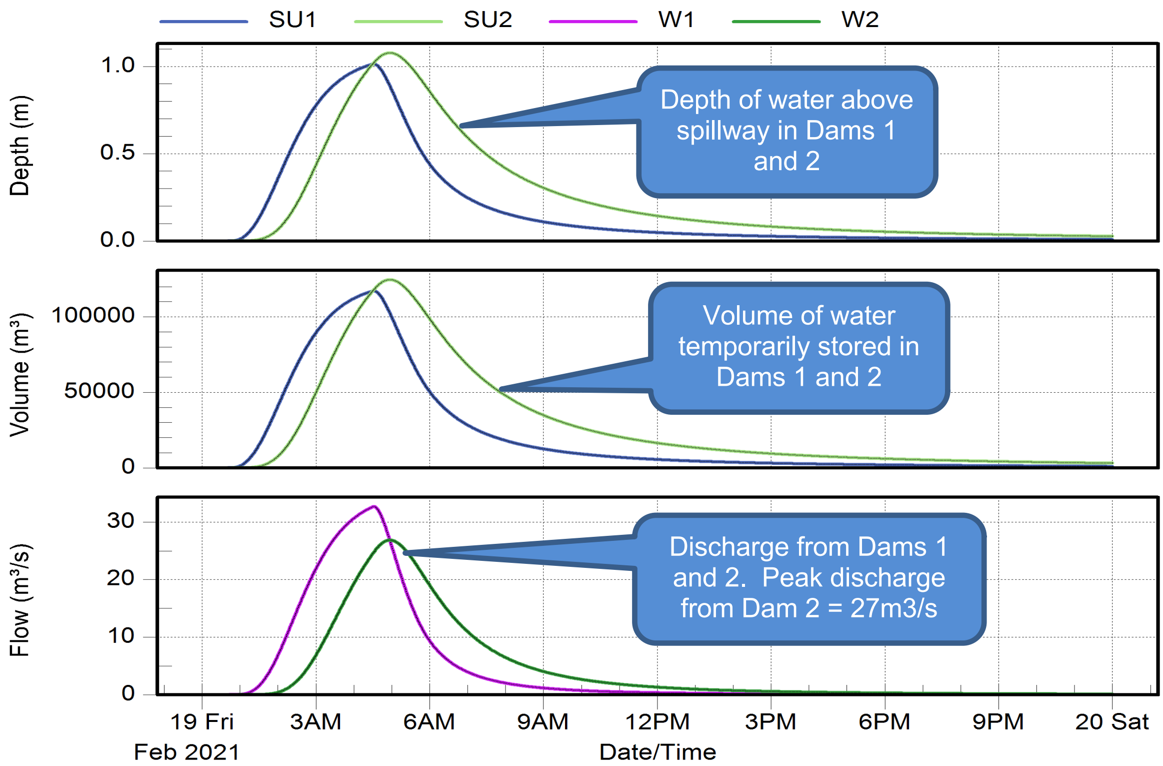
As shown below, the peak flow from the 45 minute storm is reduced by the dams from 61$m^3$/s to about 10$m^3$/s.



However, when attenuation is included in a drainage system, longer duration rain events with lower rainfall intensity become more significant in terms of peak outflow. **It was found that a storm event of about 4 hour duration resulted in the highest peak outflow of about 27m$^3$/s.** This value of flow has therefore been adopted for flood line determination at the Bertha Retreat site. Some of the information from the PCSWMM analysis for the 4 hour storm is shown below.




**As shown above, the 100-year attenuated peak flow in the stream passing Bertha Retreat is approximately 27m$^3$/s. This compares favourably to the peak flow of 33m$^3$/s that would have occurred at this point prior to the catchment being enlarged and the dams being constructed.**

# 7 Flood Risks

The Hoof Road crosses over the watercourse at the upstream corner of the development site. Twin box culverts of approximately 1.5m x 1.5m are provided at this crossing. These box culverts restrict flow, causing a build-up of water on the upstream side. Flows is excess of about 8m3/s will result in overtopping of the road on either side of the culvert, since the level of the road has been raised at this point.

Due to the topography of the area on either side of the crossing, excess flows (that do not pass through the culverts) will move overland in an uncontrolled manner, with some of this water passing through the site of the proposed development. An indication of the extent of the site that is currently affected by flooding is shown on Annexure D.

This is clearly an unacceptable situation with respect to development of the site. **It is therefore recommended that the bridge structure be enlarged or lowered to allow excess flow to pass over the structure and back into the watercourse, so as to ensure that the full flow remains in the watercourse. The flood lines for the proposed development are based on the assumption that this will be done**.

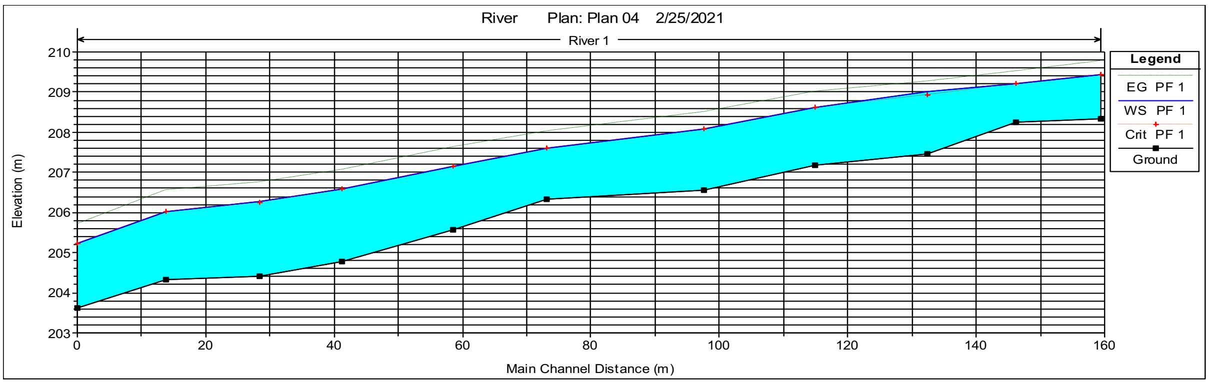
It must further be noted that the sides of the watercourse have previously been raised by the construction of longitudinal berms on either bank. This has presumably been done to contain the flow within the watercourse and prevent floodwaters from affecting the areas alongside. However, these berms vary in height, resulting in the possibility of flow escaping to the areas alongside in places where they are of insufficient height. **In order to protect the areas adjacent to the watercourse from occasional flooding it will be necessary to be repair and extend the berms in certain areas.**

