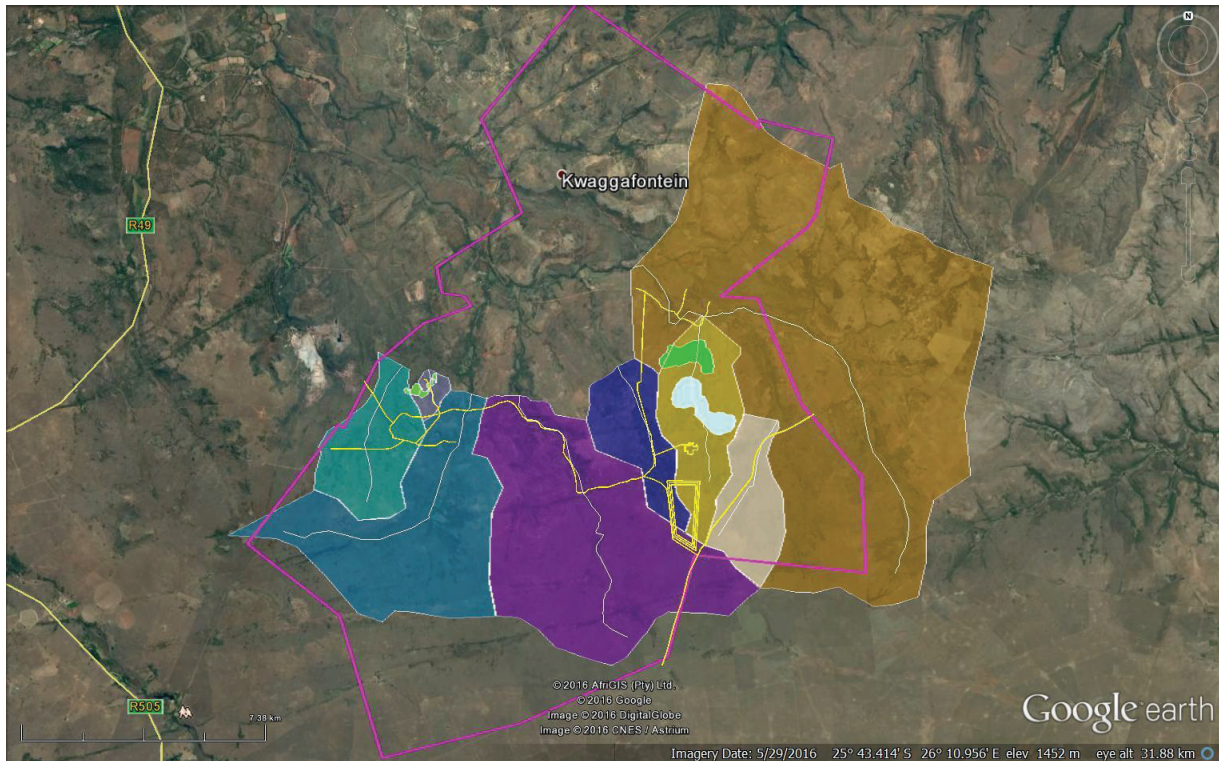


**DETERMINATION OF 1:100 YEAR FLOOD LINES STORMWATER MANAGEMENT
PLAN AND DESIGN OF SMALLER STRUCTURES TO COMPLY WITH THE
REQUIREMENTS OF GN704: PROPOSED DOORNHOEK FLUORSPAR MINE
BETWEEN ZEERUST AND MAFIKENG**



CLIENT: Exigo Sustainability (Pty) Ltd
REPORT NO CWT 722015
DATE: 5 October 2016

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PLAN AND DESIGN OF SMALLER STRUCTURES TO COMPLY WITH THE
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1. INTRODUCTION

CWT Consulting was appointed by **Exigo Sustainability (Pty) Ltd** to determine the 1:100 year flood lines and prepare a Storm Water Management Plan to comply with the requirements of GN704 for the proposed Doornhoek Fluorspar Mine between Zeerust and Lichtenburg.

According to section 144 of the National Water Act (ACT No. 36 of 1998), no person may establish a township (or erect structures) unless the layout plan shows (in a form acceptable to the local authority concerned) lines indicating the maximum level likely to be reached by floodwaters on average once in every 100 years.

2. LOCATION

The location of the area is shown below.

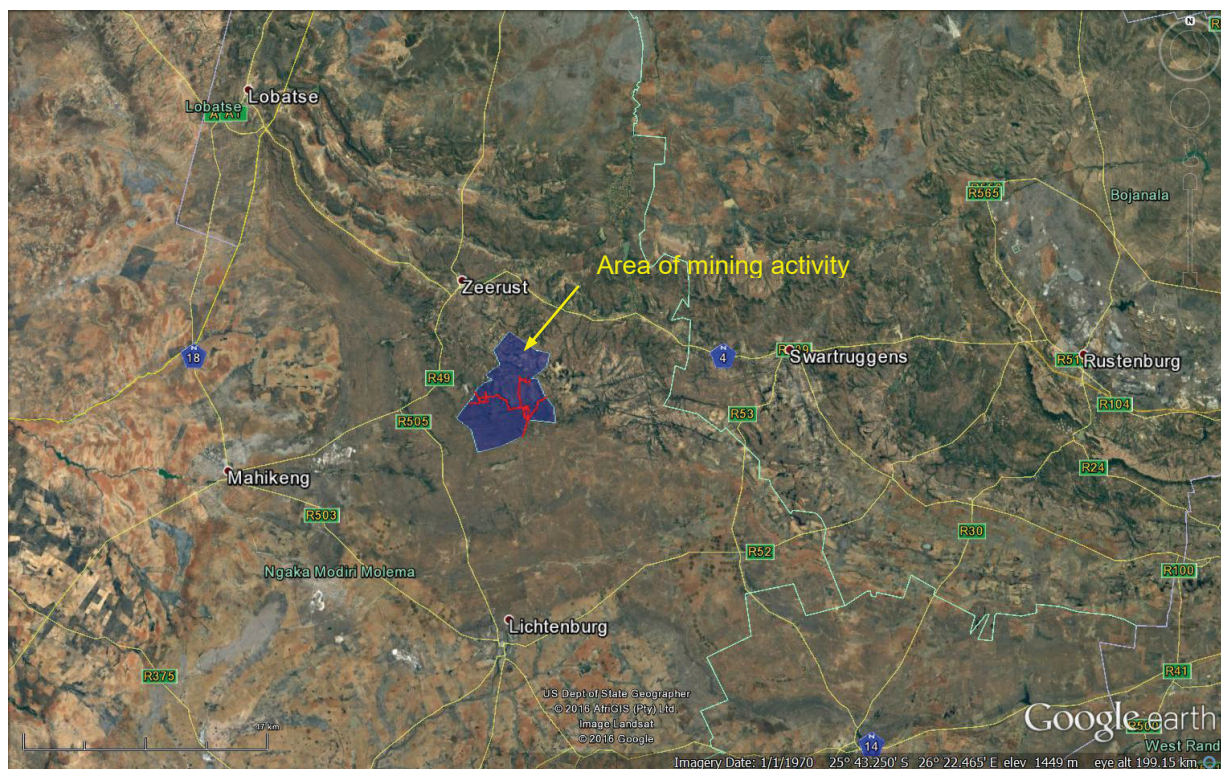


FIGURE 1

3. FACTORS RELATED TO STORMWATER RUN - OFF

3.1 LEGAL

Although not exhaustive, the statutes, ordinances, regulations, by-laws, policies and guidelines listed below have relevance, and should be considered by the developer at the planning stage. They should not however be seen as exhaustive as they are subject to ongoing amendments, revisions and additions.

- Constitution of the Republic of South Africa, 1996 (Act 108 of 1996)
- Local Government: Municipal Systems Bill (B27B-2000)
- National Water Act, 1998 (Act 36 of 1998)
- DWAF Water Quality Guidelines, 1996
- National Environmental Management Act, 1998 (Act 107 of 1998)
- Environmental Conservation Act, 1989 (Act 73 of 1989)
- Environmental Impact Assessment (EIA) Regulations (GN R. 982, 983, 984 and 985 of 4 December 2010)
- Protected Natural Environments
- Conservation of Agricultural Resources Act, 1983 (Act 43 of 1983)
- Development Facilitation Act, 1995 (Act 67 of 1995)
- National Building Regulations and Building Standards Act, 1997 (Act 103 of 1977)

National Water Act 1998 (Act 36 of 1998)

The Act defines water use as the abstraction, consumption and discharge of water.

Use of water includes the discharge of water containing waste into a water resource and the disposal of water containing waste from an industrial process in any manner. (Section 39). Sections 117 to 123 deal with the safety of dams with a safety risk. If such dams fall on the property then cognizance should be taken of the potential impact of the development on the dam. Section 144 specifies the requirement to indicate the 1:100

year flood levels. Section 145 deals with flood risk information which the local water management institution must make available to the public.

National Environmental Management Act (Act 107 of 1998)

The following Sections of the Act have relevance:

Section 2 of the Act establishes a set of principles that apply to the activities of all organs of state that may significantly affect the environment. These include the following:

- development must be sustainable
- pollution must be avoided or minimized and remedied
- waste must be avoided or minimized, reused or recycled
- negative impacts must be minimized.

3.2 MINING SURFACES

The mining will be open cast mining with stockpiling.

4. HYDROLOGY

4.1 Rainfall Data

Catchment MAP (ex HRU quaternary): Less than 600 mm

The rainfall data in the table below are derived from three sources. The modified Hershfield equation is used for durations up to four hours. The daily rainfall is from the Department of Water Affairs and Sanitation publication TR102 adjusted so that TR102 MAP = catchment MAP. Where the equation values exceed the 1-day rainfall, they are reduced to equal to the 1-day rainfall.

Weather Bureau station: 509283 @ Doornhoek

Mean annual precipitation (TR102): **563 mm**

Precipitations in **mm** associated with various storm durations are given in the Table1.

STORM DURATION	Return Period (RP)					
	2	5	10	20	50	100
13 minutes	15 mm	20 mm	25 mm	31 mm	40 mm	49 mm
38 minutes	15 mm	21 mm	26 mm	32 mm	42 mm	52 mm
53 minutes	26 mm	43 mm	56 mm	69 mm	87 mm	100 mm
100 minutes	31 mm	43 mm	54 mm	67 mm	87 mm	107 mm
103 minutes	32 mm	44 mm	55 mm	68 mm	88 mm	108 mm
164 minutes	34 mm	57 mm	74 mm	91 mm	114 mm	132 mm

Table 1

4.2 Catchment Stream 1

The study area and the catchment draining to this portion of the Stream is shown in Figure 2.

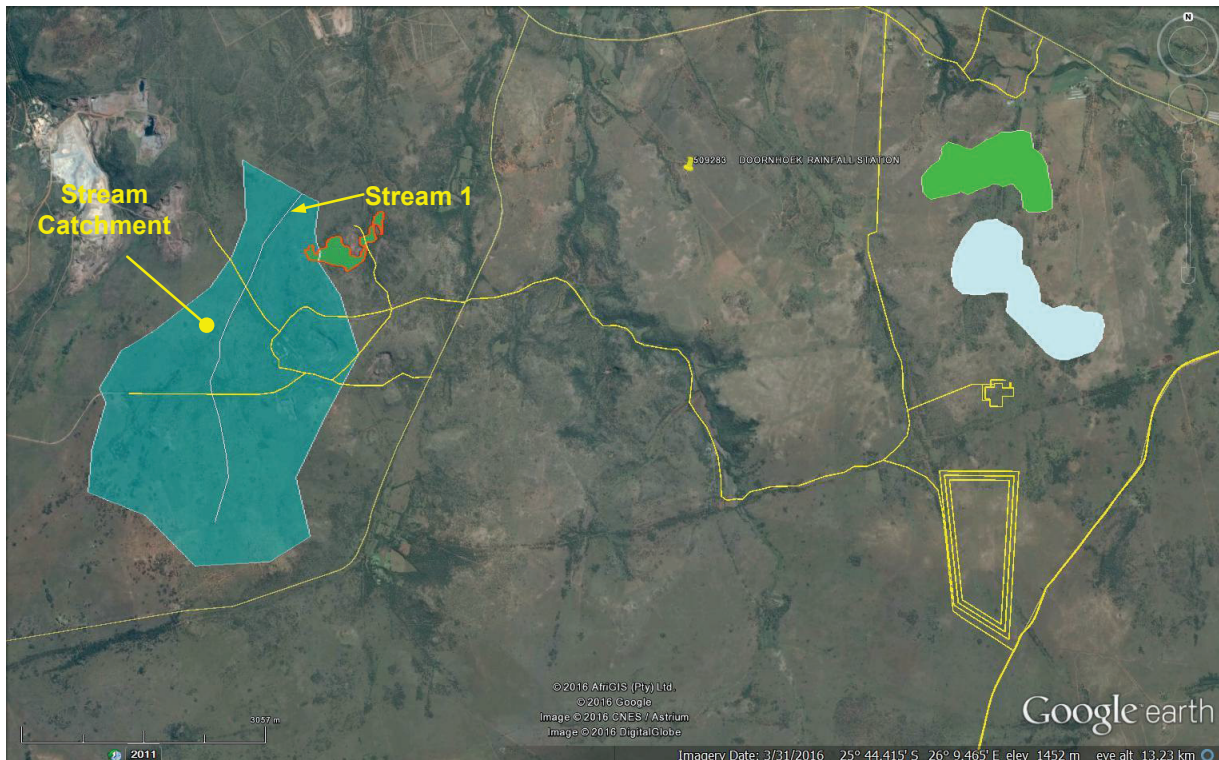


FIGURE 2

4.2.1 Characteristics

Area of catchment:	9,81 km ²
Length of longest watercourse:	4,50 km
Equal area height difference:	72,0 m
10 – 85 slope height difference:	82,7 m
Distance to catchment centroid:	2,6 km
Time of concentration	53 minutes

4.2.2 Flood Peaks

The Effect of Dams on the Flood Peaks

The effect of any dam in the catchment was not taken into account because the 1:100 year flood peak will **not** be attenuated by a dam with a smaller storage capacity than 6 *times the total mean annual runoff* of the catchment draining into the dam.

