JG AFRIKA
EXPERIENCE QUALITY INTEGRITY

ROAN SOLAR PV 2 FACILITY

STORMWATER MANAGEMENT PLAN

APRIL 2022
REVISION 2

Prepared by:

JG AFRIKA (PTY) LTD

CAPE TOWN
P.O. Box 38561
Pinelands, 7430
021 530 1800
Email: capetown@jgafrika.com
Project director: M. Manson-Kullin

VERIFICATION PAGE	Form 4.3.1
	Rev 13

TITLE:
ROAN SOLAR PV 2 FACILITY: STORMWATER MANAGEMENT PLAN

JGA REF. NO.	**DATE:**	**REPORT STATUS**
5718/SW/PV2	20/04/2022	REV 2 (for issue)

CARRIED OUT BY:	**COMMISSIONED BY:**
JG AFRIKA (PTY) LTD **CAPE TOWN** P.O. Box 38561 Pinelands, Cape Town 7430 Tel.: +27 21 530 1800 Email: capetown@jgafrika.com	**AMDA November (Pty) Ltd** 302E Sunclare Building 21 Dreyer Str Claremont 7700 Tel: +27 63 296 5255 Email: raymond@amdadevelopments.co.za
AUTHOR	**CLIENT CONTACT PERSON**
G Joubert Pr Eng	Raymond Takuba

SYNOPSIS

Stormwater Management Plan for the proposed Roan solar PV 2 development near Hartbeesfontein, North West Province.

KEY WORDS:

Hydrology, Stormwater, Stormwater management, Solar.

QUALITY VERIFICATION

This report has been prepared under the controls established by a quality management system that meets the requirements of ISO9001: 2015 which has been independently certified by DEKRA Certification under certificate number 90906882

Verification	Capacity	Name	Signature	Date
By Author	Associate	G. Joubert		23/02/2022
Checked by:	Technical Director	D. Petersen		23/02/2022
Authorised by:	Technical Director	D. Petersen		23/02/2022

Filename:	X:\Projects - (In-progress)\5718 - Roan Solar PV SWMP (MMK)\103-Report only Projects\04-Docs & Reports\Reports\5718 Roan PV2 SWMP Rev02_20220420.docx

ROAN SOLAR PV 2 FACILITY
STORMWATER MANAGEMENT PLAN

TABLE OF CONTENTS

TABLES

FIGURES

ANNEXURES

1 TERMS OF REFERENCE

JG Afrika (Pty) Ltd was appointed by AMDA November (Pty) Ltd to provide a Stormwater Management Plan for the proposed Roan Solar PV facilities located south of Hartbeesfontein, North West Province. This report focusses on the western site, designated Roan PV 2.

This scope of this study includes the following:

- quantification of stormwater runoff and peak flows;
- development of strategies for stormwater management;
- Analysis of design concepts to accommodate the anticipated runoff, while ensuring continuity of natural drainage paths; and
- determination of appropriate mitigation measures, including erosion management, attenuation of flood peaks and pollution control.

2 DESCRIPTION OF THE SITE

2.1 Location

The proposed development centroid is located approximately 6 km south of Hartbeesfontein and the R507/503 Regional / Main Road linking Ottosdal and Klerksdorp, as indicated on **Figure 2-1: Locality**.

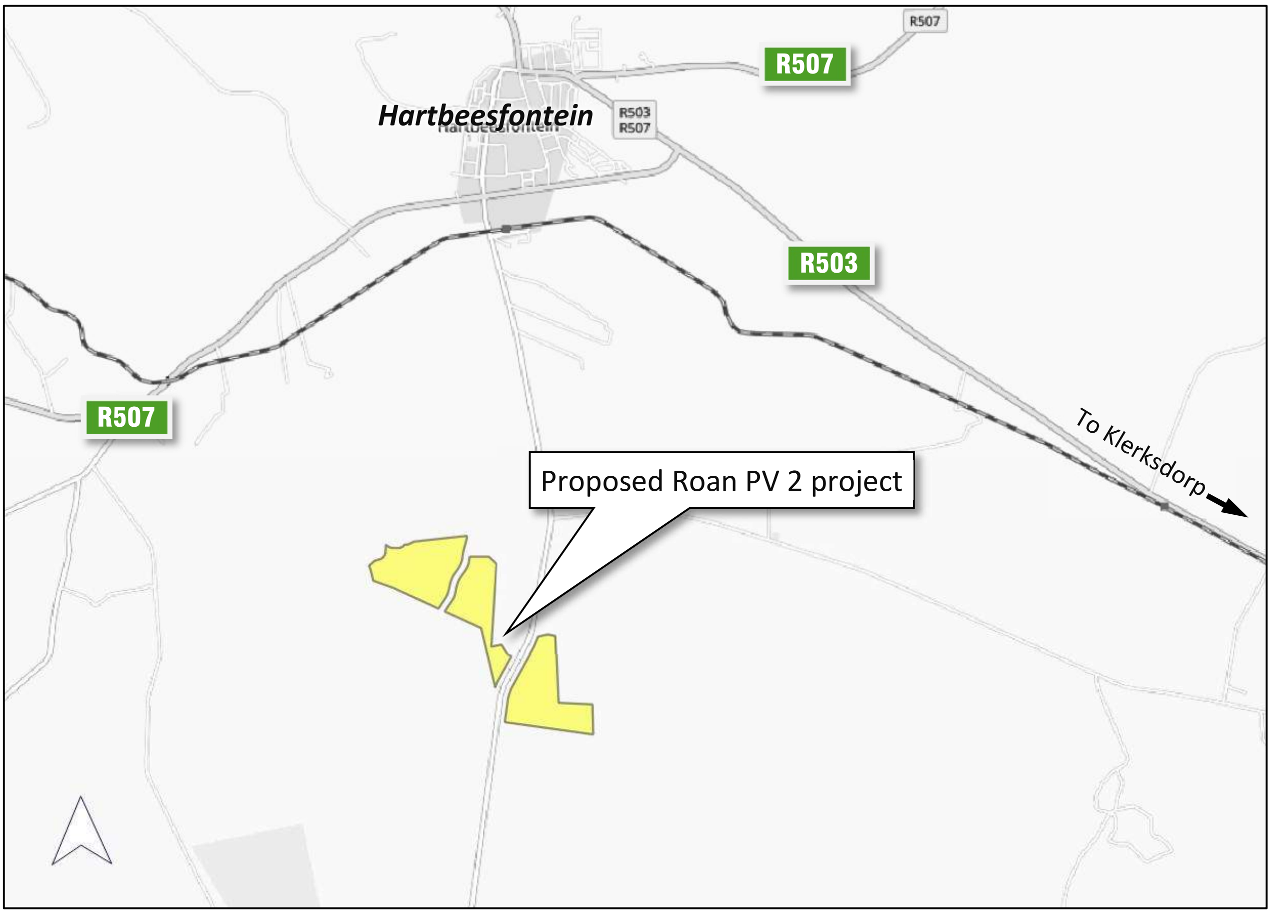


Figure 2-1: Locality

The proposed solar PV development is made up of 3 areas separated in a north / south direction by a non-perennial stream and a secondary road, respectively. The development extends over an area of 202 ha as shown on **Figure 2-2: Site**.