The analysis of flood levels and flow velocities has been done on the assumption that the existing berms will be repaired and extended in certain areas to contain the flow within the watercourse. Since the proposed development is located on the left bank of the watercourse, it may not be necessary to provide berms to protect the areas on the right bank at this time. Nevertheless, it is recommended that the improvements to the berms on the left bank be sufficient to contain the flood, should any work be carried out on the right bank in due course. The analysis has been done on this assumption.

It is further recommended that consideration be given to the prevention of further erosion on the bed and banks of the watercourse in the lower reaches, so as to prevent further development of the erosion that is already evident, as shown in Photograph 16.

The HEC-RAS River Analysis software as developed by the U S Army Corps of Engineers was used for determination of the flood elevations.

A long section of the river profile opposite the site is shown below.



Cross-sections of the channel, showing the water surface profiles are shown in Annexure C.

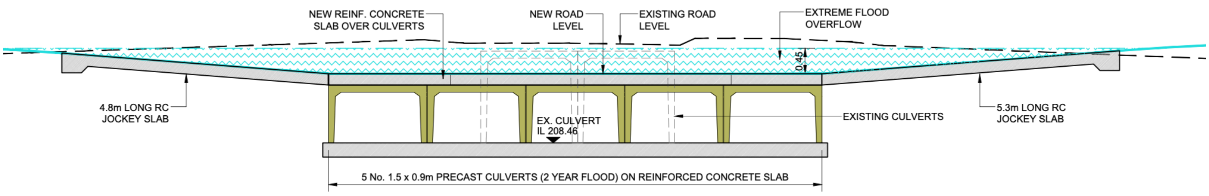
Details of the flow characteristics at each of the cross sections are shown in the table below. The 3rd column is of particular interest since it provides the energy level at each cross section.

| River Sta | W.S. Elev | E.G. Elev | Vel Chnl | Flow Area | Top Width | Froude # Chl |
|---|---|---|---|---|---|---|
| | (m) | (m) | (m/s) | (m2) | (m) | |
| -6 | 209.45 | 209.76 | 2.5 | 10.8 | 22.2 | 1.0 |
| -7 | 209.20 | 209.51 | 2.5 | 11.0 | 22.2 | 1.0 |
| -8 | 209.01 | 209.27 | 2.3 | 11.8 | 19.5 | 0.9 |
| -9 | 208.63 | 209.02 | 2.8 | 9.7 | 15.9 | 1.0 |
| -10 | 208.07 | 208.51 | 2.9 | 9.2 | 11.0 | 1.0 |
| -11 | 207.61 | 208.02 | 2.8 | 9.5 | 12.8 | 1.0 |
| -12 | 207.16 | 207.63 | 3.0 | 9.0 | 15.1 | 1.0 |
| -13 | 206.57 | 207.07 | 3.1 | 8.7 | 9.7 | 1.0 |
| -14 | 206.28 | 206.77 | 3.1 | 8.8 | 10.6 | 0.9 |
| -15 | 206.01 | 206.55 | 3.3 | 8.4 | 13.6 | 1.0 |
| -16 | 205.22 | 205.71 | 3.1 | 8.7 | 10.7 | 1.0 |

# 8 Proposed Improvements to Road Crossing

As previously discussed, the existing box culverts restrict river flow and result in water crossing the road on either side resulting in uncontrolled flooding.

The existing box culvert consists of 2 units each 1.5m wide x 1.5m high. The total cross-sectional area is 4.5m2. It is proposed that new culverts be installed consisting of 5 units each 1.5m wide x 0.9m high. The total area will then be 6.75m2. The road surface will be lowered at the culverts and raised on either side, to allow any surplus flows to pass over the road and return to the watercourse downstream.




ROAD LONG SECTION SHOWING NEW CULVERT CONCEPT
SCALE 1:50

A water depth of 100mm over the deck is the maximum depth for vehicles to safely pass. The flow passing over and through the structure in this scenario is as follows:

| | |
|---|---|
| Flow over deck | 2.6m3/s |
| Flow through | 13.5m3/s |
| Total flow | 16.1m3/s |

This exceeds the unattenuated 2-year peak flow and is therefore acceptable in terms of the guidelines provided in the Road Drainage Manual.

The flow depth over the deck that will occur in the 100-year flood, with a peak flow volume of 27m3/s is 205mm. In this scenario the breakdown is as follows:

| | |
|---|---|
| Flow over deck | 10.4m3/s |
| Flow through | 16.7m3/s |
| Total flow | 27.1m3/s |

# 9 Conclusions

The catchment area for the stream alongside the proposed Bertha Retreat development is significantly larger than what existed prior to the construction of diversion furrows and storage dams. This area has increased from about 4.0km$^2$ to the present 7.9km$^2$.

However, due to the attenuating effect of the storage dams, the peak flows alongside the proposed development are now lower than what would have occurred previously. The calculated peak flow alongside the development is now approximately 27m$^3$/s compared to the original, unattenuated flow of 33m$^3$/s.

The Hoof Road crossing of the watercourse at the upstream corner of the development is inadequate and will result in overtopping of the road on either side of the culvert. An improved low level river crossing is proposed.

Where the watercourse passes the proposed development, the sides of the channel have previously been raised to prevent floodwaters from affecting the areas alongside. However, these berms vary in height, resulting in the possibility of flow escaping to the areas alongside in places where they are insufficient. In order to protect the areas adjacent to the watercourse from occasional flooding, repair and extension of these berms is recommended.