Methods used to calculate the Flood Peaks

Various different methods were used to calculate the flood hydrology for the catchment as this increases the accuracy of the final flood peak calculation. All the methods used take the following into account:

All factors relating to storm water run-off.

- Evaporation during rain storm
- Wind during rainstorm
- Depth of rainstorm
- Infiltration
- Flow roughness of area.

The following methods were considered:

1. Rational method as implemented by the Department of Water and Sanitation.
2. Rational method using an alternative implementation.
3. Standard Design Flood (SDF) method as developed at Pretoria University.
4. The HRU algorithm developed by Midgley and Pitman.

Most applicable methods for the catchments

Due to the size of the catchment the results obtained from all the methods are deemed to be applicable for this study.

Results of the calculations

The results are listed below. The flows indicated are in cubic meter per second.

Details of the calculations are shown in Addendum 33.

Return Period Year	Rational method DWA	Rational method Alternative algorithm	SDF method	HRU Algorithm	Unit Hydro- graph
1:50	85	103	90	50	65
1:100	110	125	113	64	89

Table 2

Recommended Flood Peaks

The flood peaks were calculated by applying the following algorithm:

$$Q_T = [RMDWA + RMAL + SDF + HRU] / N$$

With:

- Q_T** = Flood peak for return period T
- T** = Return Period
- RMDWA** = Rational method DWA
- RMAL** = Rational method Alternative algorithm
- SDF** = SDF method
- N** = 4

The recommended flood peaks in cumec (cubic meter per second) at the site are listed in Table 3 below:

Return Period Year	Flood peak in the Stream 1
1:50	82
1:100	103

Table 3

4.3 Catchment Stream 2

The study area and the catchment draining to this portion of the Stream is shown in Figure 3.

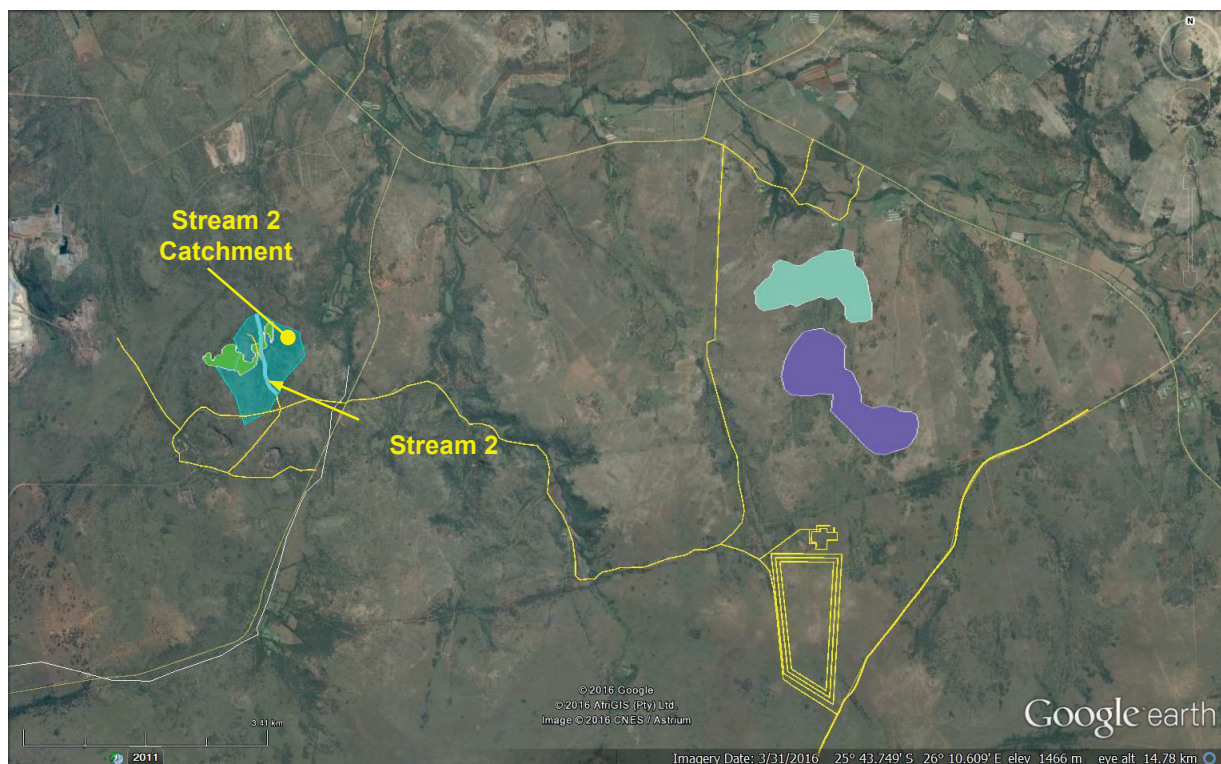


FIGURE 3

4.3.1 Characteristics

Area of catchment:	1,05 km ²
Length of longest watercourse:	1,20 km
Equal area height difference:	51 m
10 – 85 slope height difference:	59 m
Distance to catchment centroid:	0,7 km
Time of concentration	13 minutes

4.3.2 Flood Peaks

The same different deterministic methods were used as described above.

The effect of any dam in the catchment was not taken into account because the 1:100 year flood peak will **not** be attenuated by a dam with a smaller storage capacity than 6 *times the total mean annual runoff* of the catchment draining into the dam.

Results of the calculations

The results are listed below. The flows indicated are in cubic meter per second.

Details of the calculations are shown in Addendum 34.

Return Period Year	Rational method DWA	Rational method Alternative algorithm	SDF method	HRU Algorithm	Unit Hydro-graph
1:50	22	29	25	16	3
1:100	30	35	30	20	4

Table 4

Recommended Flood Peaks

The flood peaks were calculated by applying the following algorithm:

$$Q_T = [RMDWA + RMAL + SDF + HRU] / N$$

With:

- Q_T** = Flood peak for return period T
- T** = Return Period
- RMDWA** = Rational method DWA
- SDF** = SDF method
- RMAL** = Rational method Alternative algorithm
- N** = 4

The recommended flood peaks in cumec (cubic meter per second) at the site are listed in Table 5 below:

Return Period Year	Flood peak in Stream 2
1:50	23
1:100	29

Table 5

4.4 Catchment Stream 3

The study area is shown in Figure 4.

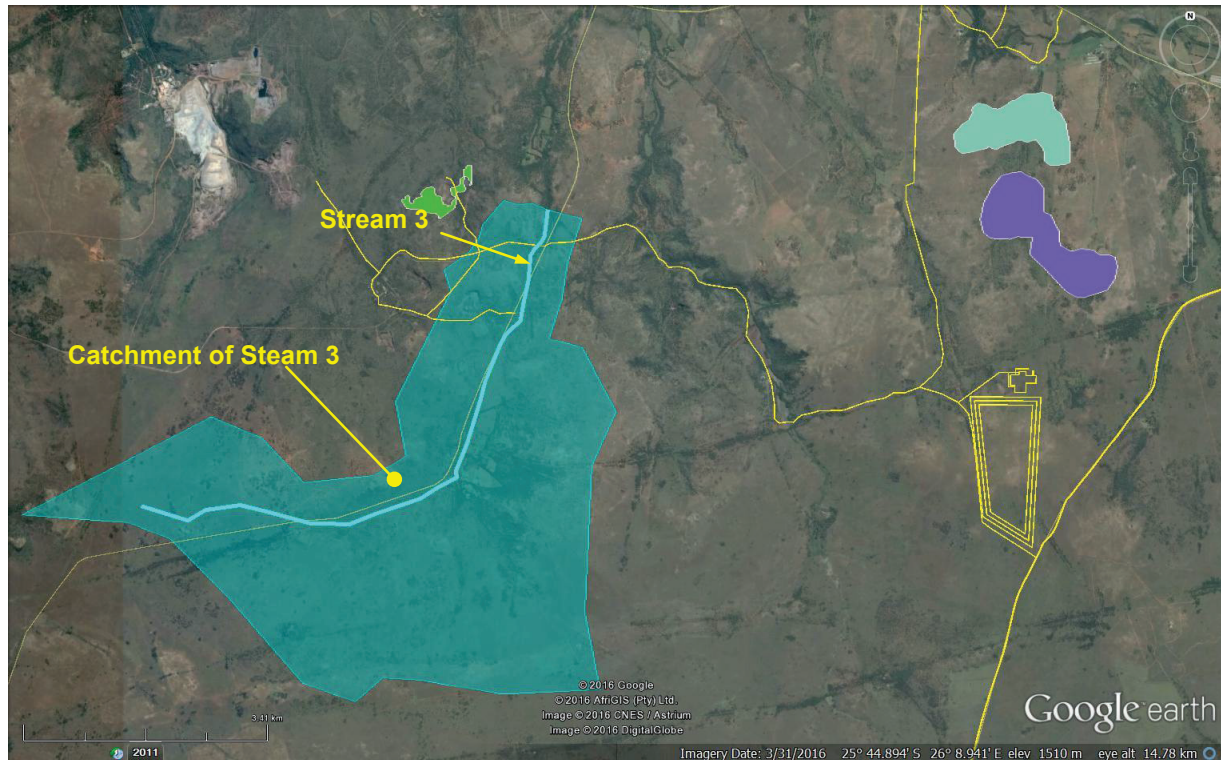


FIGURE 4

4.4.1 Characteristics of the catchment: Stream 3

Area of catchment:	25,72 km ²
Length of longest watercourse:	8,7 km
Equal area height difference:	120 m
10 – 85 slope height difference:	116 m
Distance to catchment centroid:	4,6 km
Time of concentration	100 minutes

Flood Peaks

The same different deterministic methods were used as described above.

The effect of any dam in the catchment was not taken into account because the 1:100 year flood peak will **not** be attenuated by a dam with a smaller storage capacity than 6 *times the total mean annual runoff* of the catchment draining into the dam.

Results of the calculations for Stream 3

The results are listed below in Table 6. The flows indicated are in cubic meter per second. Details of the calculations are shown in Addendum 23.

Return Period Year	Rational method DWA	Rational method Alternative algorithm	SDF method	HRU Algorithm	Unit Hydro-graph
1:50	143	172	143	84	16
1:100	183	208	181	107	23

Table 6

Recommended Flood Peaks

The flood peaks were calculated by applying the following algorithm:

$$Q_T = [RMDWA + RMAL + SDF + HRU] / N$$

With:

Q_T = Flood peak for return period T

T = Return Period

RMDWA = Rational method DWA

RMAL = Rational method Alternative algorithm

SDF = SDF method

N = 4

The recommended flood peaks in cumec (cubic meter per second) at the site are listed in Table 7 below:

Return Period Year	Flood peak in Stream 3
1:50	136
1:100	170

Table 7

4.5 Catchment Stream 4

This study area and the catchment draining to this stream is shown in Figure 5.