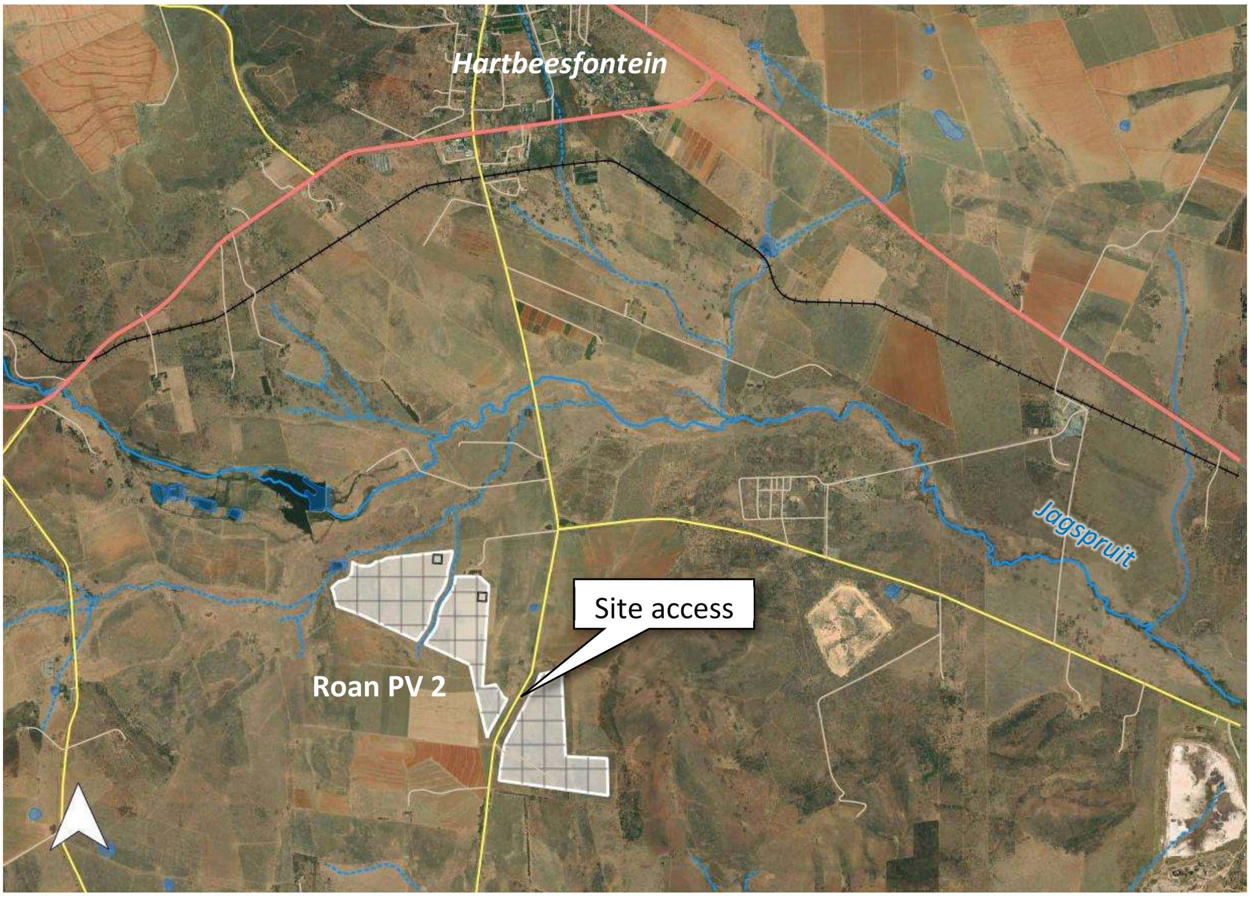


Figure 2-2: Site

2.2 Infrastructure

The site is currently accessed via an existing secondary road that links to Hartbeesfontein to the north as well as several gravel farm access roads that intersect with the secondary road. There are several proposed access points to the development. One such access in indicated above on **Figure 2-2: Site** and is shown on **Figure 2-3: Access**.

The secondary road is asphalt surfaced and its upkeep is the responsibility of the provincial roads authority.

Although there are widely-spaced culvert crossings beneath the secondary roads and gravel tracks adjacent to the proposed site, there is no other formal stormwater infrastructure and runoff is conveyed overland in open earth channels via preferential drainage routes.

2.3 Topography

The topography was assessed using digital elevation model (DEM) data from the 1:10 000 Orthophoto series provided by National Geo-Spatial Information (Department of Rural Development and Land Reform).

There is a ridge line to the east of the site, but the remainder of the catchment is made up of relatively flat (less than 1%), undulating terrain. This ridge acts as a watershed diverting runoff westwards across the eastern portion of the site. In the vicinity of the ridge, the topography is variable and consists of portions that are relatively steep (> 3%) areas with more obvious drainage routes.

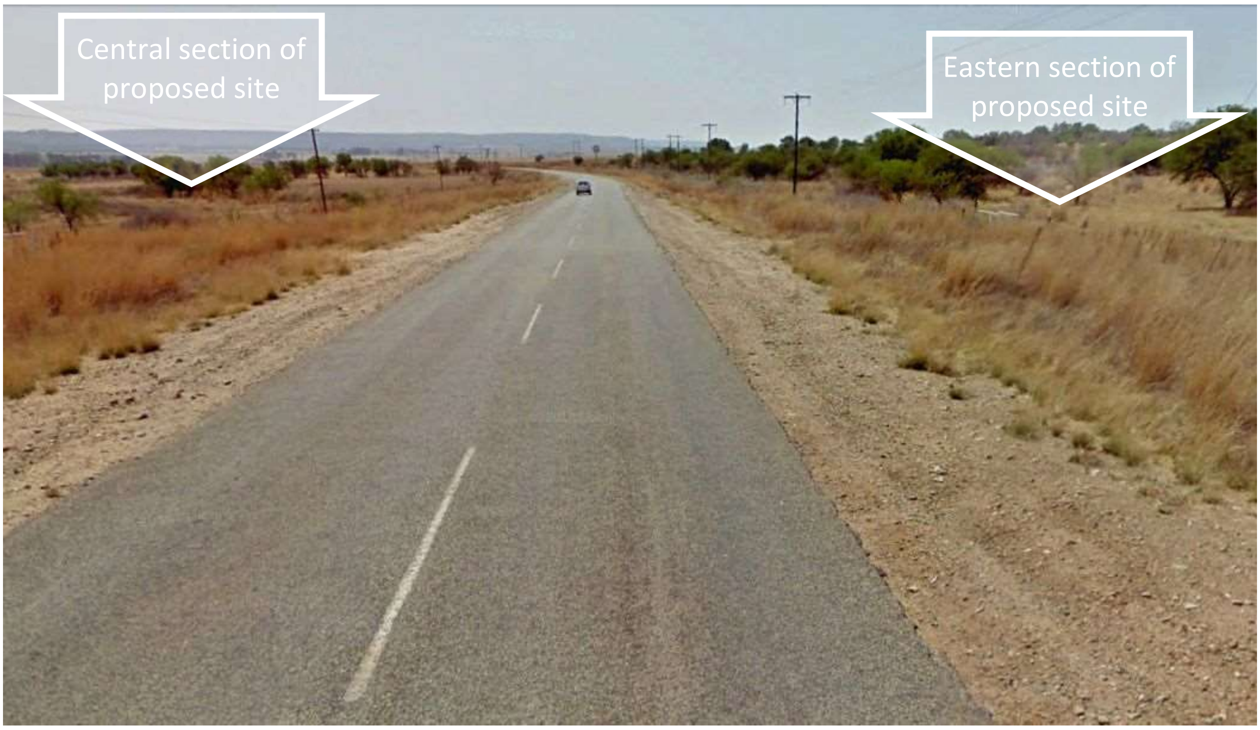


Figure 2-3: Access (secondary road heading north)

The aerial photography indicates the presence of farm dams and minor localised depressions where stormwater accumulates during rainfall events, although these are not evident from the contours.

The bulk of the stormwater from the catchment drains towards the Jagspruit River, which is located immediately to the north, but there is a minor component that is directed in a southerly direction into an unnamed non-perennial stream.