Finally, it is recommended that consideration be given to erosion protection in the lower reaches of the stream.

\

# References

Design Rainfall and Flood Estimation in South Africa by J C Smithers and R E Schulze. WRC Report No: K5/1060, December 2002

SANRAL: Road Drainage Manual. 5th Edition.

# Annexure A:  Photographs

Photographs of some of the drainage features captured at the time of the site inspection are included below, together with relevant comments.

| | |
|---|---|
|  | **Photo 1:**<br>The site for the proposed development, looking from the road. Some of the excess floodwater that cannot pass through the existing road crossing culverts will flow onto the site at this point, unless the road crossing is improved. |
|  | **Photo 2:**<br>Site entrance, looking upstream. The watercourse and road crossing are on the left. |
|  | **Photo 3:**<br>Road crossing, looking SW.  The site is on the RHS. |

| | |
|---|---|
|  | **Photo 4:**<br>Road crossing, looking NE. |
|  | **Photo 5:**<br>The stream, looking upstream from the road crossing, |
|  | **Photo 6:**<br>The stream, looking downstream from the road crossing. The development site is on the left bank. |

**Photo 7:**
The existing double box culvert road crossing.

**Photo 8:**
Another view of the existing road crossing. When the culvert surcharges, excess flows are likely to pass overland on either side.

**Photo 9 (above):**
Panoramic view of watercourse entry and exit point from dam.

**Photo 10:**
Diversion furrow looking upstream.

**Photo 11:**
Diversion furrow looking downstream.

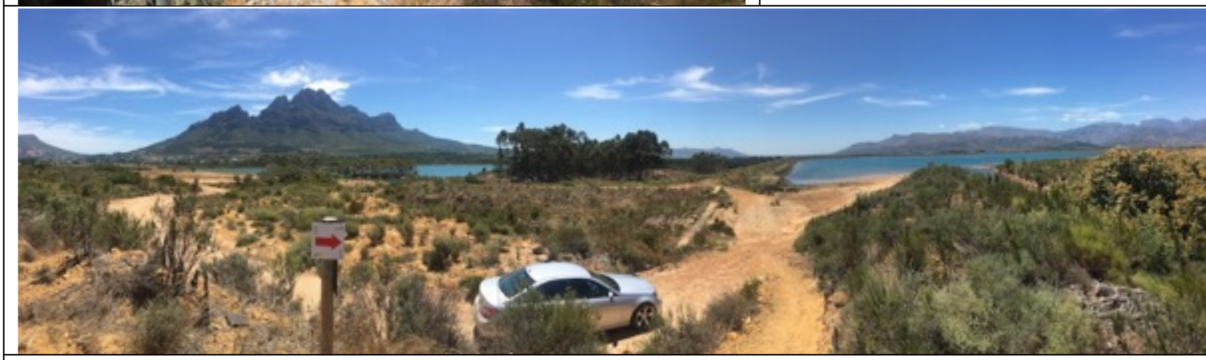


**Photo 12 (above):**
Spillway directing outflow from upper dam to lower dam.

**Photo 13:**
Top dam, with outflow directed towards watercourse in background.

**Photo 14:**
Watercourse alongside the proposed development site.

**Photo 15:**
Watercourse alongside the proposed development site.

**Photo 16:**
Erosion in the watercourse near the site of the development.

# Annexure B1: Rational Method Calculations: Present Catchment

## Determination of peak runoff

| | | | |
|---|---|---|---|
| **Position of Centroid** | 33deg 54min S | 18deg 59min E | (from Google Earth) |
| **MAP (mm)** | 1082 | From Rainfall | |

| Catchment Area (from Google Earth) km$^2$ | Length of defined watercourse (from Google Earth trace) km | Elevation at 0.1L (from Google Earth elevation profile) mMSL | Elevation at 0.85L (from Google Earth elevation profile) mMSL | H m | Avg Slope (1085 Slope Method) m/m | Tc for defined watercourse (SCS Formula) hours | Length of overland flow km | Roughness coefficient (Drainage Manual table 3.9) | Height difference m | Slope m/m |
|---|---|---|---|---|---|---|---|---|---|---|
| 7.90 | 6.4 | 229 | 1000 | 771 | 0.1606 | 0.56 | 0.1 | 0.50 | 60 | 0.500 |

| Tc for overland portion (Kerby Formula) hours | **Tc total hours** | **Tc total mins** | ARF (Formula 3.13) % |
|---|---|---|---|
| 0.19 | **0.75** | **45** | 100 |

**Runoff Coefficient (Drainage Manual Table 3.7 for MAP >900mm): Rural**

| Component | | Classification | Factor |
|---|---|---|---|
| Cs | Surface Slope | Hilly (10-30deg) | 0.20 |
| Cp | Permeability | Semi-permeable | 0.20 |
| Cv | Vegetation | plantation | 0.05 |
| | | C1 | 0.45 |

**Adjustment factor for initial saturation (Table 3.8)**

| Return Period (years) | | 1 | 2 | 5 | 10 | 20 | 50 | 100 | 200 |
|---|---|---|---|---|---|---|---|---|---|
| Steep and impermeable Catchment | | | 0.75 | 0.80 | 0.85 | 0.90 | 0.95 | 1.00 | 1.00 |
| Flat and permeable Catchment | | | 0.50 | 0.55 | 0.60 | 0.67 | 0.83 | 1.00 | 1.00 |
| Adopted adjustment factor (Ft) | | | 0.68 | 0.73 | 0.78 | 0.83 | 0.91 | 1.00 | 1.00 |
| C | Runoff coefficient | | **0.30** | **0.33** | **0.35** | **0.37** | **0.41** | **0.45** | **0.45** |