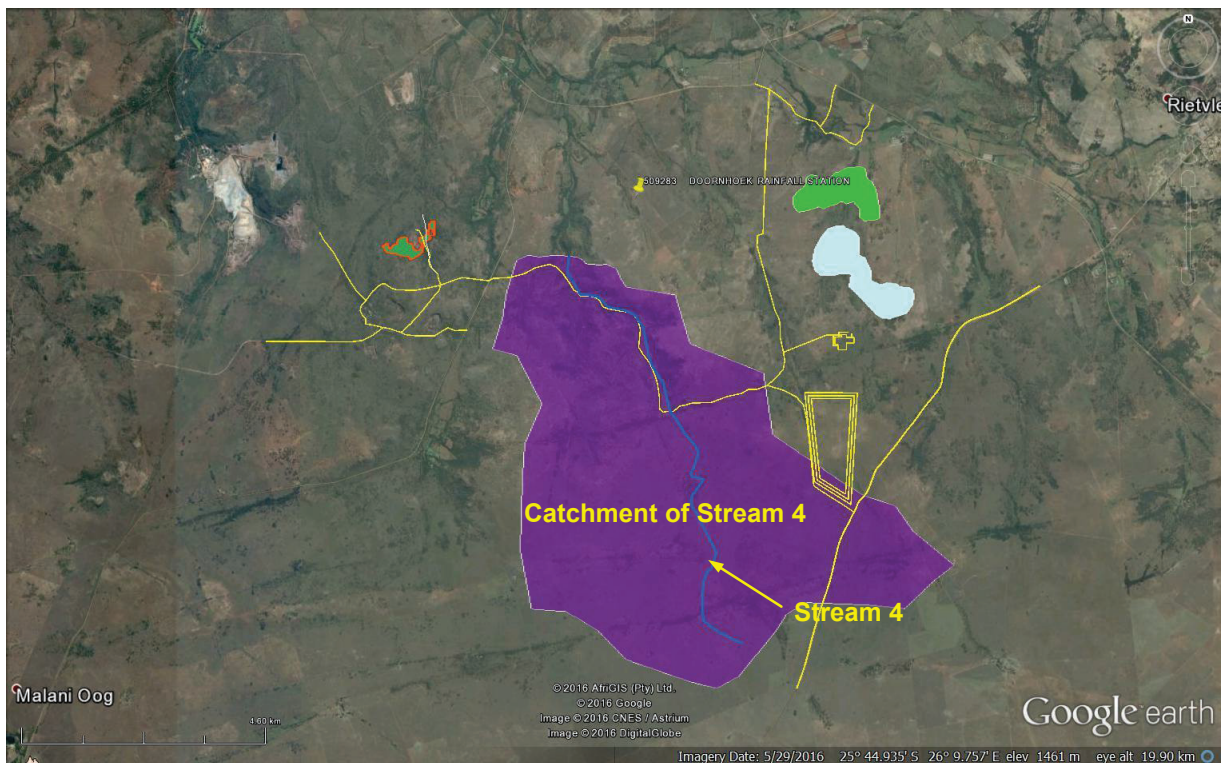


FIGURE 5

4.5.1 Characteristics of Stream 4.

Area of catchment:	37,9 km ²
Length of longest watercourse:	9,3 km
Equal area height difference:	143 m
10 – 85 slope height difference:	131 m
Distance to catchment centroid:	1,04 km
Time of concentration	103 minutes

Flood Peaks

The same different deterministic methods were used as described above.

The effect of any dam in the catchment was not taken into account because the 1:100 year flood peak will **not** be attenuated by a dam with a smaller storage capacity than 6 times the total mean annual runoff of the catchment draining into the dam.

Results of the calculations for Stream 4

The results are listed below in Table 8. The flows indicated are in cubic meter per second. Details of the calculations are shown in Addendum 36.

Return Period Year	Rational method DWA	Rational method Alternative algorithm	SDF method	HRU Algorithm	Unit Hydro-graph
1:50	204	244	202	110	23
1:100	260	296	256	139	31

Table 8

Recommended Flood Peaks for this stream.

The flood peaks were calculated by applying the following algorithm:

$$Q_T = [RMDWA + RMAL + SDF + HRU] / N$$

With:

- Q_T** = Flood peak for return period T
T = Return Period
RMDWA = Rational method DWA
RMAL = Rational method Alternative algorithm
SDF = SDF method
N = 4

The recommended flood peaks in cumec (cubic meter per second) at the site are listed in Table 9 below:

Return Period Year	Flood peak in Stream 4
1:50	190
1:100	238

Table 9

4.6 Catchment Stream 5

This study area and the catchment draining to this stream is shown in Figure 6.

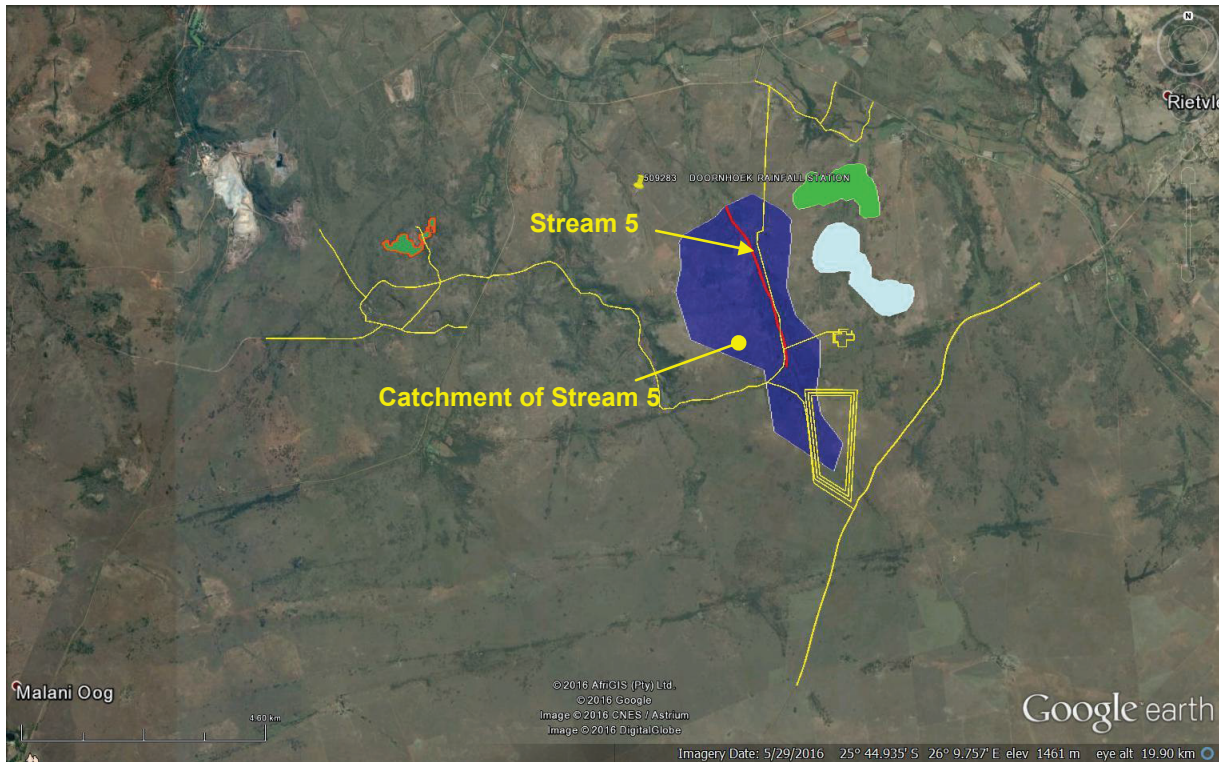


FIGURE 6

4.6.1 Characteristics of Stream 5.

Area of catchment:	7,8 km ²
Length of longest watercourse:	3,3 km
Equal area height difference:	143 m
10 – 85 slope height difference:	73,3 m
Distance to catchment centroid:	3,4 km
Time of concentration	38 minutes

Flood Peaks

The same different deterministic methods were used as described above.

The effect of any dam in the catchment was not taken into account because the 1:100 year flood peak will **not** be attenuated by a dam with a smaller storage capacity than 6 *times the total mean annual runoff* of the catchment draining into the dam.

Results of the calculations for Stream 5

The results are listed below in Table 9. The flows indicated are in cubic meter per second. Details of the calculations are shown in Addendum 37.

Return Period Year	Rational method DWA	Rational method Alternative algorithm	SDF method	HRU Algorithm	Unit Hydro-graph
1:50	103	125	89	44	8
1:100	132	152	112	55	11

Table 9

Recommended Flood Peaks for this stream.

The flood peaks were calculated by applying the following algorithm:

$$Q_T = [RMDWA + RMAL + SDF + HRU] / N$$

With:

- Q_T** = Flood peak for return period T
- T** = Return Period
- RMDWA** = Rational method DWA
- RMAL** = Rational method Alternative algorithm
- SDF** = SDF method
- N** = 4

The recommended flood peaks in cumec (cubic meter per second) at the site are listed in Table 10 below:

Return Period Year	Flood peak in Stream 4
1:50	90
1:100	113

Table 10

4.7 Catchment Stream 6

This study area and the catchment draining to this stream is shown in Figure 7.

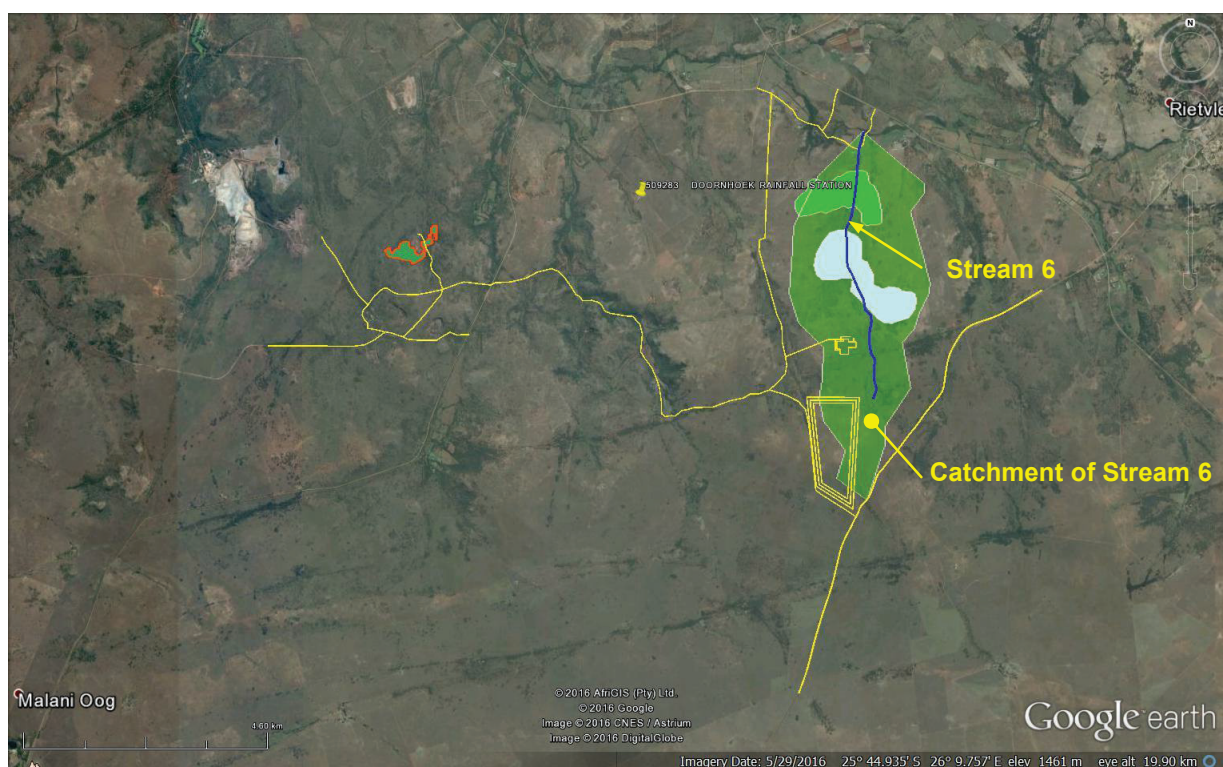


FIGURE 7

4.7.1 Characteristics of Stream 6.

Area of catchment:	11,3 km ²
Length of longest watercourse:	5,2 km
Equal area height difference:	110,5 m
10 – 85 slope height difference:	118,7 m
Distance to catchment centroid:	3,4 km
Time of concentration	55 minutes

Flood Peaks

The same different deterministic methods were used as described above.

The effect of any dam in the catchment was not taken into account because the 1:100 year flood peak will **not** be attenuated by a dam with a smaller storage capacity than 6 times the total mean annual runoff of the catchment draining into the dam.

Results of the calculations for Stream 6

The results are listed below in Table 10. The flows indicated are in cubic meter per second. Details of the calculations are shown in Addendum 38.

Return Period Year	Rational method DWA	Rational method Alternative algorithm	SDF method	HRU Algorithm	Unit Hydro-graph
1:50	102	122	100	54	66
1:100	131	149	127	68	90

Table 10

Recommended Flood Peaks for this stream.

The flood peaks were calculated by applying the following algorithm:

$$Q_T = [RMDWA + RMAL + SDF + HRU] / N$$

With:

- Q_T** = Flood peak for return period T
- T** = Return Period
- RMDWA** = Rational method DWA
- RMAL** = Rational method Alternative algorithm
- SDF** = SDF method
- N** = 4

The recommended flood peaks in cumec (cubic meter per second) at the site are listed in Table 11 below:

Return Period Year	Flood peak in Stream 6
1:50	95
1:100	119

Table 11

4.8 Catchment Stream 7

This study area and the catchment draining to this stream is shown in Figure 8.