The topography is indicated on **Figure 2-4**.

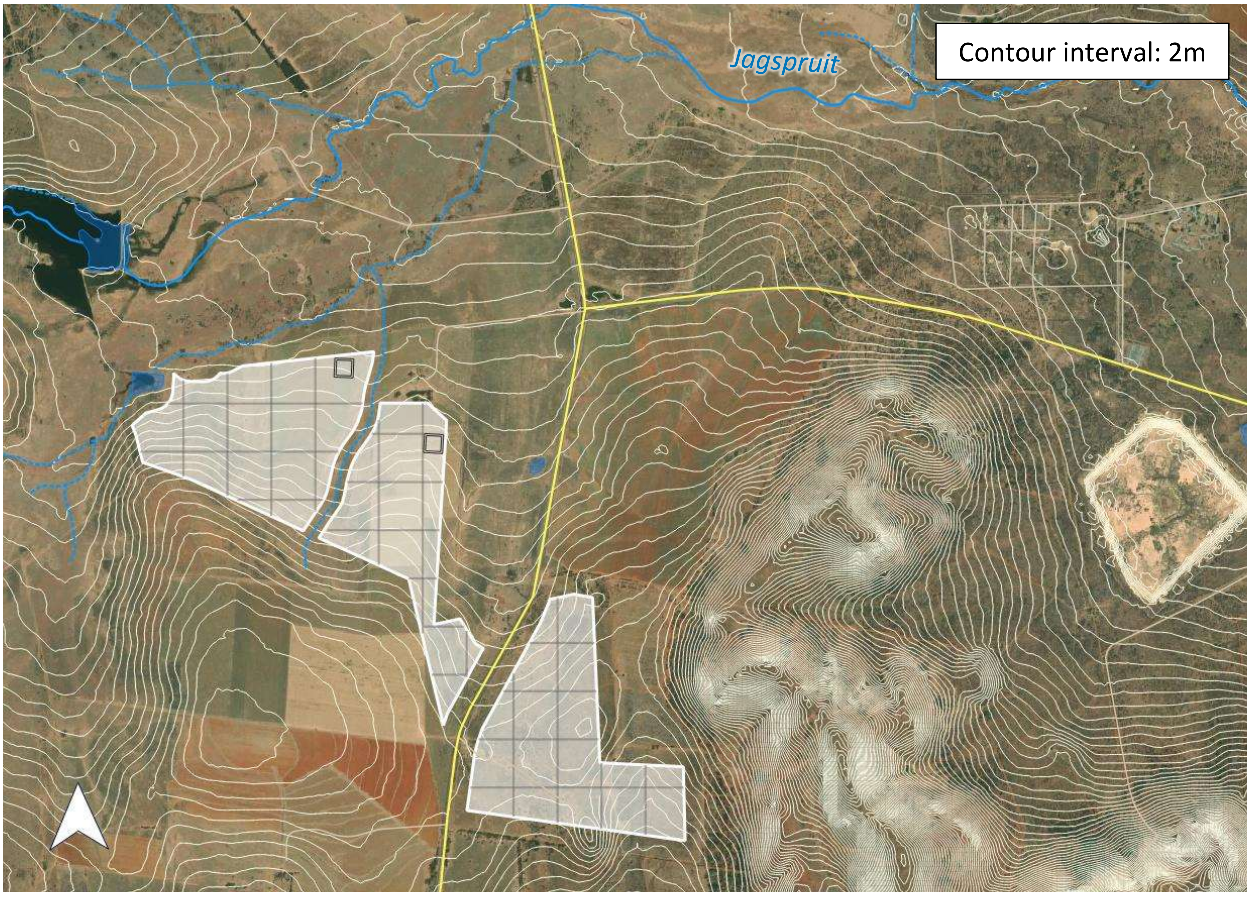


Figure 2-4: Topography

2.4 Catchment

The two catchments draining the area affecting the proposed development is shown on **Figure 2-5: Catchment**. The main catchment covers an area of 6.2 km^2 and the southern catchment (Sub-catchment 08) extends over an area of 3.1 km^2. Together, the total area drained is 9.4 km^2.

For modelling purposes, the two catchments were subdivided into 8 smaller sub-catchments. The modelling is covered in more detail in **Section 3**.

Even though there are relatively steep zones in the catchment, the majority of aboveground runoff is likely to be in the form of shallow sheet flow and consequently, flow velocities will be relatively low.

The proposed development does not encroach on the floodplain of the Jagspruit.

The above assessment is of a high level and appropriate to the scope of the study. Detailed survey will be required to determine the actual dimensions of drainage paths, but examination of the available topographical information and aerial photography reveals no obvious areas where erosion is taking place.

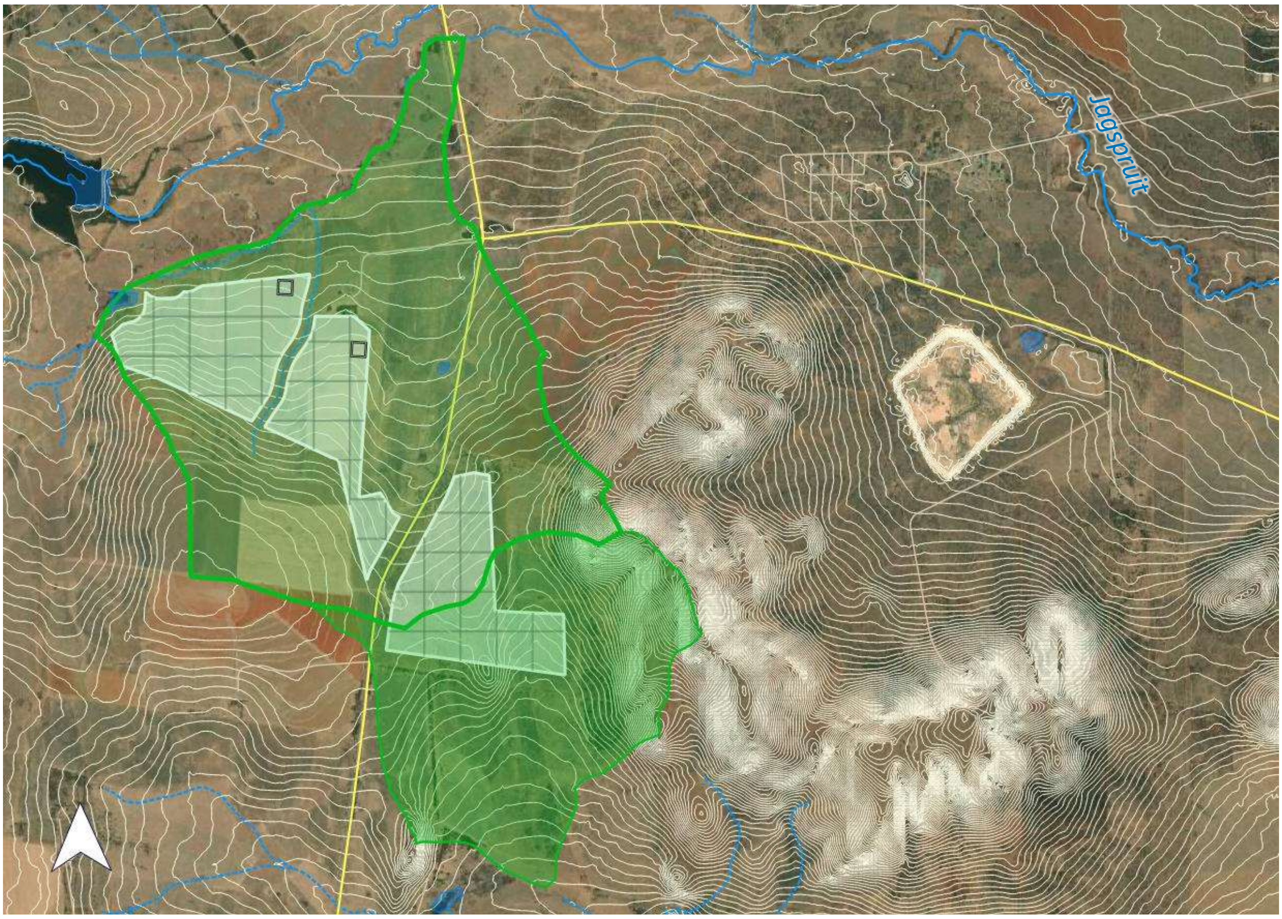


Figure 2-5: Catchment

2.5 Soils

Detailed geotechnical testing would be required to determine the necessary infiltration parameters for explicit groundwater modelling, but in terms of general hydrological response, the soils in the catchment fall into a single broad category.