**Rational Method** (not applicable to catchments > 15 km2)

Time of Concentration (from calculation above) **45** **mins**

| Return Period (years) | | 1 | 2 | 5 | 10 | 20 | 50 | 100 | 200 |
|---|---|---|---|---|---|---|---|---|---|
| Rainfall depth (mm) at centroid of catchment for Tc as follows: | 45 | | 17.2 | 23.6 | 28.4 | 33.4 | 40.4 | 46.3 | 52.6 |
| | 60 | | | | | | | 50.0 | |
| Rainfall at centroid of catchment for Tc of catchment | Depth (mm) | | 17.1 | 23.5 | 28.3 | 33.3 | 40.3 | 46.3 | 52.4 |
| | Intensity (mm/hr) | | 22.8 | 31.3 | 37.7 | 44.3 | 53.6 | 61.7 | 69.8 |
| Peak Flow (m3/sec) | | | **15.2** | **22.4** | **28.9** | **36.4** | **48.4** | **61.0** | **69.0** |

# Annexure B2: Rational Method Calculations: Original Catchment

**Determination of peak runoff - original catchment**

| | | | |
|---|---|---|---|
| **Position of Centroid** | 33deg 54min S | 18deg 59min E | (from Google Earth) |
| **MAP (mm)** | 1082 | From Rainfall | |

| Catchment Area (from Google Earth) | Length of defined watercourse (from Google Earth trace) | Elevation at 0.1L (from Google Earth elevation profile) | Elevation at 0.85L (from Google Earth elevation profile) | H | Avg Slope (1085 Slope Method) | Tc for defined watercourse (SCS Formula) | Length of overland flow | Roughness coefficient (Drainage Manual table 3.9) | Height difference | Slope |
|---|---|---|---|---|---|---|---|---|---|---|
| $km^2$ | km | mMSL | mMSL | m | m/m | hours | km | | m | m/m |
| 4.00 | 3.5 | 221 | 467 | 246 | 0.0937 | 0.43 | 0.1 | 0.50 | 60 | 0.500 |

| Tc for overland portion (Kerby Formula) | **Tc total** | **Tc total** | ARF (Formula 3.13) |
|---|---|---|---|
| hours | **hours** | **mins** | % |
| 0.19 | **0.62** | **37** | 100 |

**Runoff Coefficient (Drainage Manual Table 3.7 for MAP >900mm): Rural**

| Component | | Classification | Factor |
|---|---|---|---|
| Cs | Surface Slope | Hilly (10-30deg) | 0.20 |
| Cp | Permeability | Semi-permeable | 0.20 |
| Cv | Vegetation | plantation | 0.05 |
| | | C1 | 0.45 |

**Adjustment factor for initial saturation (Table 3.8)**

| Return Period (years) | | 1 | 2 | 5 | 10 | 20 | 50 | 100 | 200 |
|---|---|---|---|---|---|---|---|---|---|
| Steep and impermeable Catchment | | | 0.75 | 0.80 | 0.85 | 0.90 | 0.95 | 1.00 | 1.00 |
| Flat and permeable Catchment | | | 0.50 | 0.55 | 0.60 | 0.67 | 0.83 | 1.00 | 1.00 |
| Adopted adjustment factor (Ft) | | | 0.68 | 0.73 | 0.78 | 0.83 | 0.91 | 1.00 | 1.00 |
| C | Runoff coefficient | | **0.30** | **0.33** | **0.35** | **0.37** | **0.41** | **0.45** | **0.45** |

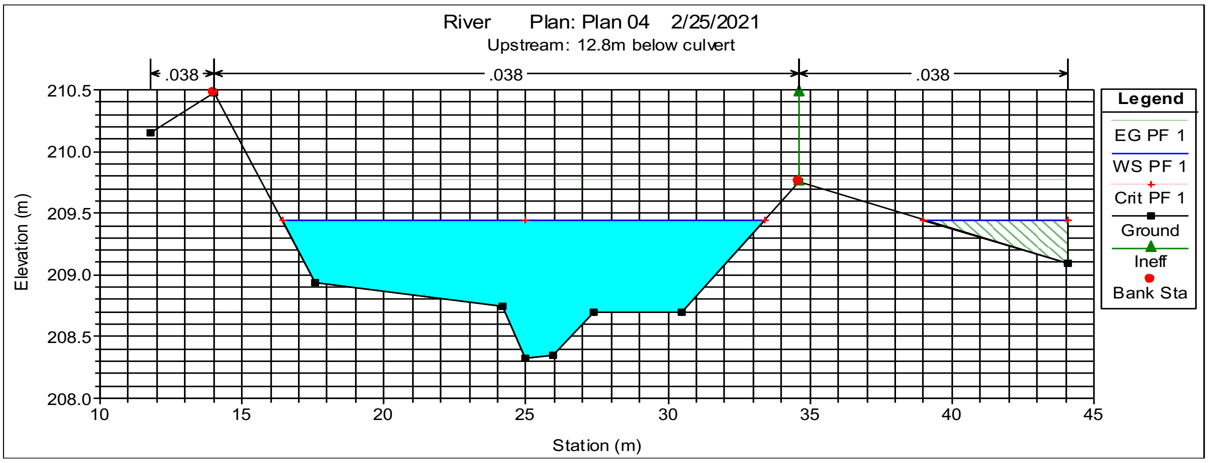
**Rational Method** (not applicable to catchments > 15 km2)