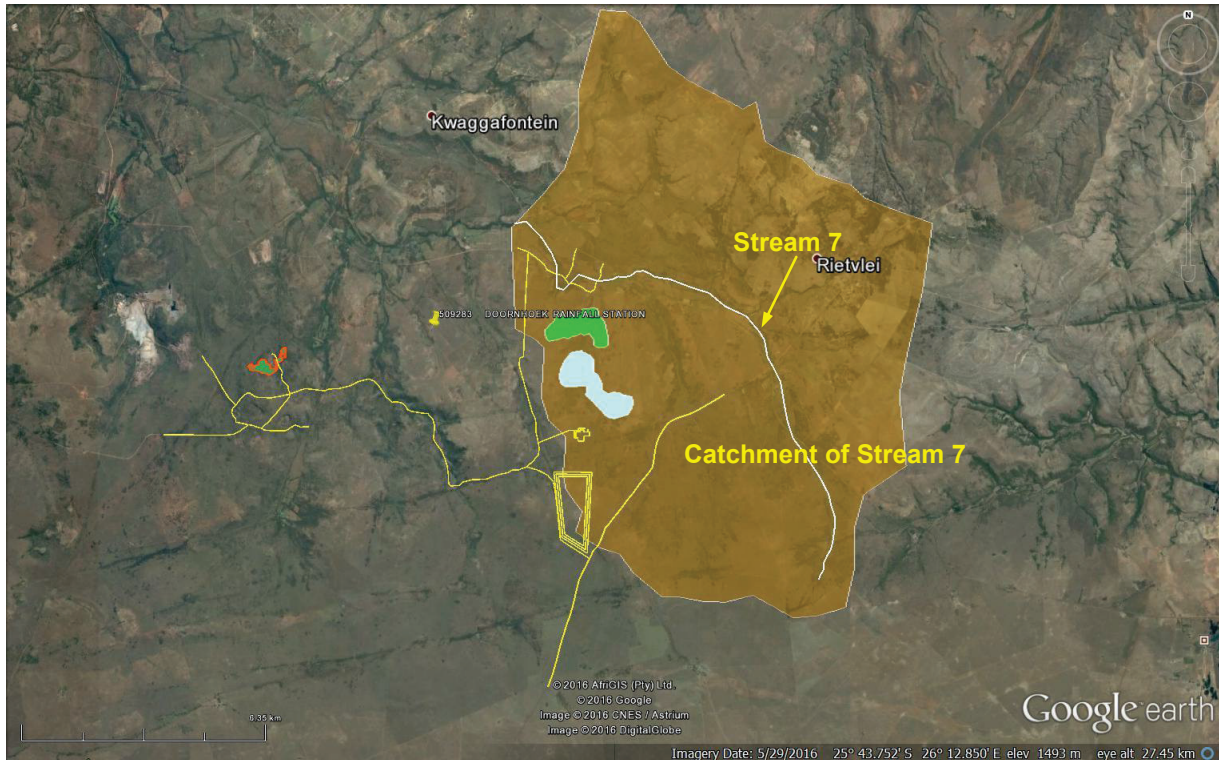


FIGURE 8

4.8.1 Characteristics of Stream 7.

Area of catchment:	112,4 km ²
Length of longest watercourse:	15,3 km
Equal area height difference:	129,3 m
10 – 85 slope height difference:	170,7 m
Distance to catchment centroid:	6,7 km
Time of concentration	164 minutes

Flood Peaks

The same different deterministic methods were used as described above.

The effect of any dam in the catchment was not taken into account because the 1:100 year flood peak will **not** be attenuated by a dam with a smaller storage capacity than 6 *times the total mean annual runoff* of the catchment draining into the dam.

Results of the calculations for Stream 7

The results are listed below in Table 11. The flows indicated are in cubic meter per second. Details of the calculations are shown in Addendum 39.

Return Period Year	Rational method DWA	Rational method Alternative algorithm	SDF method	HRU Algorithm	Unit Hydro-graph
1:50	410	480	397	217	244
1:100	521	582	503	274	324

Table 11

Recommended Flood Peaks for this stream.

The flood peaks were calculated by applying the following algorithm:

$$Q_T = [RMDWA + RMAL + SDF + HRU] / N$$

With:

- Q_T** = Flood peak for return period T
- T** = Return Period
- RMDWA** = Rational method DWA
- RMAL** = Rational method Alternative algorithm
- SDF** = SDF method
- N** = 4

The recommended flood peaks in cumec (cubic meter per second) at the site are listed in Table 12 below:

Return Period Year	Flood peak in Stream 7
1:50	376
1:100	470

Table 12

4.9 Catchment Stream 8

This study area and the catchment draining to this stream is shown in Figure 9.

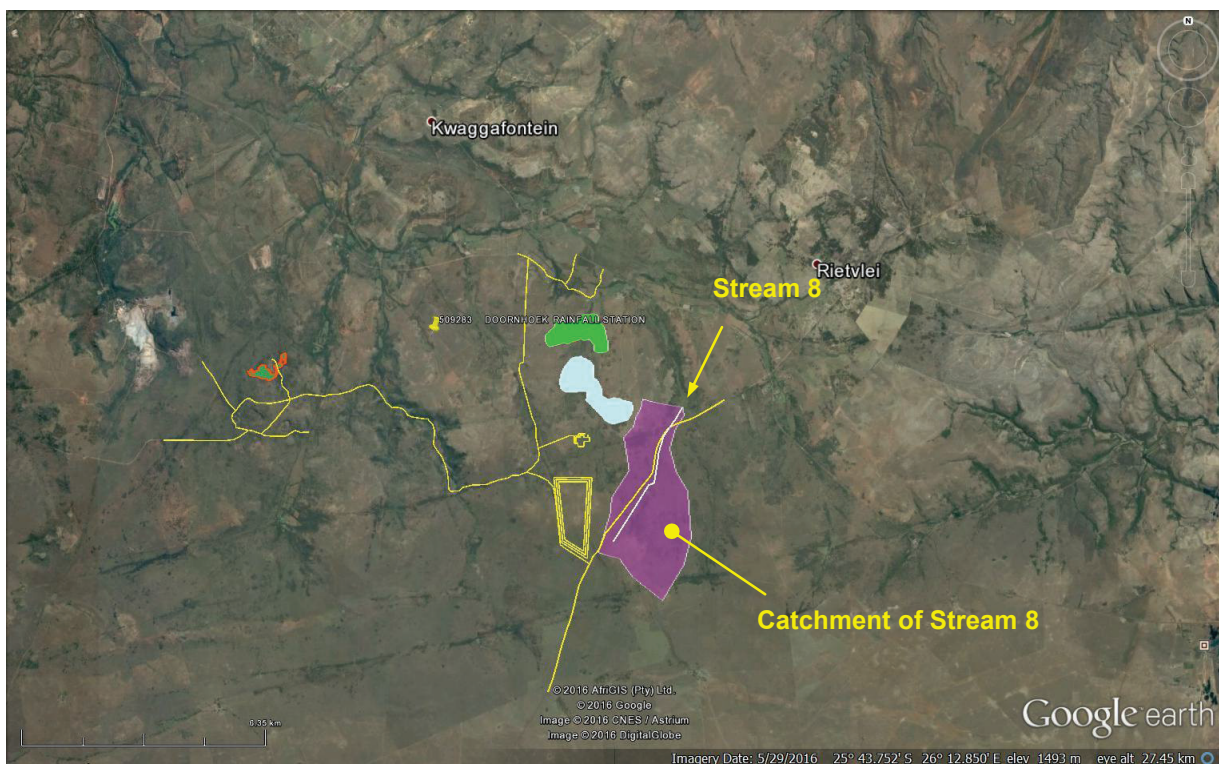


FIGURE 9

4.9.1 Characteristics of Stream 8.

Area of catchment:	7,7 km ²
Length of longest watercourse:	4,0 km
Equal area height difference:	63,4 m
10 – 85 slope height difference:	76,7 m
Distance to catchment centroid:	2,8 km
Time of concentration	45 minutes

Flood Peaks

The same different deterministic methods were used as described above.

The effect of any dam in the catchment was not taken into account because the 1:100 year flood peak will **not** be attenuated by a dam with a smaller storage capacity than 6 *times the total mean annual runoff* of the catchment draining into the dam.

Results of the calculations for Stream 8

The results are listed below in Table 12. The flows indicated are in cubic meter per second. Details of the calculations are shown in Addendum 40.

Return Period Year	Rational method DWA	Rational method Alternative algorithm	SDF method	HRU Algorithm	Unit Hydro-graph
1:50	86	104	78	42	52
1:100	111	126	99	53	72

Table 12

Recommended Flood Peaks for this stream.

The flood peaks were calculated by applying the following algorithm:

$$Q_T = [RMDWA + RMAL + SDF + HRU] / N$$

With:

- Q_T** = Flood peak for return period T
- T** = Return Period
- RMDWA** = Rational method DWA
- RMAL** = Rational method Alternative algorithm
- SDF** = SDF method
- N** = 4

The recommended flood peaks in cumec (cubic meter per second) at the site are listed in Table 13 below:

Return Period Year	Flood peak in Stream 8
1:50	78
1:100	97

Table 13

5 DESCRIPTION OF HYDRAULIC MODEL

5.1 HYDRAULIC MODEL

The HEC-RAS model was used to perform the calculations of the water levels.

HEC-RAS is an integrated package of hydraulic analysis programs, in which the user interacts with the system through the use of a Graphical User Interface (GUI).

HEC-RAS is equipped to model a network of channels, a dendritic system or a single river reach. Certain simplifications must be made in order to model some complex flow

situations using the HEC-RAS one-dimensional approach. It is capable of modeling subcritical, supercritical, and mixed flow regime flow along with the effects of bridges, culverts, weirs, and structures.

5.2 PROCEDURE

The basic computational procedure of HEC-RAS for steady flow is based on the solution of the one-dimensional energy equation. Energy losses are evaluated by friction and contraction / expansion. The momentum equation may be used in situations where the water surface profile is rapidly varied. These situations include hydraulic jumps, hydraulics of bridges, and evaluating profiles at river confluences.

For unsteady flow, HEC-RAS solves the full, dynamic, Saint-Venant equation using an implicit, finite difference method. The unsteady flow equation solver was adapted from Dr. Robert L. Barkau's UNET package.

6 STREAMS GEOMETRY

The geometric models were compiled from the following number of sections:

Stream 1 : 9 Sections
Stream 2 : 11 Sections
Stream 3 : 9 Sections
Stream 4 : 14 Sections
Stream 5 : 8 Sections
Stream 6 : 14 Sections
Stream 7 : 5 Sections
Stream 8 : 6 Sections

Total : 66 Sections

Sections were interpolated at 5 m intervals between all the sections to facilitate the calculation.

7 STORM WATER MANAGEMENT PLAN

7.1 EROSION EVALUATION

The water levels, flood peak calculations and sections are included in the Addenda.

The positions of the sections used in the calculation are shown in the addenda as listed in the Index. The proposed development will interfere with the run-off patterns along some of the streams. Structures to prevent erosion are shown in Addendum 41.

7.2 STREAM 1

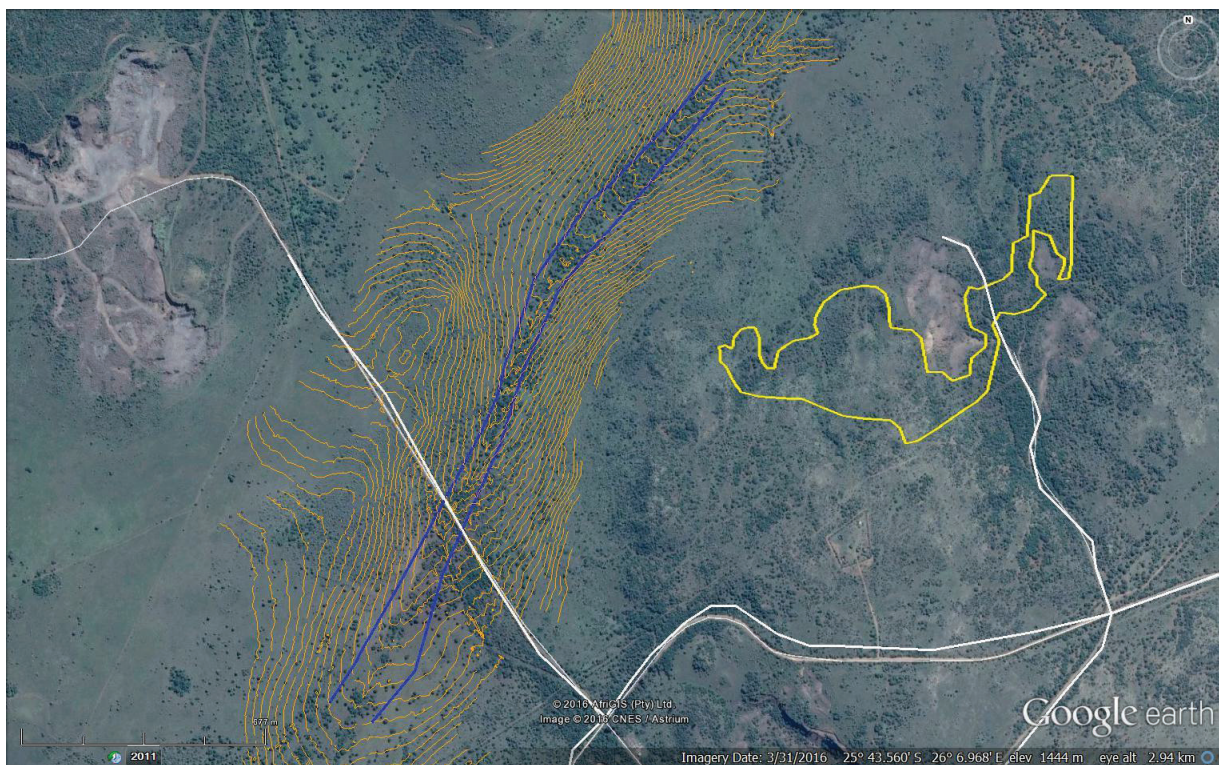


Figure 10

The 1:100 year flood (**103 m³/s**) that will be generated in this stream will cause flooding of the existing road and flow velocities of **2 m/s** can be expected. Measures to control erosion as well as a designed culvert will be necessary if this road is to be used for mining operations.