The catchment consists of soil of intermediate depth (500mm - 1000mm) Hutton Form (Hu) that is mainly a combination of the Clansthal (Hu 24) and Msinga (Hu 26) Series. In terms of Textural Class, it is classified as sandy loam / sandy, clayey loam with a Soil Conservation Service (SCS) Grouping ranging from A to B and a low to moderate runoff potential.

2.6 Vegetation

The vegetation across the catchment is grassland interspersed with scrub and scattered shrubs, as indicated on **Figure 2-7**: **Vegetation**. There are portions that have been used for agriculture and where row cropping was used.

In hydrological terms, it can be classified as a combination of fallow row crops and veld or range in "good" condition.

Figure 2-6: Vegetation

3 MODELLING

The catchment was split into eight sub-catchments for modelling purposes (as shown on **Figure 3-1**) and these were modelled in EPA-SWMM 5.1, using the rainfall and runoff data below. To ensure consistency with the PV 2 project, an integrated model, incorporating the catchments affecting both PV 1 and PV 2 was compiled.

3.1 Design Rainfall

The Intensity-duration-frequency data was derived from *Rainfall Statistics for Design Flood Estimation in South Africa* (Smithers & Schulze. 2012) for reference point 26°49' S; 26°26 E. It is tabulated below for design storm events with return period of 5 and 50 years for various durations:

T (years)	Design storm duration								
	10 min	**15 min**	**30 min**	**1 hr**	**2 hr**	**4 hr**	**8 hr**	**12 hr**	**24 hr**
	Average intensity (mm/h)								
5	113.4	95.2	62.0	34.6	24.2	14.1	8.3	6.0	3.5
50	181.8	152.4	98.0	61.1	38.7	22.6	13.2	9.7	5.6

Table 3-1: Design Rainfall

3.2 Runoff Parameters

The runoff parameters used are listed below:

- Impervious area roughness coefficient: 0.018
- Pervious area roughness coefficient: 0.050
- Impervious area depression storage: 1 mm

- Pervious area depression storage: 5 mm
- Infiltration method: SCS
- SCS Curve Number (CN): Hu Form soils: 61

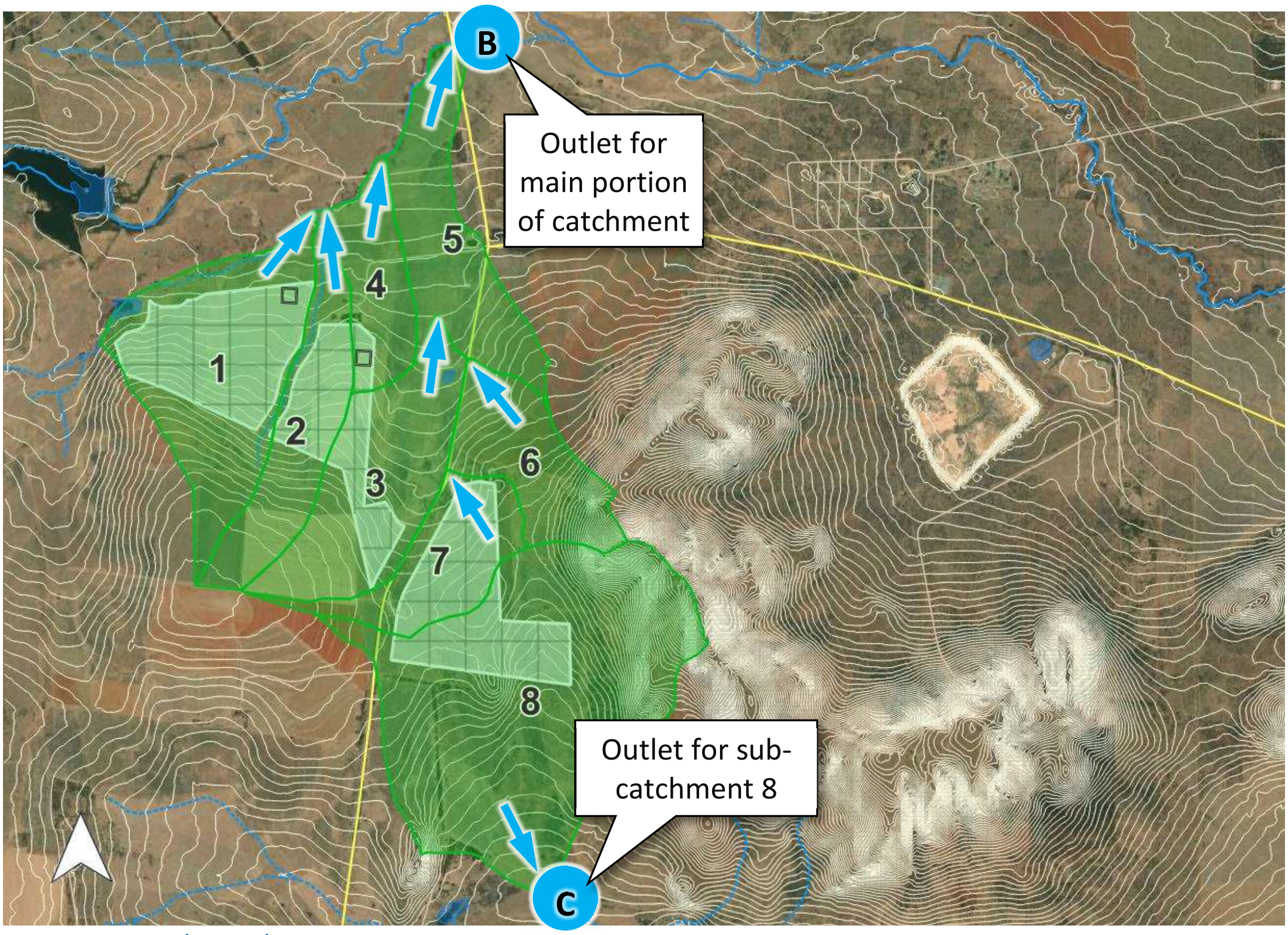


Figure 3-1: Sub-catchments

3.3 Pre-Development Runoff

Runoff was computed for both minor (5-year) and major (50-year) design events of various durations up to 24 hours. The peak flows were cross-checked via the Rational Method and found to be reasonable.

The peak flows are tabulated below:

Sub-Catchment	5-year return period		50-year return period	
	Peak flow (m^3/s)	Critical design storm duration (h)	Peak flow (m^3/s)	Critical design storm duration (h)
S01	1.20	8	3.44	4
S02	0.63	8	1.80	4
S03	0.90	4	2.59	4
S04	0.47	4	1.30	4
S05	0.64	8	1.78	4
S06	0.77	4	2.06	2
S07	0.58	4	1.63	4
S08	2.46	4	7.09	4

Table 3-2: Pre-development (existing) peak flows

Owing to the relatively pervious nature of the soil, the bulk of the rainfall resulting from short duration events infiltrates. It is only once the soil becomes saturated that substantial overland runoff takes place and consequently, longer duration storms produce the highest peak flows. Saturation takes place sooner for high-order events, so the peak flows typically occur for shorter duration design storms versus low-order events.