Time of Concentration (from calculation above) **37** **mins**

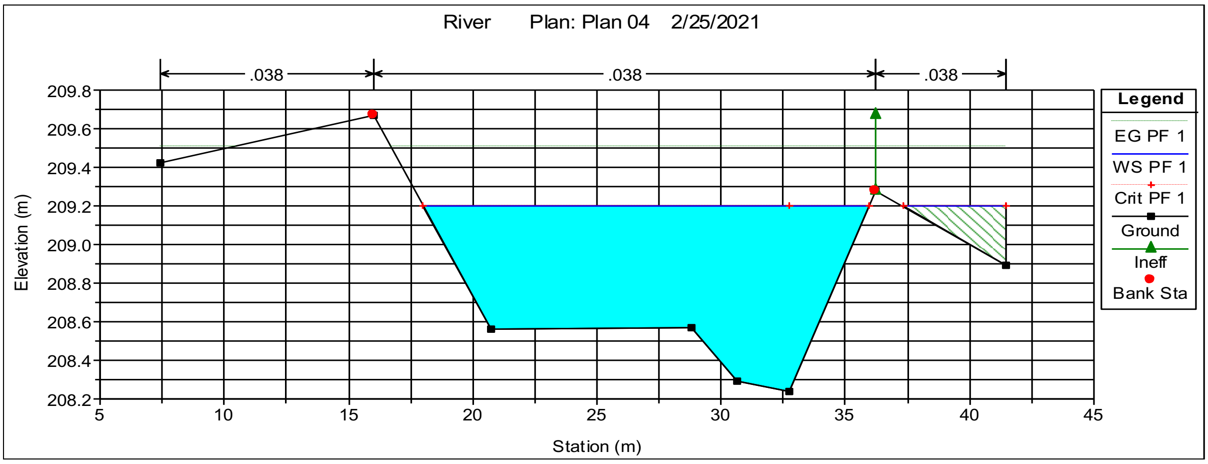
| Return Period (years) | | 1 | 2 | 5 | 10 | 20 | 50 | 100 | 200 |
|---|---|---|---|---|---|---|---|---|---|
| Rainfall depth (mm) at centroid of catchment for Tc as follows: | 30 | | 13.9 | 19.1 | 22.9 | 26.9 | 32.6 | 37.1 | 42.5 |
| | 45 | | 17.2 | 23.6 | 28.4 | 33.4 | 40.4 | 46.3 | 52.6 |
| Rainfall at centroid of catchment for Tc of catchment | Depth (mm) | | 15.5 | 21.3 | 25.6 | 30.1 | 36.5 | 41.7 | 47.5 |
| | Intensity (mm/hr) | | 24.9 | 34.2 | 41.1 | 48.3 | 58.4 | 66.8 | 76.1 |
| Peak Flow (m3/sec) | | | **8.4** | **12.4** | **15.9** | **20.1** | **26.7** | **33.4** | **38.1** |