7.3 STREAM 2

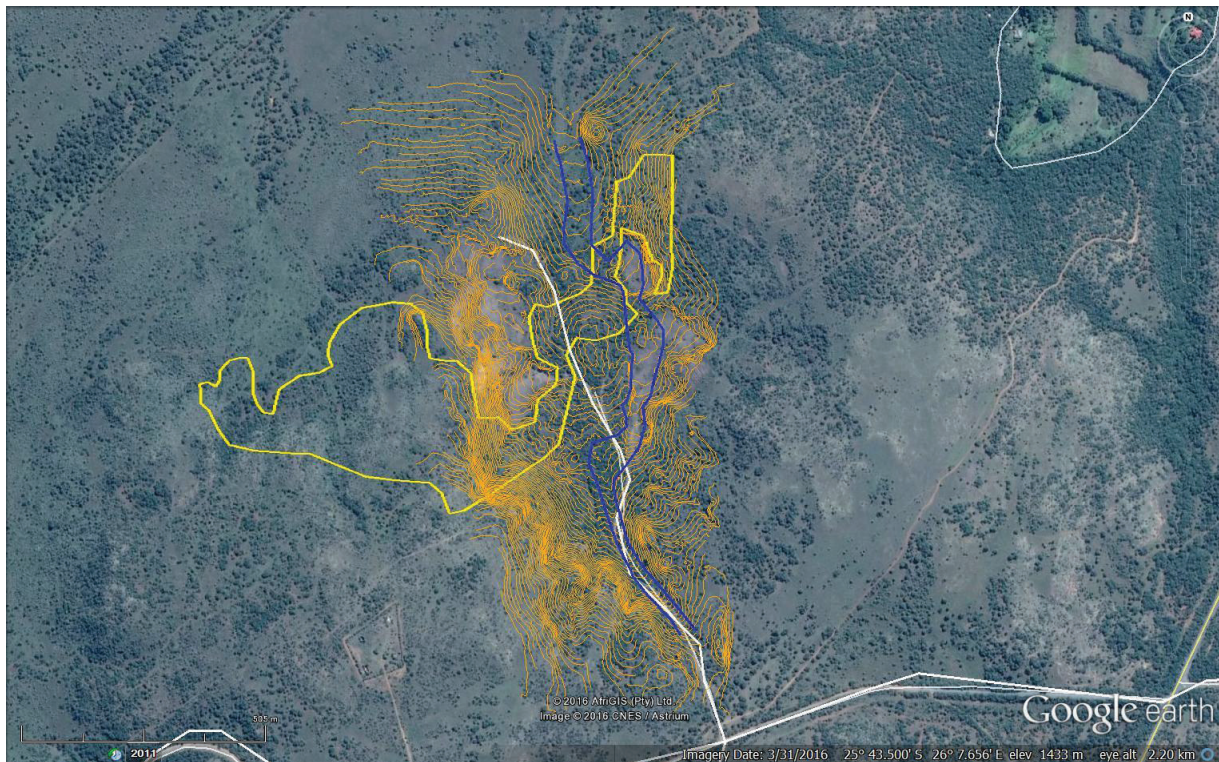


Figure 11

The 1:100 year flood (**29 m³/s**) will flood most of the existing road and will cause flooding of the mining area. Flow velocities from **2 m/s to 3 m/s** can be expected and will cause medium to severe erosion of the road. The possibility to re-align this road or design another access road should be investigated.

The flood will have to be diverted to prevent flooding of the proposed open cast mining operations.

The design of the diverting works is necessary before any mining can commence at this site.

7.4 STREAM 3

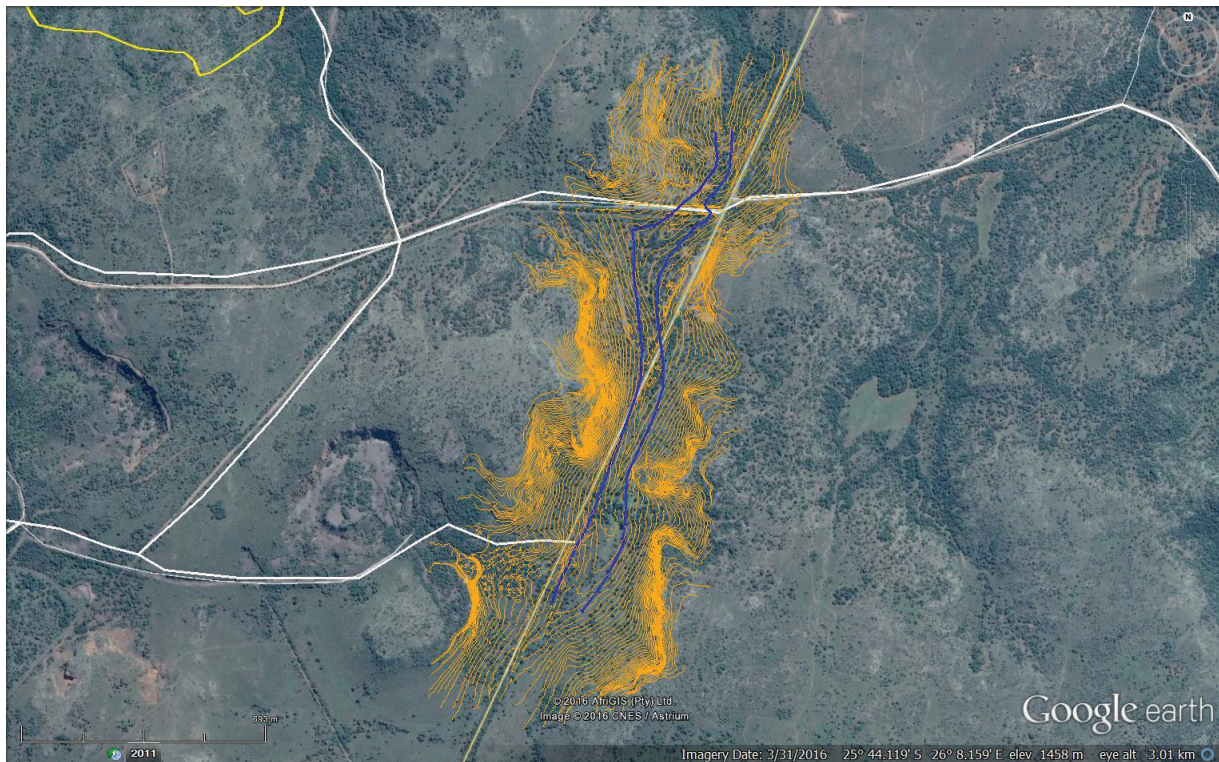


Figure 12

The 1:100 year flood will overtop the existing road as shown above in Figure 12. The flood peak will be **170 m³/s** which will cause flow velocities of **3 m/s to 4 m/s** and may cause severe erosion if not controlled.

The design of a culvert bridge is necessary if this road will be used regarding the mining works.

7.5 STREAM 4

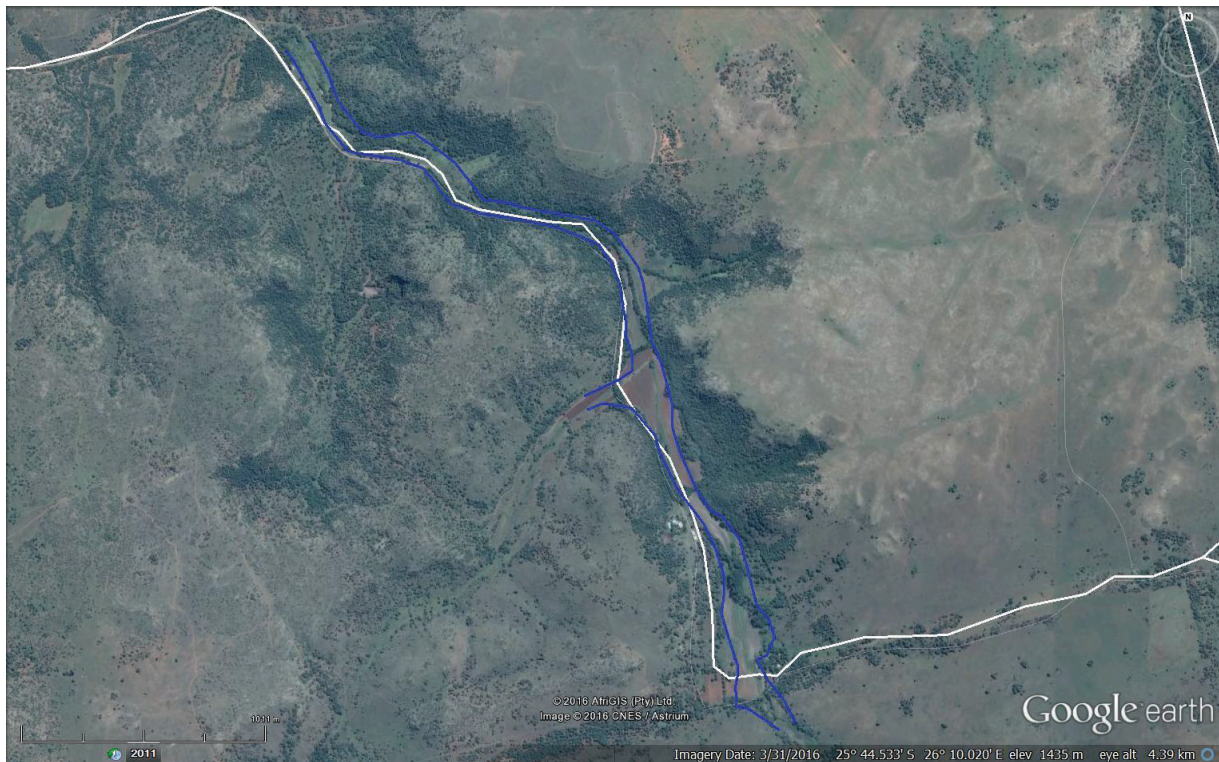


Figure 13

The existing road will be flooded for most of the length of the road. The flood peak will be **238 m³/s** and the flow velocities will be above **3 m/s** which will cause severe erosion to this existing road.

A re-alignment of the road is proposed. If this road will be used for the mining operations, a proper design of the road and reconstruction is necessary.

7.6 STREAM 5

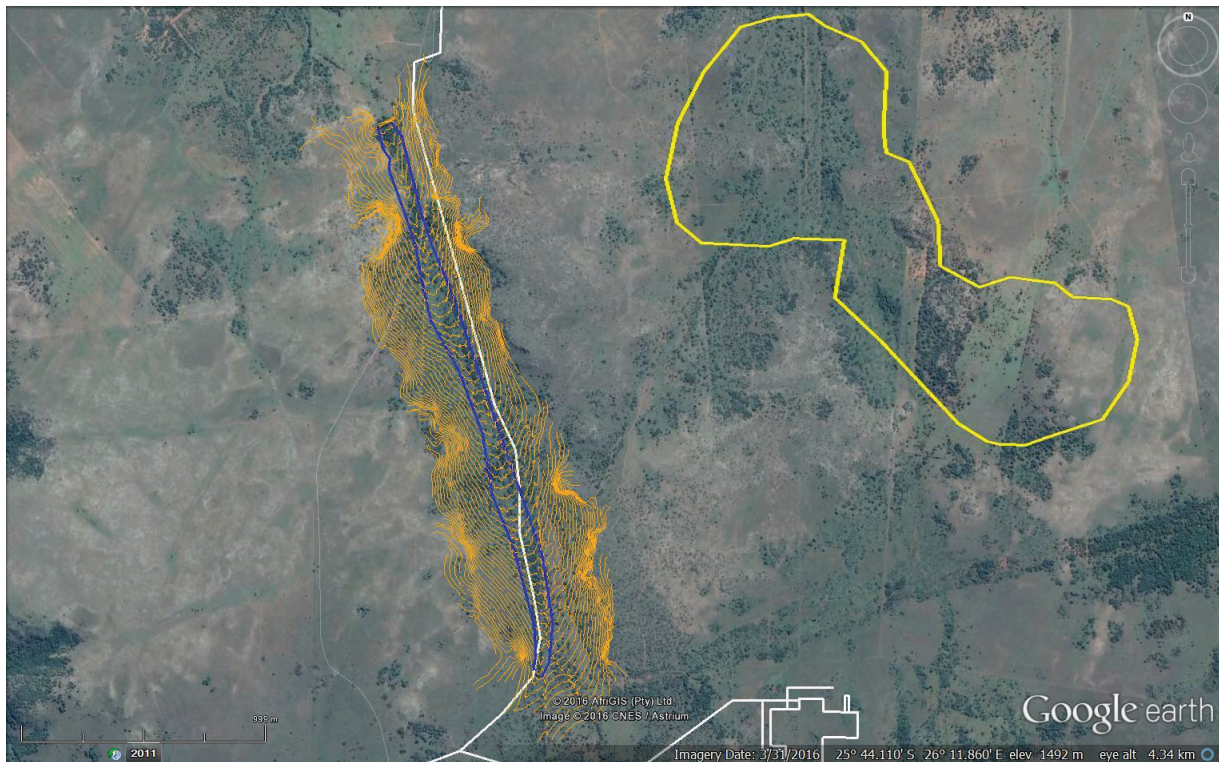


Figure 14

The existing road will be flooded for almost 25% of the length of the road. The flood peak will be **113 m³/s** and the flow velocities will be in the order **3 m/s** which will cause severe erosion to 25 % of this existing road.