The peak flows for the various sub-catchments are of a similar order. The topography is such that the runoff is spread out in the lower reaches of the catchment. As a consequence, for design storm events of return period up to 50 years, flow velocities will be low (< 0.5m/s). Flow depths outside of preferential drainage paths are likely to be shallow, but where preferential drainage routes converge to form natural earth channels that are more clearly defined, the depth of flow will increase substantially. Flow depths of up to 1m can be anticipated.

Detailed survey will be required to model specific drainage paths and provide more accurate flow computations.

3.4 Post-Development Runoff

The primary difference between the pre-development and the post-development scenarios is the presence of the solar PV panels and associated infrastructure. The solar PV panels themselves are impervious, but since they are widely distributed and raised above natural ground level, they will not behave like typical hardened surfaces. Essentially, they do not interfere with infiltration to any significant degree and do not obstruct existing flow paths.

This does not apply to the access and internal roads or the site management / plant areas. These form effectively impervious surfaces and thus increase runoff. The increase in impervious area for the post-development scenario was measured using GIS overlays and estimated coverage percentages for the relevant items.

Runoff was computed for both minor (5-year) and major (50-year) design events of various durations up to 24 hours.

The peak flows are tabulated below:

Sub-Catchment	5-year return period		50-year return period	
	Peak flow (m^3/s)	% increase over pre-dev runoff	Peak flow (m^3/s)	% increase over pre-dev runoff
S01	1.47	22.5	3.91	13.7
S02	0.73	15.9	1.98	10.0
S03	0.99	10.0	2.74	5.8
S03	0.56	19.1	1.43	10.0
S05	0.64	0.0	1.78	0.0
S06	0.77	0.0	2.06	0.0
S07	0.69	19.0	1.81	11.0
S08	2.58	4.9	7.30	3.0

Table 3-3: Post-development sub-catchment peak flows

The increase in runoff from the various sub-catchments over the pre-development situation is small – both in quantity and percentage. Nevertheless, the increase does have an impact on the area downstream of the proposed development.

Consequently, post-development mitigation in the form of detention storage will be required to reduce the peak flows to align more closely with pre-development runoff. It would also be prudent to incorporate sediment management interventions to the detention areas to limit the degradation in the quality of the receiving waters.

In terms of aggregate peak flow, the post-development peaks arriving at Outlet Point B in the Jagspruit River (as indicated on **Figure 3-1: Sub-catchments**) is indicated below in **Table 3-4** and compared with the reduced post-development runoff that would result from the introduction of detention storage facilities to mitigate the peak flows.

Detention storage	50-year return period max peak flow (m^3/s)		Change	
	Pre-development	Post-development	(m^3/s)	%
No	13.15	14.28	1.13	8.6
Yes	13.15	12.80	-0.35	-2.7

Table 3-4: Change in maximum peak flow at Outlet B

3.5 Post-Development Storage

The storage requirements to reduce the peak flows from the 5-year return period design storm would fit within the footprint of a detention pond designed to accommodate flow from a 50-year return period design storm. Consequently, only the storage requirements for the 50-year event are reported on here.

The optimal locations for the detention ponds (indicated below on **Figure 3-2: Detention Pond Locations** as Storage Nodes 108, 118 and 228).

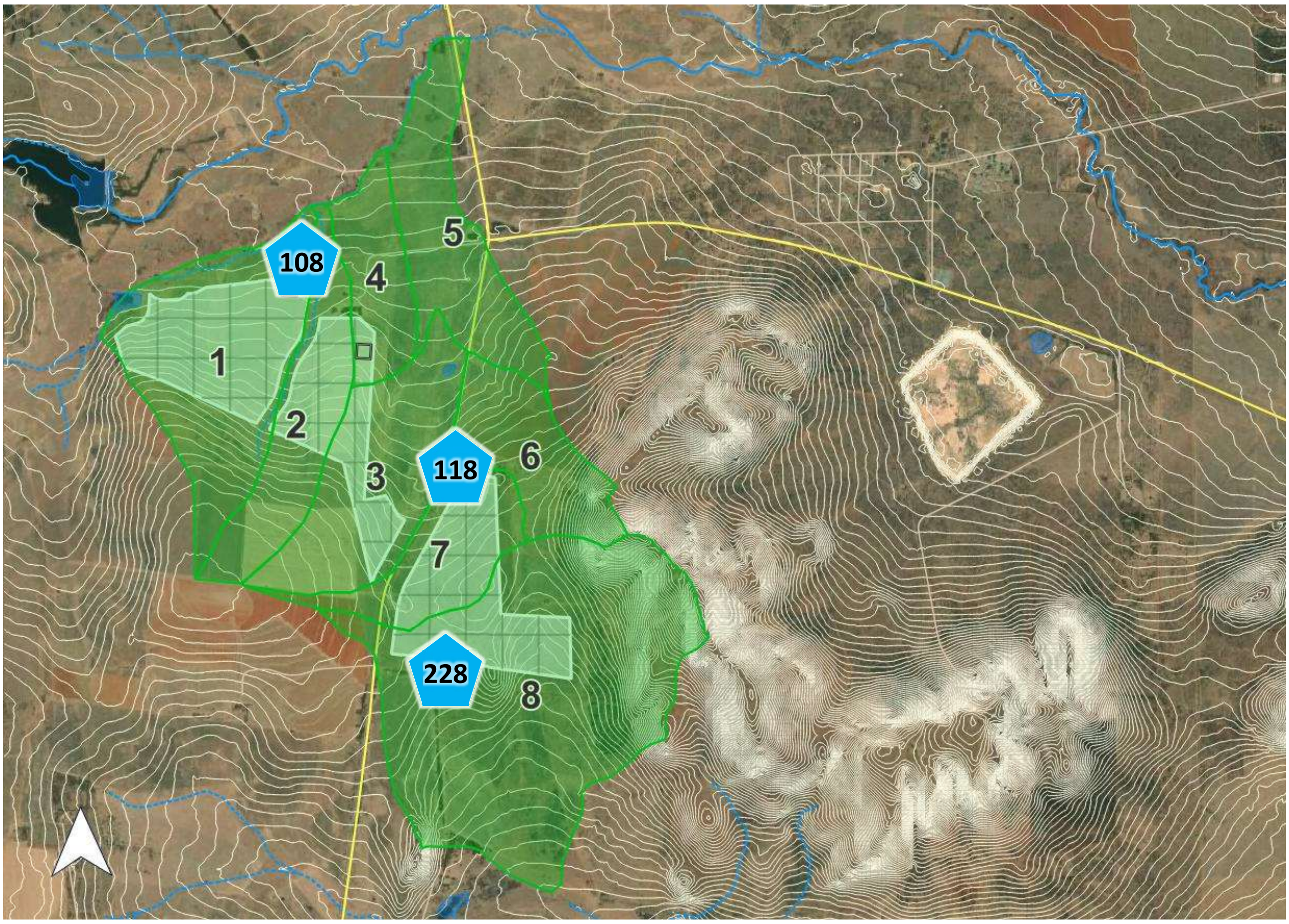


Figure 3-2: Detention Pond Locations

This selected locations allow the ponds to intercepts the bulk of the runoff from the main catchment and sub-catchment 8, given the natural drainage paths and other topographical constraints of the proposed development, while remaining close to the proposed development.