# Annexure C: Hec-Ras Cross-sections

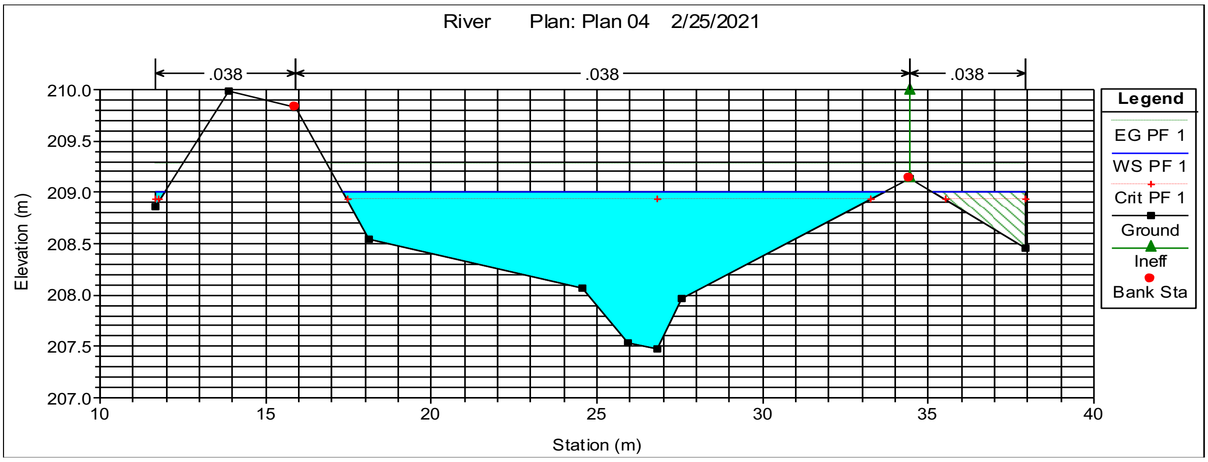
## CH-6



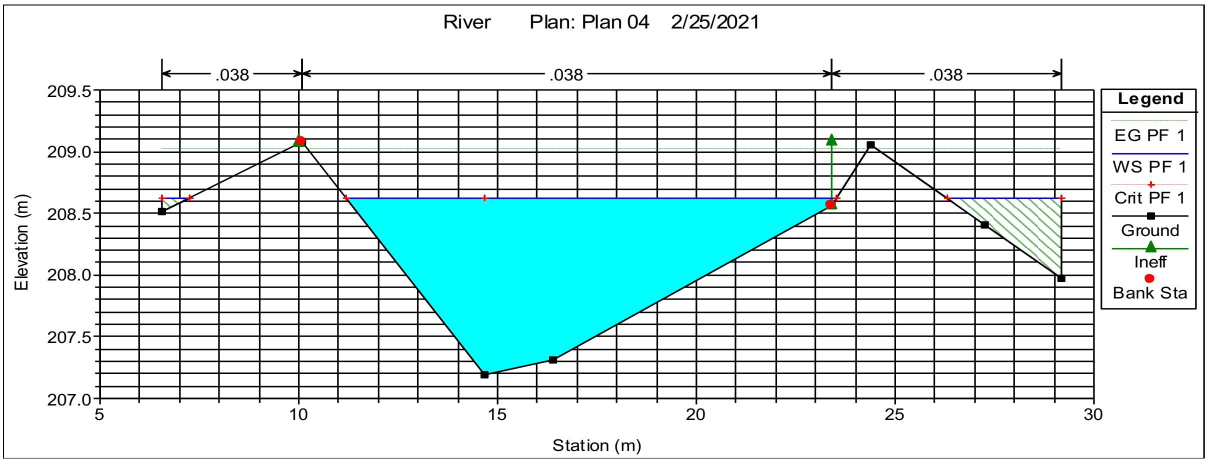

## CH-7



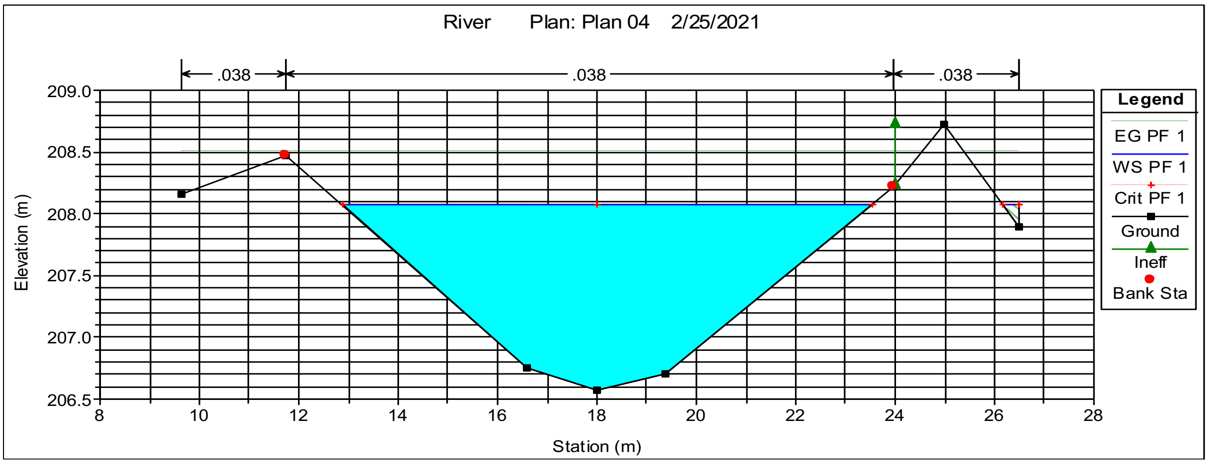

## CH-8



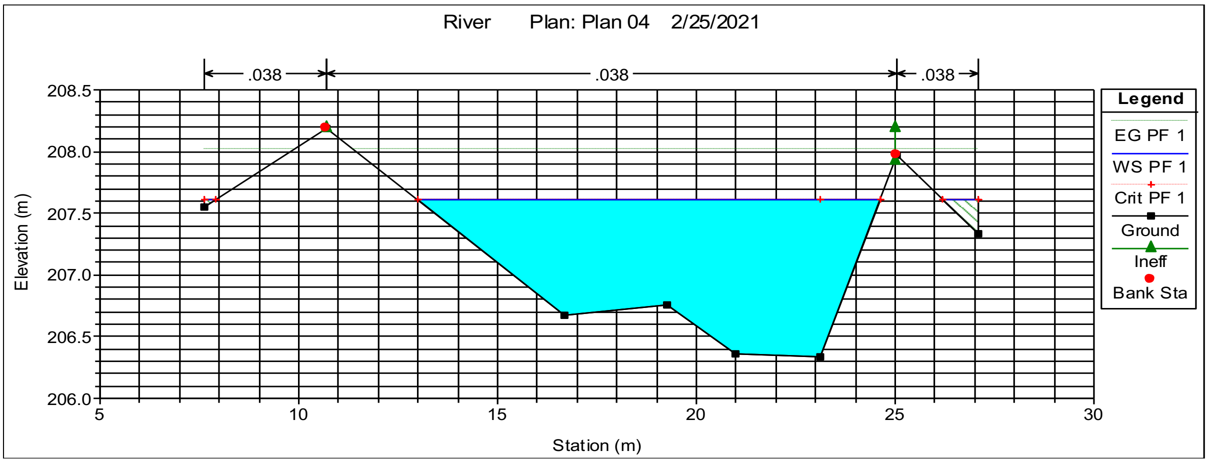
## CH-9



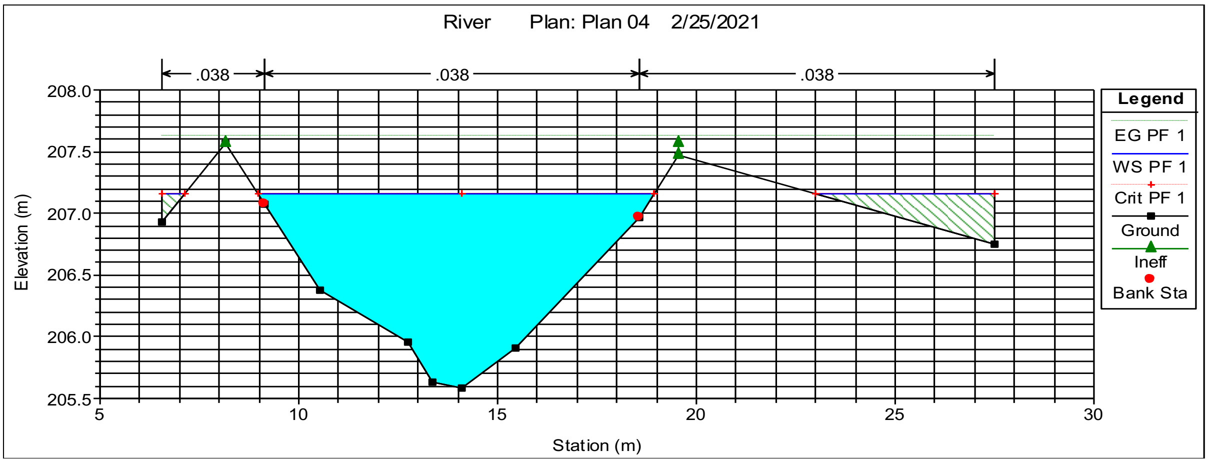

## CH-10



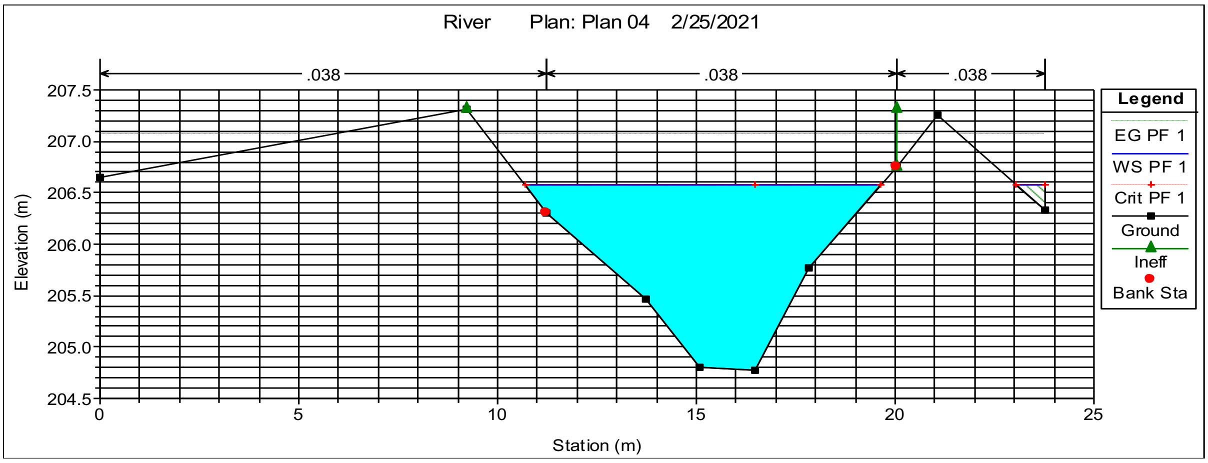

## CH-11



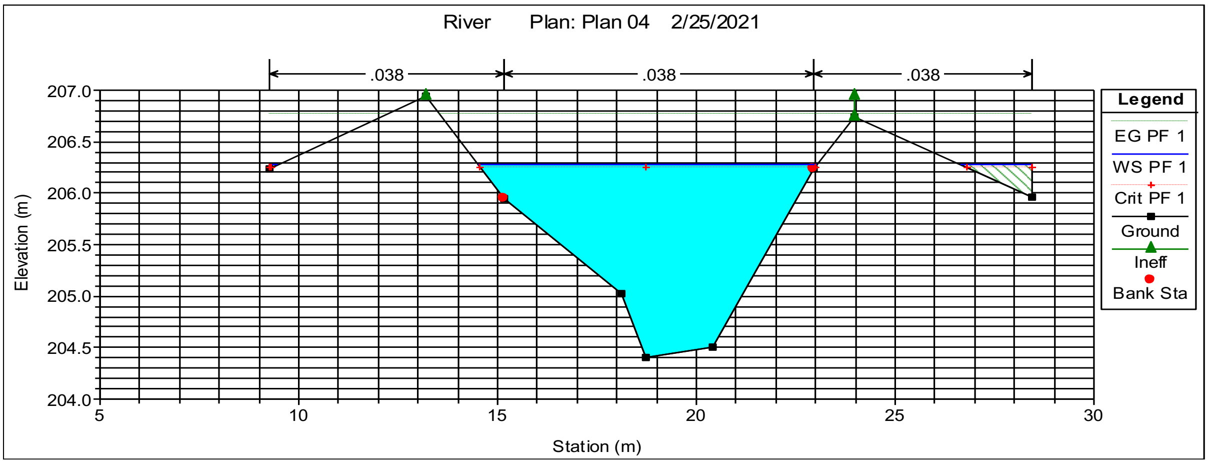
## CH-12



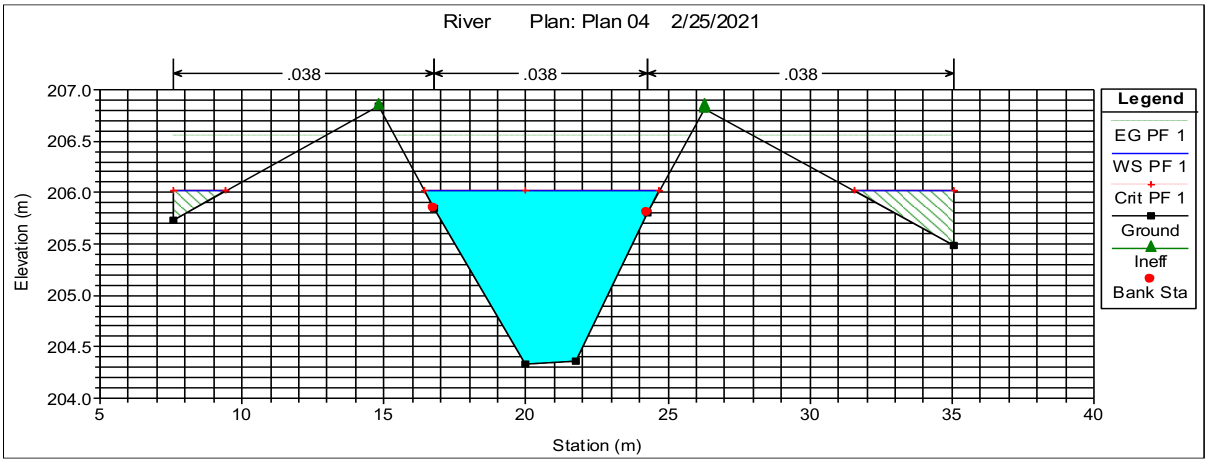