A re-alignment of the road is proposed. If this road will be used for the mining operations, a proper design of the road and reconstruction is necessary.

7.7 STREAM 6

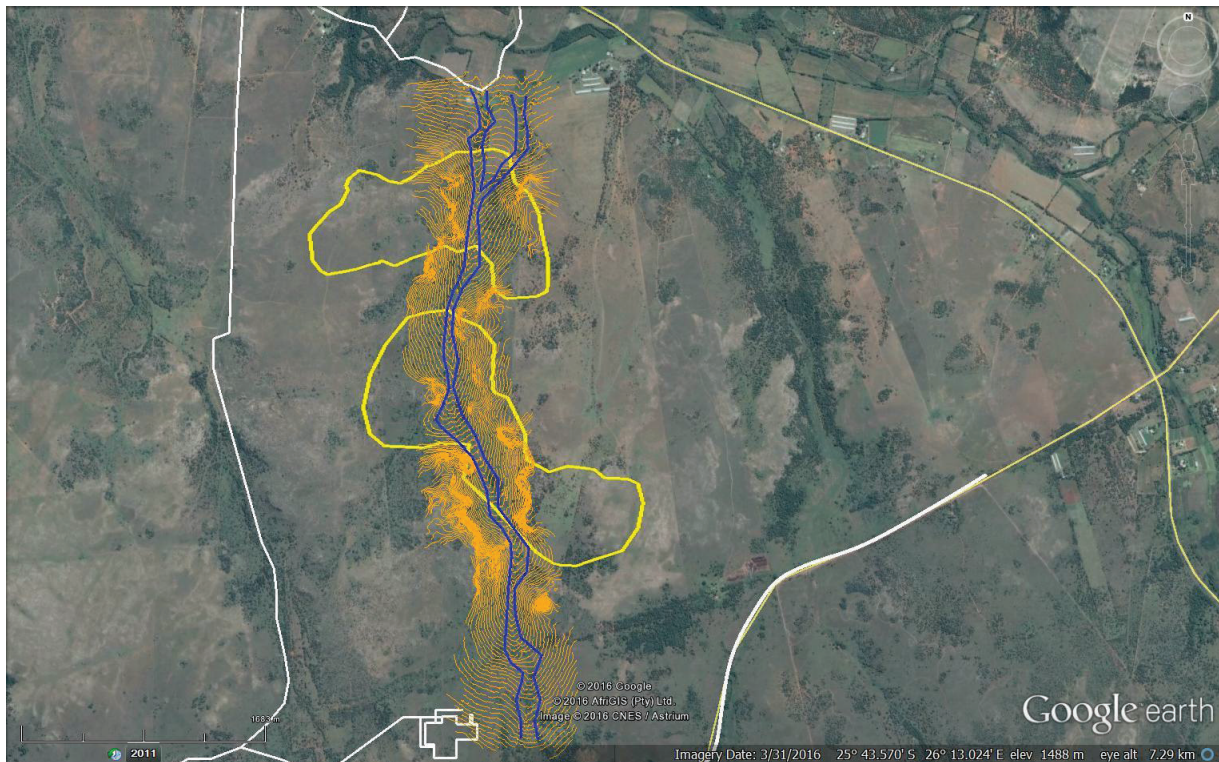


Figure 15

The 1:100 year flood (**119 m³/s**) will cause flooding of the mining area. Flow velocities from **2 m/s to 3 m/s** can be expected.

The flood will have to be diverted to prevent flooding of the proposed open cast mining operations.

The design of the diverting works is necessary before any mining can commence at this site.

7.8 STREAM 7



Figure 16

The 1:100 year flood will overtop the existing roads as shown above in Figure 12. The flood peak will be **470 m³/s**. Flow velocities will be **3 m/s to 4 m/s** and may cause severe erosion if not controlled.

The design of 3 culverts/bridges is necessary if this road will be used for the mining operations.

7.9 STREAM 8

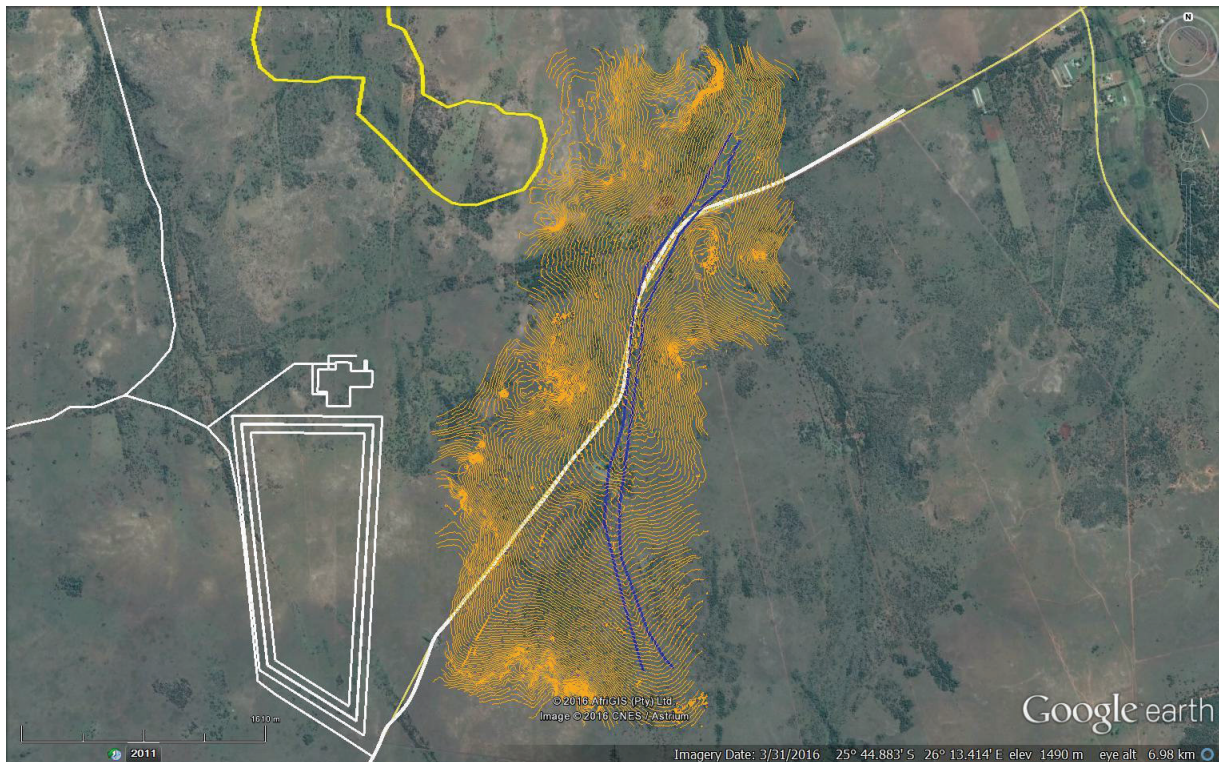


Figure 17

The existing road will be inundated for the length of the road indicated above. The flood peak will be **97 m³/s** and the flow velocities will be in the order **2,5 m/s** which will cause mild to severe erosion of this existing road.

If this road will be used for the mining operations, a proper design of the road and reconstruction is necessary.

7.10 FLOOD DIVERSION WORKS OF STREAM 2

The flood diversion works will include a diversion dam wall as well as 4 smaller dams and a canal system as detailed in Addendum 41.

The layout is shown in Figure 18. The diversion dam wall will be 4m high and will have storage capacity of 5422 m³. The 4 small dams will all be 4 m high. See Addendum 41.

The system as detailed will prevent any storm water from flowing onto the mining area.

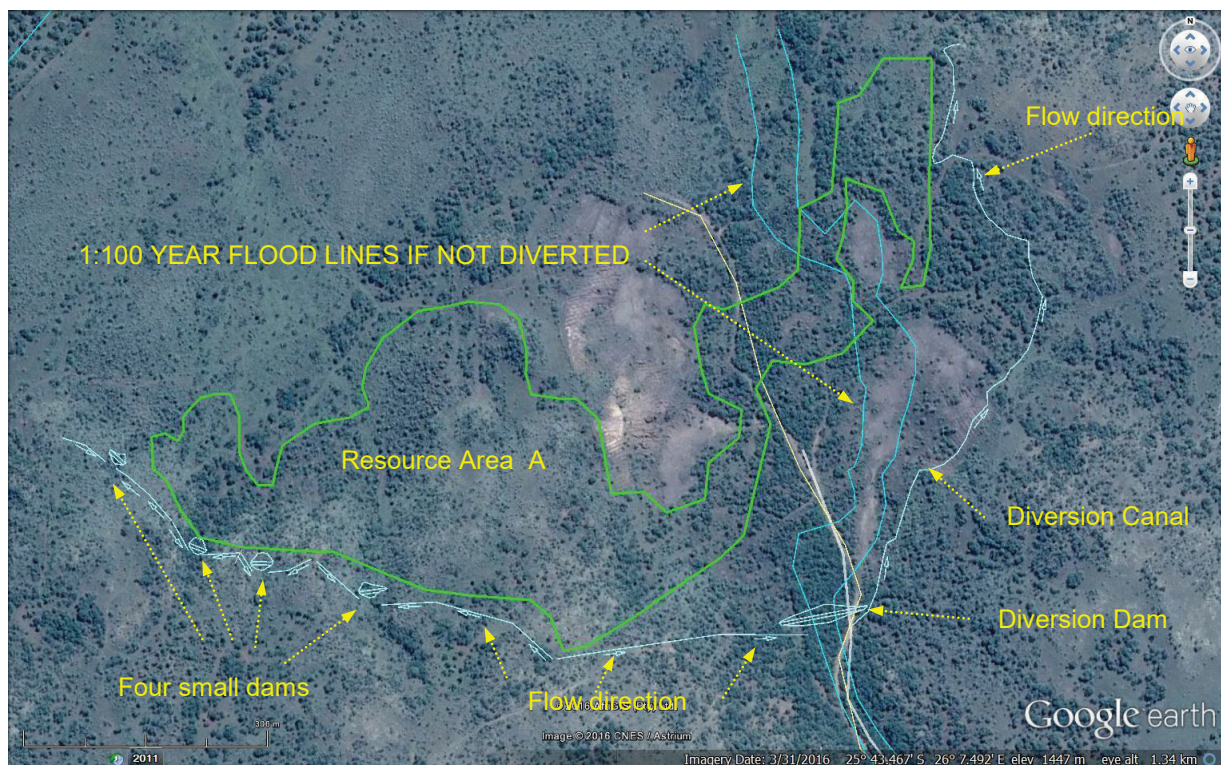


Figure 18

7.11 FLOOD DIVERSION WORKS OF STREAM 6

The flood diversion works is detailed in Addendum 42.

The general layout of the diversion works to prevent storm water flowing into the proposed mining areas is shown in Figure 19.