Alternative locations could be viable, but these would depend on factors such as property ownership, required link infrastructure and environmental considerations.

The required storage capacity and associated water depth for each detention pond is listed in **Table 3-5**, below and illustrated graphically on **Figure 3-3: Detention storage requirement per design storm duration.**

The detention ponds would function as “dry” facilities in that they would intercept runoff, detain it for a short duration and release it downstream at a controlled rate. They will not impede existing watercourses and will not “store” water. Consequently, they will not require a WUL.

Detention Pond	Max storage volume (x 10^3 m^3)	Max depth (m)	Critical storm duration (hours)
108	2.80	0.86	4
118	5.23	1.10	8
228	0.78	0.55	2

Table 3-5: Detention pond storage capacity for 50-year design storm

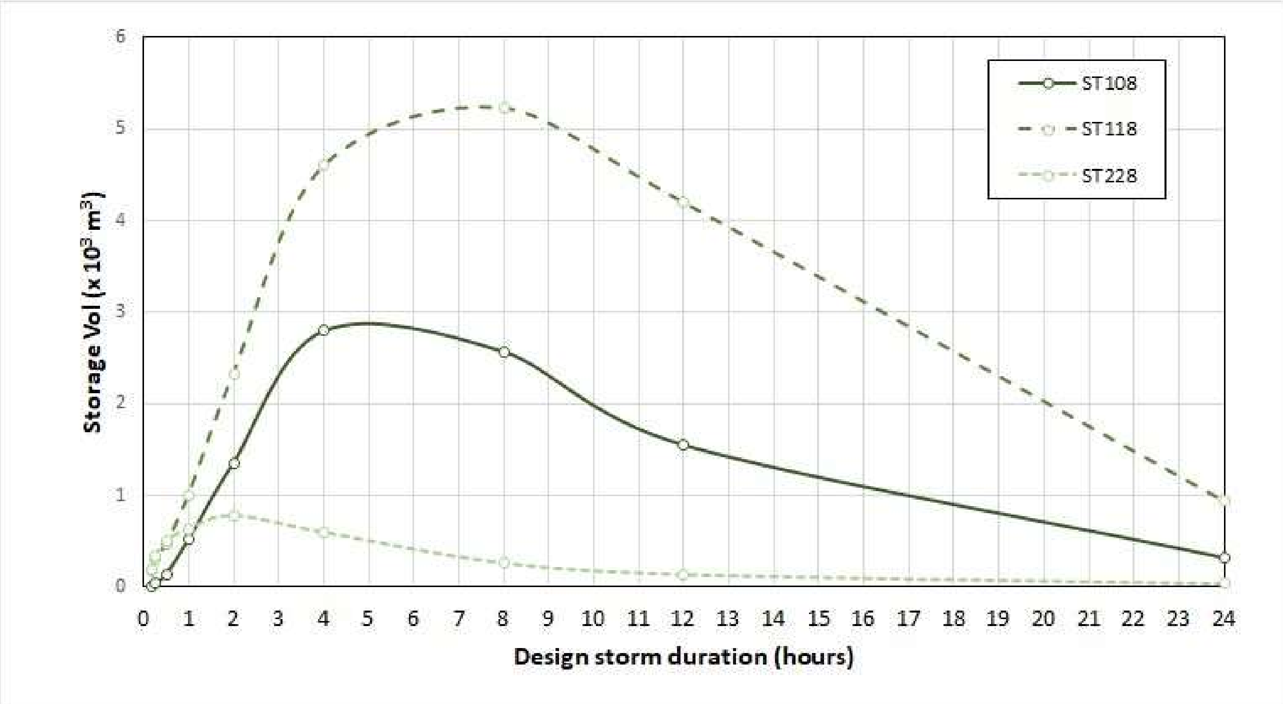


Figure 3-3: Detention storage requirement per design storm duration

The maximum water depth in Pond 118 could be a potential hazard to staff and would need to be addressed by means of an appropriate risk-management intervention – e.g. signage, area demarcation etc.

An example of a typical detention pond is indicated below on **Figure 3-4: Detention Pond Example**

Figure 3-4: Detention Pond Example

4 PROPOSED INFRASTRUCTURE

4.1 Proposed Infrastructure

It is anticipated that the Solar PV facility will contain the following infrastructure:

- On-site switching station / substation;
- Photovoltaic (PV) solar panels;
- Mounting structures to support the PV panels;
- On-site inverters;
- Transformer and internal electrical reticulation (underground cabling);
- Auxiliary buildings (such as gate houses and security, control centre, office, warehouse, canteen and visitors centre);
- Temporary laydown areas;
- Internal and perimeter access roads and fencing;
- Rainwater tanks; and
- Battery Energy Storage System.

Access will be gained either via the existing secondary road or from farm access roads linking to the secondary road. No improvements to the secondary road's existing surfaced standard are anticipated, but suitable provisions for the management of stormwater may be required. This will require a detailed survey and assessment of the condition and capacity of existing culvert crossings. The gravel farm access roads will require upgrading to an appropriate standard if these are used for access. This includes the provision of stormwater management interventions.

4.2 Pre-Construction Conditional Assessment

A conditional assessment of the site and access roads must be carried out prior to the commencement of construction. The areas to be used for the site camp, stockpiles and other temporary works must similarly be assessed.

The existing state of the downstream properties and infrastructure, as well as areas earmarked for temporary works, must be photographed and compiled into a baseline record.

4.3 Proposed Stormwater Mitigation Measures

To avoid creating downstream issues, it is essential that any disturbance of the areas earmarked for development must be minimised. In this regard, vegetation must be preserved; overland runoff must be permitted to continue unimpeded as far as possible; and concentration of flow must be avoided.

4.3.1 Internal and Perimeter Access Roads

Gravel access roads should be constructed at-grade to allow continuity of flow from upstream to downstream. Side drains will interrupt and concentrate the natural flow paths and should be avoided where possible. Where the roads are intersected by preferential drainage paths, stabilisation by means of stone protection on either side will mitigate against scour.

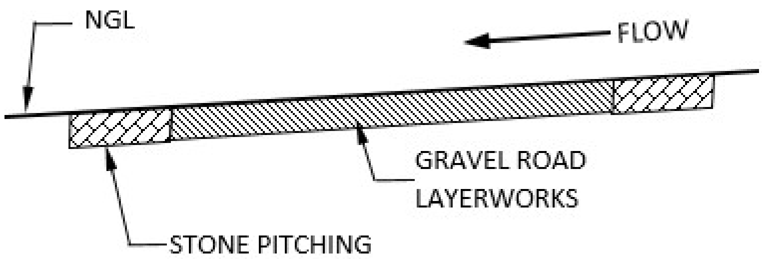


Figure 4-1: Access Road Cross-Section

4.3.2 Structures

Structures (e.g. substations, buildings etc.) will need to be protected by means of channels to divert runoff around them. However, the runoff must be returned to its original flow path as rapidly as possible, with suitable erosion protection downstream of the structure to reduce the velocity. Gabions or stone pitching should be used to encourage infiltration.