## CH-13



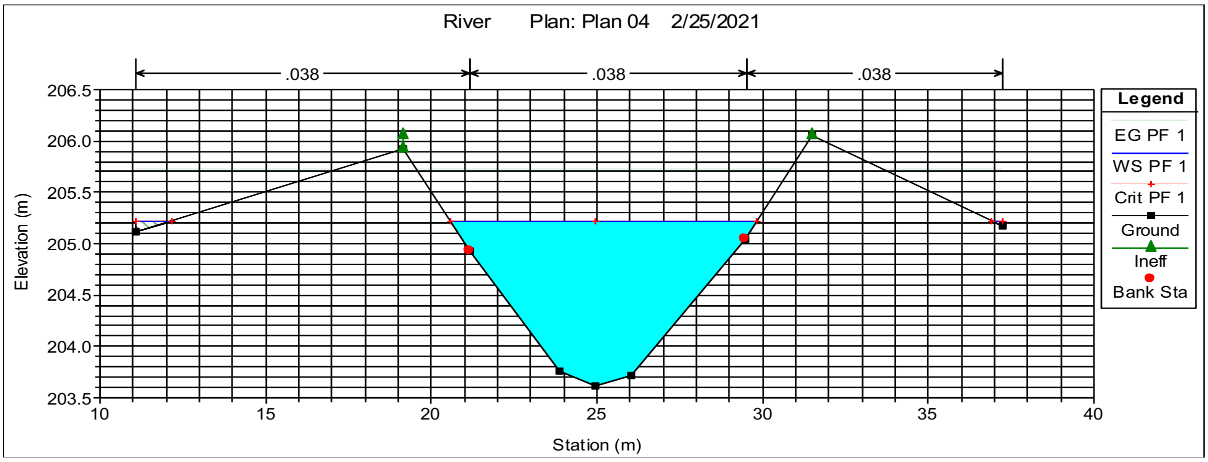

## CH-14



## CH-15

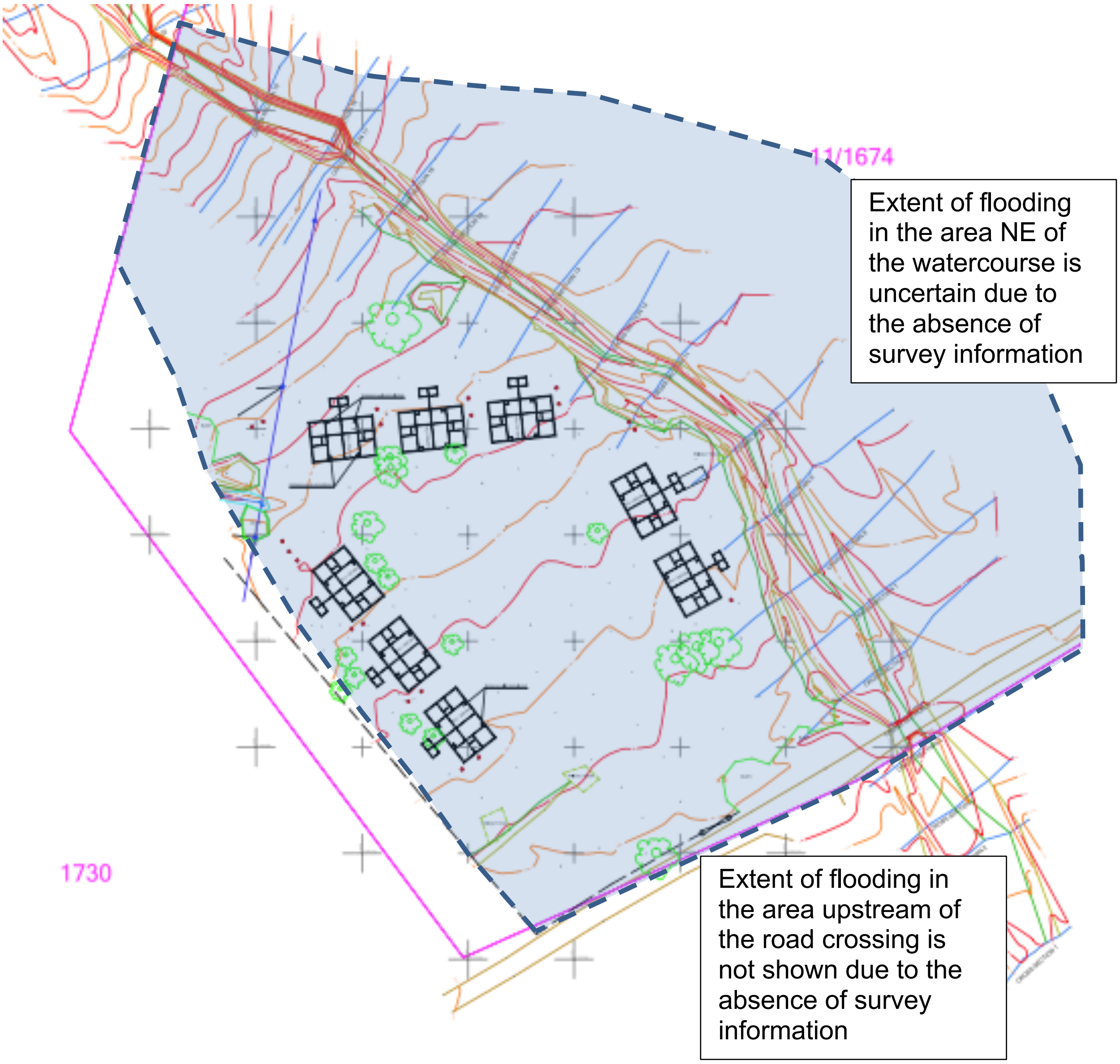

River Plan: Plan 04 2/25/2021
.038
.038
.038
207.0
206.5
206.0
205.5
205.0
204.5
204.0
Elevation (m)
5
10
15
20
25
30
35
40
Station (m)
Legend
EG PF 1
WS PF 1
Crit PF 1
Ground
Ineff
Bank Sta

## CH-16


River Plan: Plan 04 2/25/2021
.038
.038
.038
206.5
206.0
205.5
205.0
204.5
204.0
203.5
Elevation (m)
10
15
20
25
30
35
40
Station (m)
Legend
EG PF 1
WS PF 1
Crit PF 1
Ground
Ineff
Bank Sta

# Annexure D: Flood Lines: Existing

The shaded area on this diagram indicates the extent of the site that is likely to be affected by the 100-year flood due to water bypassing and/or overtopping the existing road crossing. The extent of flooding upstream of the road crossing is not shown.



# Annexure E:  Flood Lines: Proposed