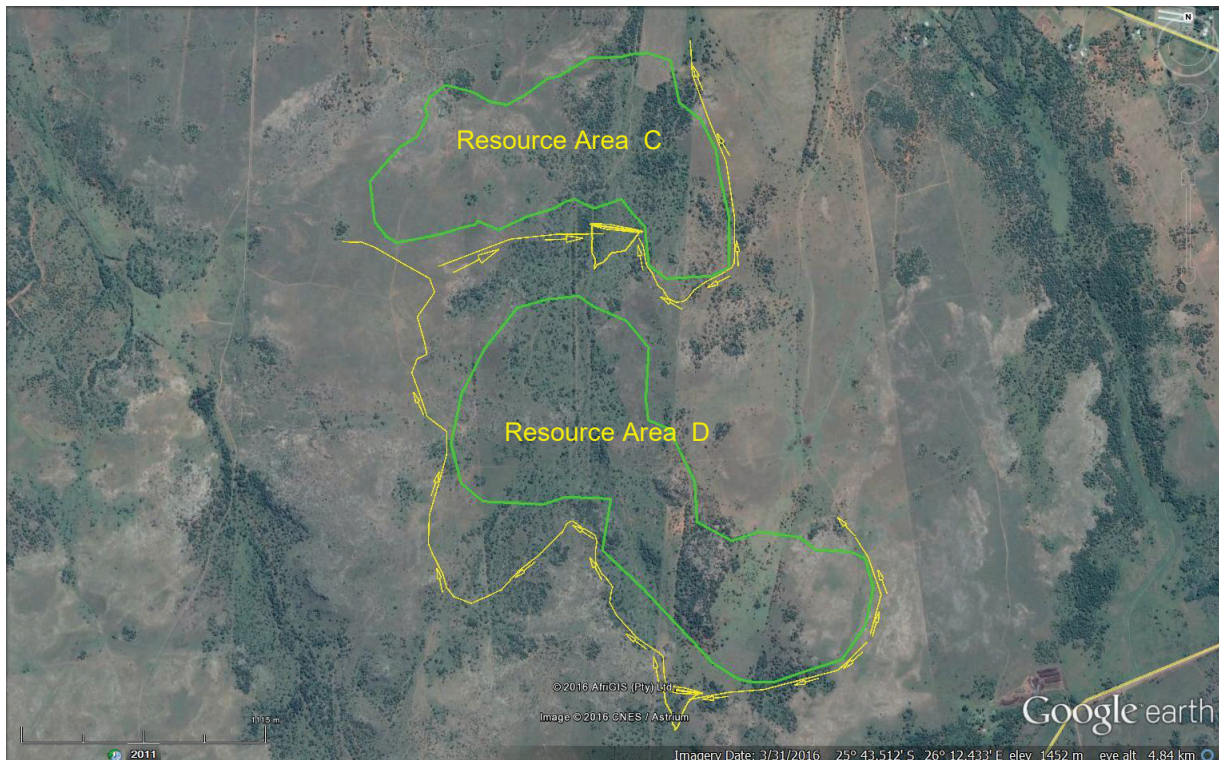


Figure 19

The details of the diversion infrastructure for the two Resource Areas are discussed separately.

Resource Area D

The diversions works will consist of one retention dam and two canals as shown below in Figure 20. The dam wall will be 4 m high and will be able to store 33204 m³.

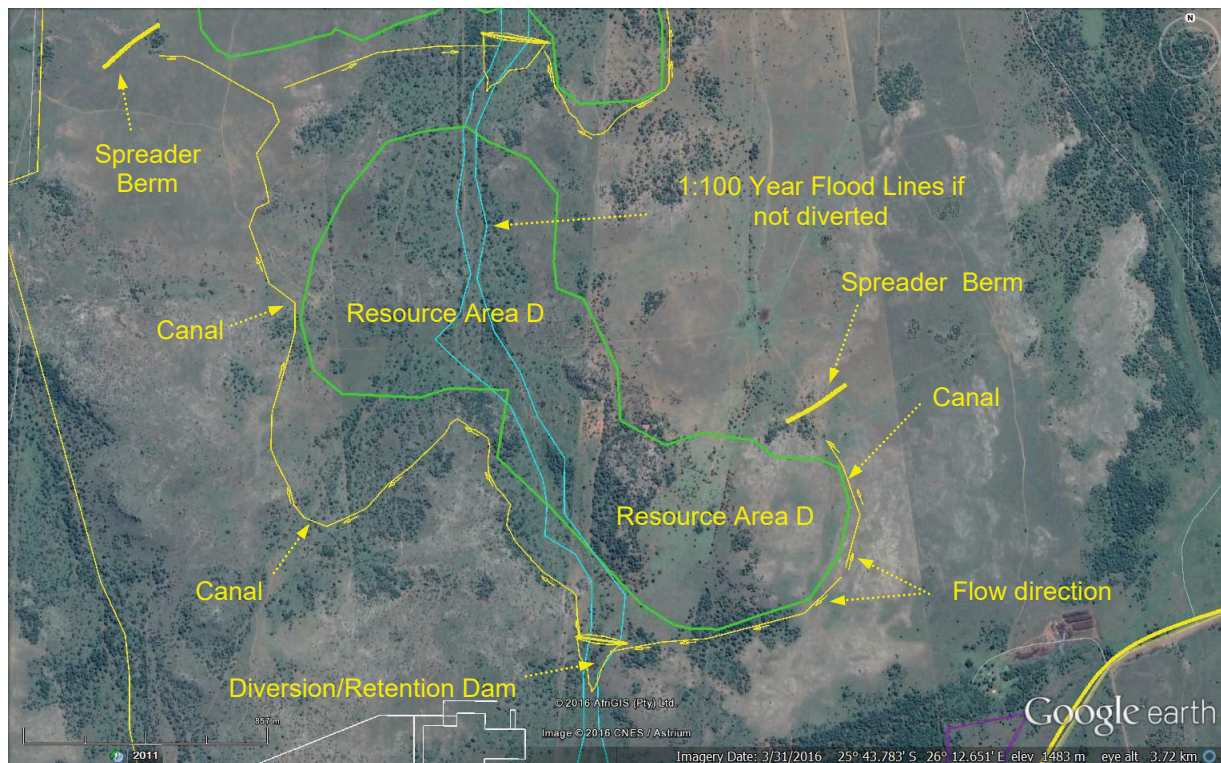


Figure 20

Resource Area C

The diversions works will consist of one retention dam and two canals as shown below in Figure 21. The dam wall will be 4 m high and will be able to store 56900 m³.

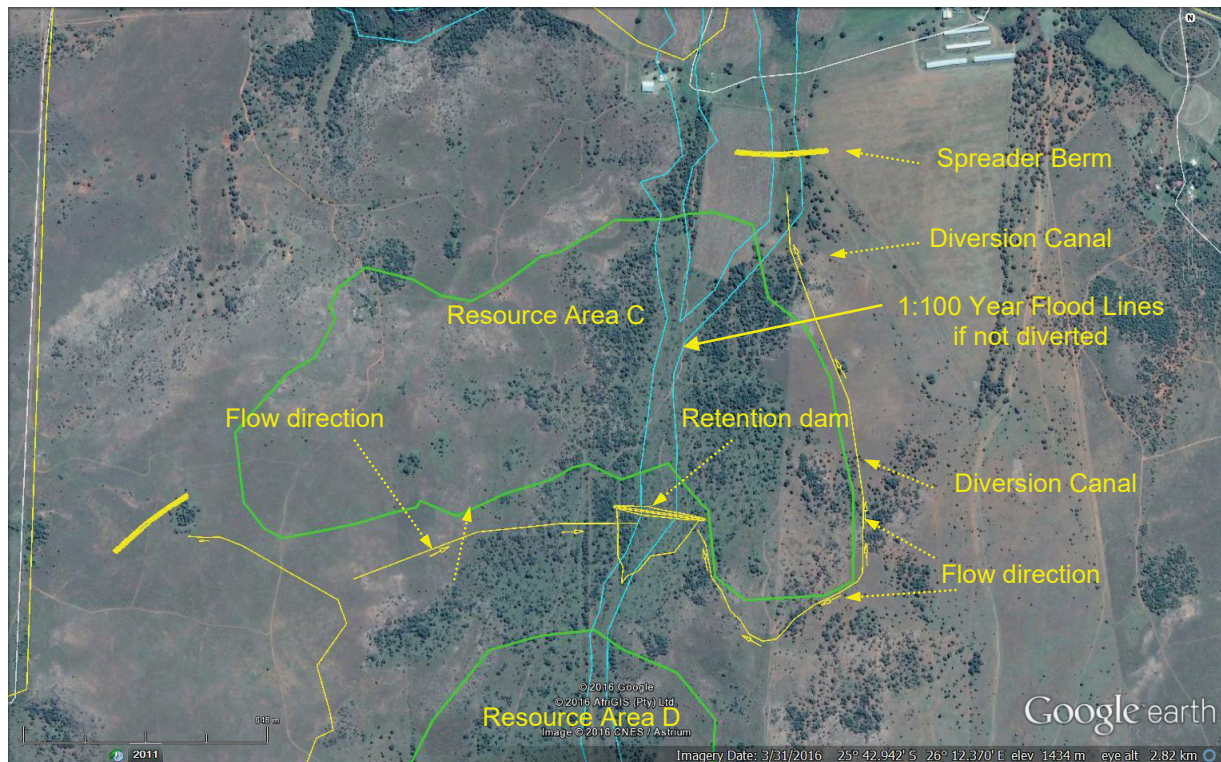


Figure 21

8. DIRTY WATER

Four different layout options regarding the mining plant, offices, parking and ore dump sites were investigated. All storm water and other effluent originating from these areas were treated as dirty water. The dirty water will have to be intercepted and prevented from flowing into the clean water systems after a rain storm. The four layouts of the four proposed systems to prevent contamination of the clean water systems are discussed below. The two proposed layouts of the overburden dumps were also investigated.

The proposed infrastructure to intercept and contain dirty water will consists of berms designed to intercept and contain both surface and sub-surface flow as well as dams to permanently store dirty effluent.

The details of the system can be found in Addenda 48 to 51.

8.1 Layout Option 1.2

See **Addendum 48** for more detail and the positions of the canals to intercept dirty water. Apart from the canals three dams are also needed.

The dam wall heights will vary from 3,0 m to 4,5 m.

The total combined storage capacity will be 110 500 m³.



Figure 22

8.2 Layout Option 2.2

See **Addendum 49** for detail and the positions of the canals to intercept dirty water.

Apart from the canals four dams are also needed.

The dam wall heights will vary from 3,0 m to 4,5 m.

The total combined storage capacity will be 129 210 m³.

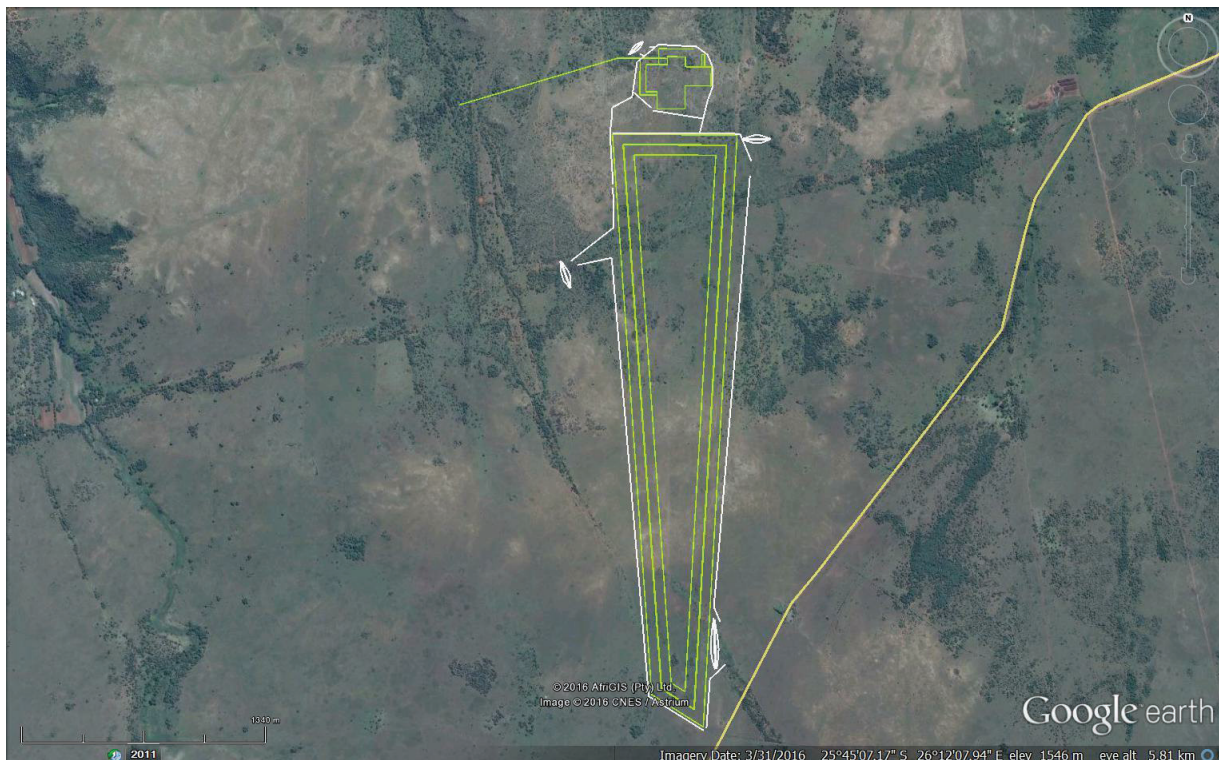


Figure 23

8.3 Layout Option 3.1

See **Addendum 50** for detail and the positions of the canals to intercept dirty water.

Apart from the canals four dams are also needed.

The dam wall heights will vary from 2,0m to 4,5m.

The total combined storage capacity will be 119 300 m³.