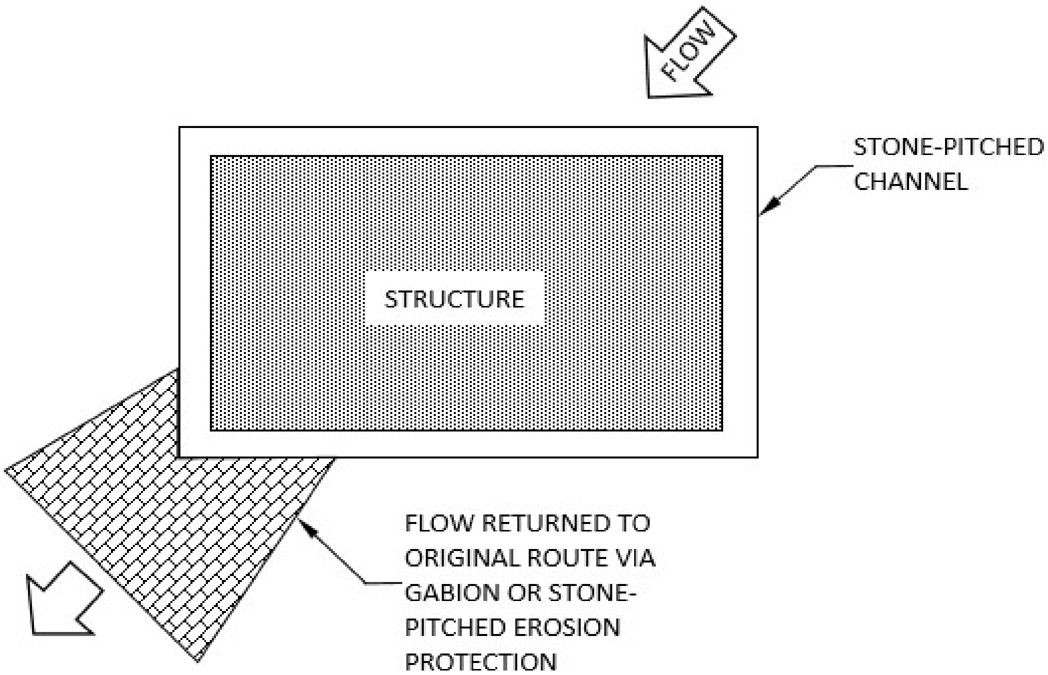


Figure 4-2: Plan View on Typical Structure

4.3.3 PV Panels

The supports to the PV panels should be designed to limit their impact on natural drainage patterns.

If the panels are constructed close to ground level, the runoff from individual panels will not increase the risk of erosion, irrespective of the panel orientation.

4.4 Management of Stormwater Impacts During Construction

4.4.1 Open trenches

Open trenches will be kept to a minimum and will be filled in progressively as construction proceeds. Excavated material to be used as backfill will be placed close to the trench on the upstream side to avoid loose material from washing away.

4.4.2 Stockpiles

Material stockpiles must be located away from any identified preferential drainage paths. Gravel, sand and stone stockpiles must be covered or kept damp to minimise dust. Temporary silt curtains or straw bales must be located immediately downstream of stockpiles to intercept grit wash-off.

4.4.3 Construction traffic

The crossing of any preferential drainage paths by construction traffic must be limited to a set number of strategic crossing points. Use of the final access road reserves during construction would address this issue. The crossings must be protected with stone pitching and the downstream drainage paths must be protected with appropriately-placed temporary silt curtains or straw bales.

Refuelling and maintenance of construction vehicles must be carried out in a controlled manner on impermeable surfaces to avoid hydrocarbon contamination of the soil.

4.4.4 Rehabilitation

Periodic monitoring during construction will be necessary to ensure that if damage does occur, it is addressed immediately and not be permitted to escalate.

Areas not occupied by permanent infrastructure (e.g. roads, parking areas etc.) must be rehabilitated to their original condition after construction is complete.

Any downstream damage directly attributable to construction activities must be repaired and the areas returned to their original condition.

4.5 Operation Phase Management of Stormwater Impacts

4.5.1 Detention

On-site treatment of stormwater will be by means of both formal and informal infiltration. The construction of formal structures such detention ponds and swales linking to them will encourage infiltration.

4.5.2 Waste Water Management

The washing of the solar panels will take place at set intervals using clean water to remove wind-blown dust and accumulated residue. As long as no detergents are used, there is consequently no risk of groundwater pollution, as the material that will collect on the panels currently settles directly on the ground surface and/or vegetation in the area.

The volume of water required for cleaning panels is approximately 3 litres/m^2 and the process will be carried out over a period of several weeks. The maximum flow will therefore be limited by the number of simultaneous cleaning operations taking place in close proximity to one another. The flow can therefore be considered negligible when compared to the runoff from design storm events. In this regard, the cleaning of panels is also not likely to take place during rainfall events of any significance.

The low flow rates mean that there is no erosion risk from the cleaning operation. Furthermore, owing to the infiltration potential of the soil, the cleaning water will be absorbed directly into the soil and no additional collection or treatment will be required.

4.5.3 Monitoring

Runoff from the site will be largely unchanged. Periodic monitoring of drainage paths and access roads downstream of the site must be done against the baseline assessment during the construction maintenance period to check for evidence of scour and / or siltation. Any damage directly attributable to operational activities will need to be repaired and the drainage paths returned to their original condition. Appropriate mitigation measures will need to be put in place to prevent recurrence of damage.

The erosion management strategy can be summarised in the following flow-chart:

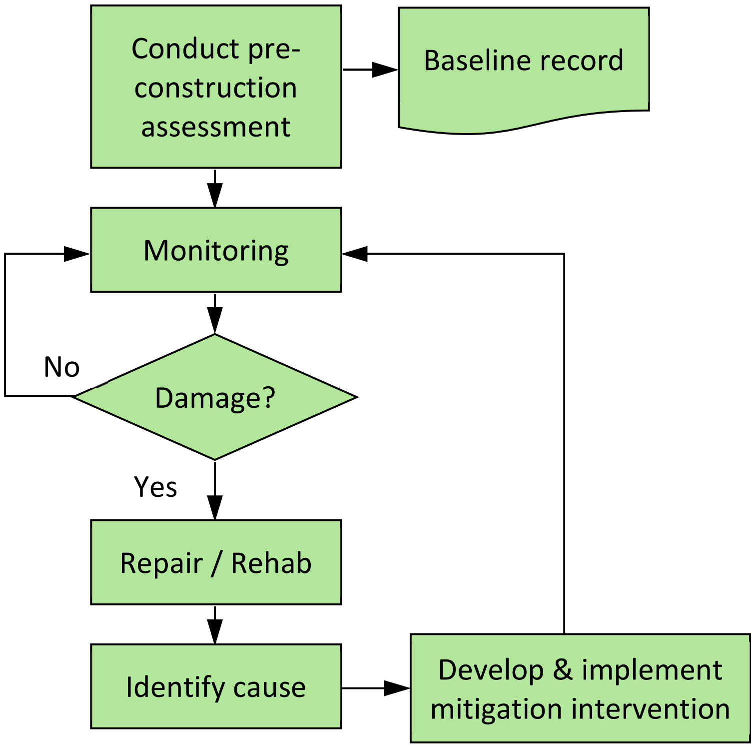


Figure 4-3: Erosion Management flow chart

5 CONCLUSIONS AND RECOMMENDATIONS

It may be concluded that:

- As long as the proposed new infrastructure is designed to maintain existing drainage patterns, the requirement for formal stormwater interventions will be limited in scope to detention storage, open channels and minor conduits;
- A pre-construction assessment will be necessary to ensure that construction and operational stormwater impacts are managed;
- For most storm events, overland flow via existing drainage paths will be the primary form of conveyance; and
- Detention storage will be required to limit post-development runoff to pre-development levels. The specific requirements are covered in detail in Sub-section 3.5.

It is recommended that:

- The safety aspects of proposed detention ponds be allowed for in the development planning;
- The interventions described in Sub-sections 4.2 to 4.5 be implemented; and
- The interventions described in Sub-section 4.4 be incorporated into the construction specification.