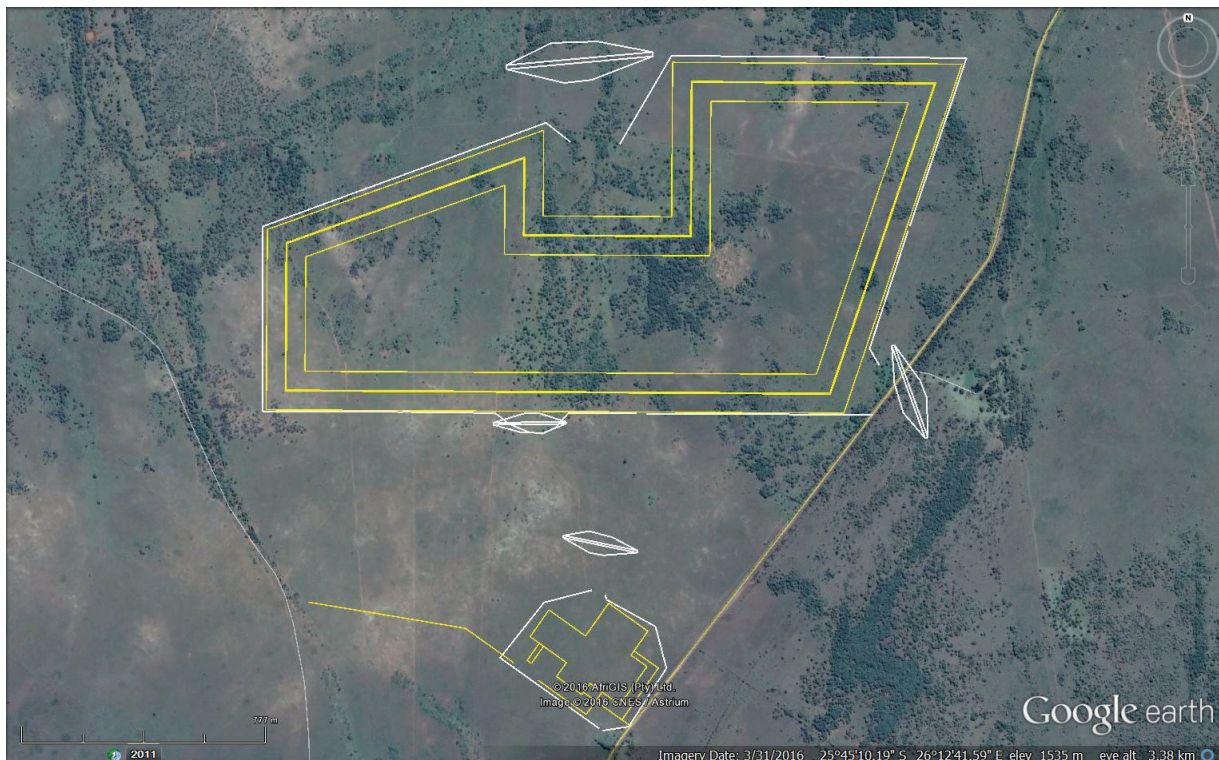


Figure 24

8.4 Layout Option 4.2

See **Addendum 51** for detail and the positions of the canals to intercept dirty water.

Apart from the canals four dams are also needed.

The dam wall heights will vary from 2,5m to 4,5m.

The total combined storage capacity will be 149 010 m³.

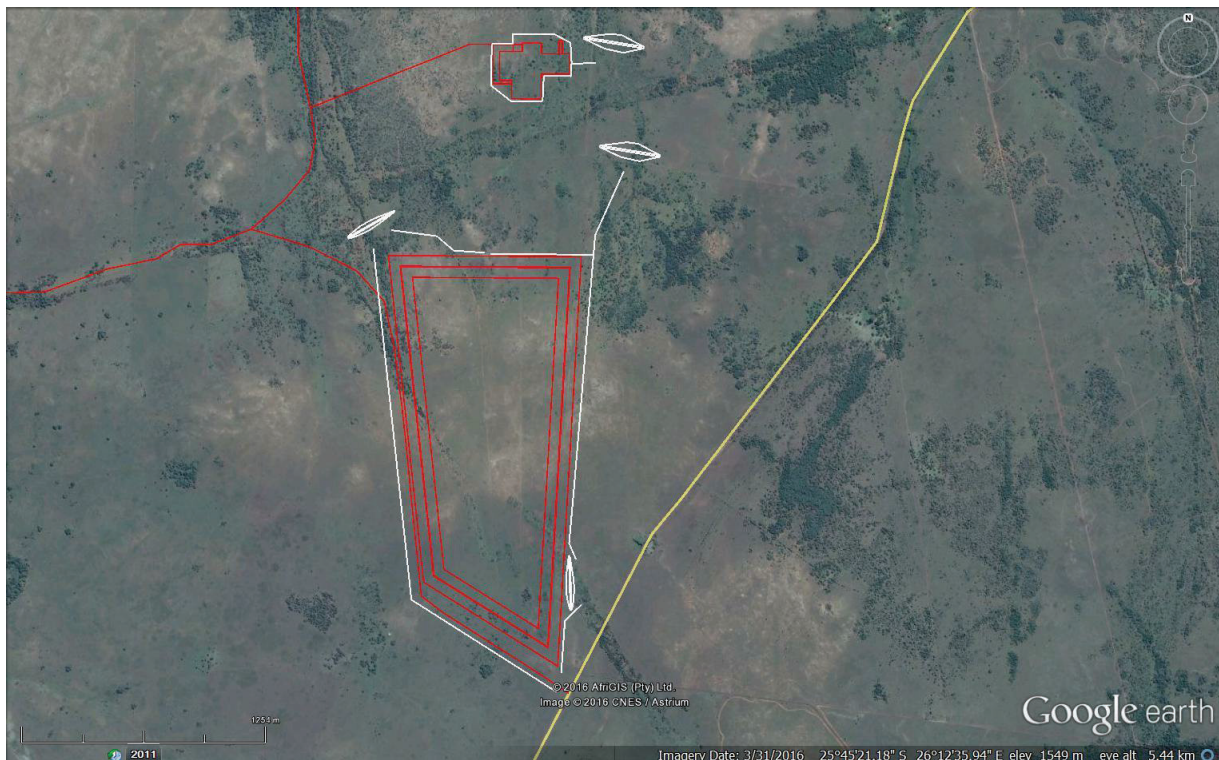


Figure 25

8.5 Overburden Dump at Resource Area A

See **Addendum 52** for detail and the positions of the canals to intercept dirty water.

Apart from the canals a dam is also needed.

The dam wall height will be 2,5m.

The storage capacity will be 1 700 m³.

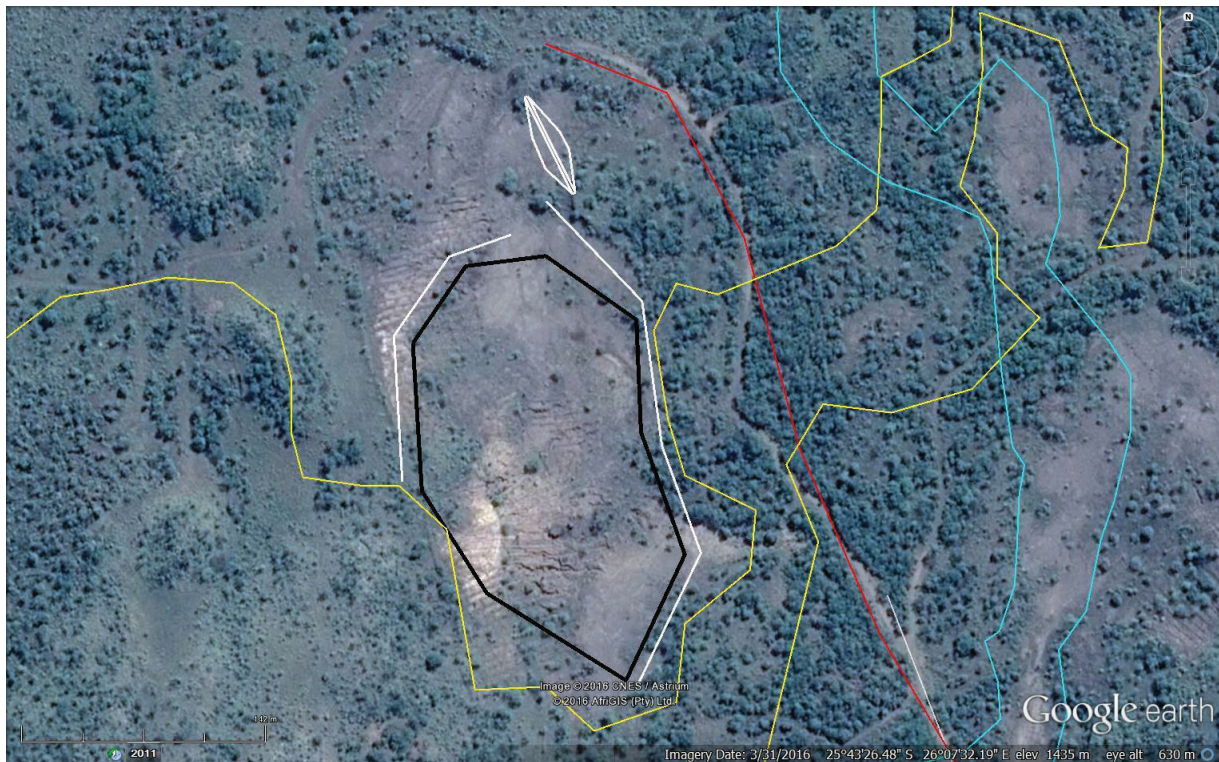


Figure 26

8.5 Overburden Dump at Resource Area C

See **Addendum 53** for detail and the positions of the canals to intercept dirty water.

Apart from the canals a dam is also needed.

The dam wall height will be 2,5m.

The total combined storage capacity will be 19 000 m³.

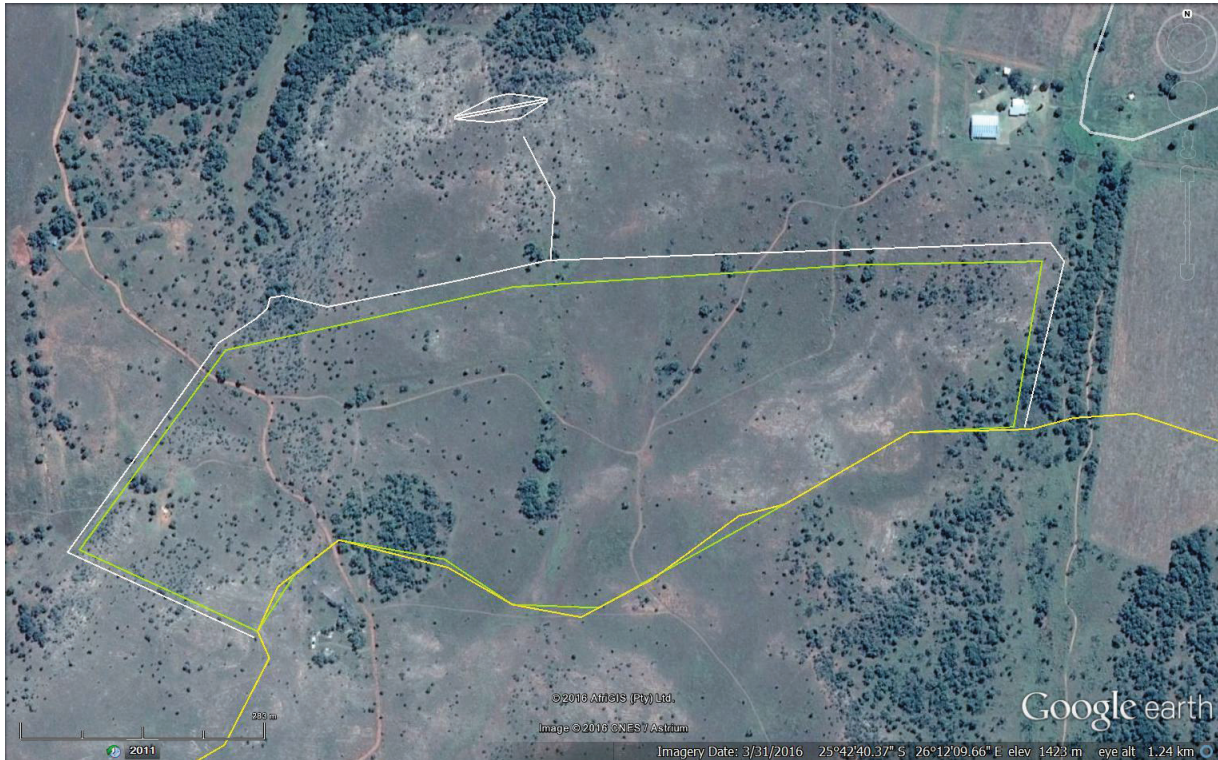


Figure 27

9. MAINTENANCE

It is of prime importance that the storm water management system is well maintained and operated in accordance with the intentions of the design. Checks should be done regularly for erosion damage to embankments.

The erosion protection works as shown on the plans will be able to withstand erosion during the 1:100 year storm occurrence.

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Pr. Eng

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5 October 2016