6 REFERENCES

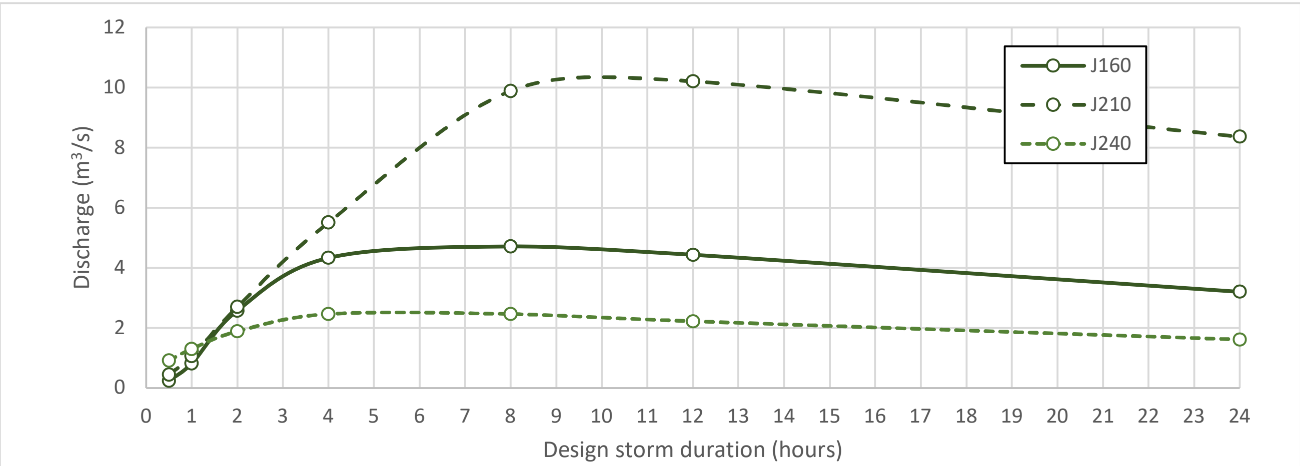
1. Smithers, JC & Schulze, RE. 2012. *Rainfall Statistics for Design Flood Estimation in South Africa*. WRC Project K5/1060.
2. Bailey, AK & Pitman, WV. 2015. *Water Resources of South Africa, 2012 Study (WR2012)*. WRC Report No K5/2143/1.
3. Schmidt, EJ & Schulze, RE. 1987. *Flood Volume and Peak Discharge from Small Catchments in Southern Africa, based on the SCS Technique*. WRC Report No TT 31/87.

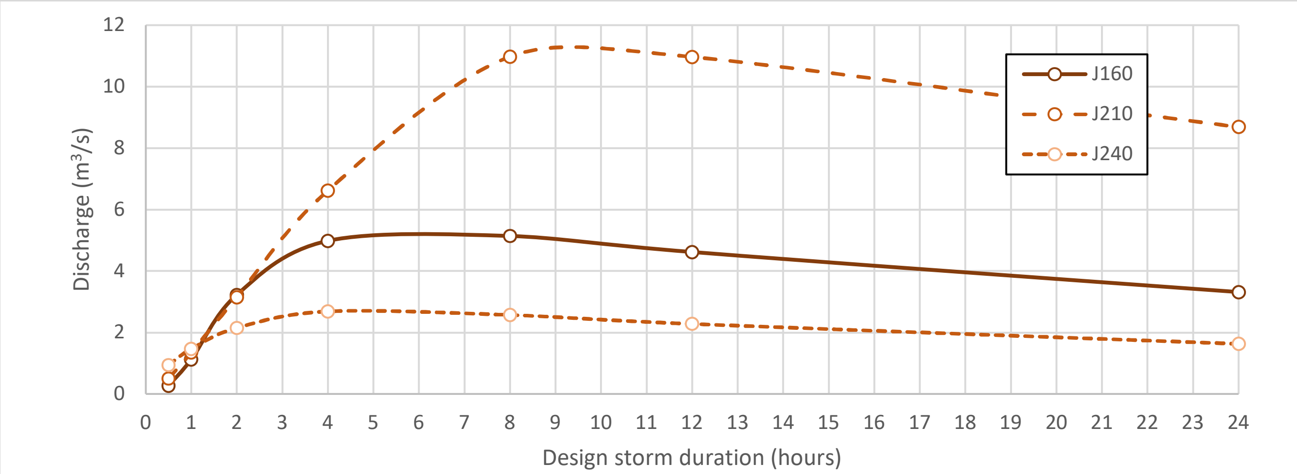
Annexure A: Modelling Results

Roan PV SWMP
SWMM summary results (Runoff)
Rev 01 08/02/2022

Peak runoff

Sub-Catchment	Discharge point	Peak runoff (m^3/s) for Return Period T = 5 years								
		Design storm duration (hours)								
		0.17	0.25	0.5	1	2	4	8	12	24
Pre-development										
S01	J110	0.21	0.30	0.44	0.63	0.91	1.19	1.20	1.08	0.78
S02	J110	0.11	0.15	0.23	0.33	0.47	0.62	0.63	0.57	0.42
S03	J140	0.16	0.23	0.33	0.48	0.69	0.90	0.90	0.81	0.59
S04	J150	0.09	0.13	0.19	0.27	0.38	0.47	0.44	0.38	0.27
S05	J160	0.09	0.13	0.20	0.29	0.42	0.59	0.64	0.60	0.46
S06	J130	0.18	0.25	0.36	0.50	0.67	0.77	0.66	0.55	0.37
S07	J120	0.11	0.15	0.23	0.32	0.46	0.58	0.56	0.49	0.35
S08	J240	0.43	0.62	0.91	1.30	1.88	2.46	2.46	2.22	1.61
S10	J170	0.14	0.20	0.30	0.44	0.66	0.94	1.08	1.06	0.86
S11	J180	0.20	0.29	0.43	0.63	0.94	1.35	1.55	1.52	1.24
S12	J200	0.26	0.37	0.55	0.78	1.13	1.50	1.51	1.37	1.00
S13	J190	0.45	0.63	0.92	1.29	1.76	2.08	1.84	1.55	1.06
S14	J210	0.17	0.24	0.36	0.54	0.82	1.21	1.45	1.48	1.28
Post-development										
S01	J110	0.25	0.37	0.58	0.85	1.19	1.47	1.37	1.19	0.83
S02	J110	0.12	0.18	0.28	0.41	0.58	0.73	0.70	0.61	0.43
S03	J140	0.17	0.25	0.38	0.54	0.77	0.99	0.96	0.85	0.60
S03	J150	0.11	0.16	0.24	0.35	0.47	0.56	0.49	0.41	0.28
S05	J160	0.09	0.13	0.20	0.29	0.42	0.59	0.64	0.60	0.46
S06	J130	0.18	0.25	0.36	0.50	0.67	0.77	0.66	0.55	0.37
S07	J120	0.13	0.19	0.29	0.42	0.58	0.69	0.63	0.53	0.37
S08	J240	0.46	0.66	0.97	1.40	2.00	2.58	2.54	2.27	1.63
S10	J170	0.15	0.22	0.34	0.51	0.76	1.07	1.18	1.14	0.89
S11	J180	0.22	0.32	0.50	0.75	1.12	1.57	1.73	1.66	1.30
S12	J200	0.28	0.41	0.61	0.88	1.26	1.62	1.60	1.43	1.02
S13	J190	0.52	0.74	1.08	1.51	2.01	2.28	1.94	1.62	1.09
S14	J210	0.18	0.26	0.40	0.61	0.92	1.34	1.57	1.58	1.33





Peak flow

Conduit	Discharge point	Peak flow (m^3/s) for Return Period T = 5 years								
		Design storm duration (hours)								
		0.17	0.25	0.5	1	2	4	8	12	24
Pre-development										
R_160-170	J160	0.10	0.14	0.24	0.81	2.57	4.33	4.71	4.43	3.20
CH210-220	J210	0.18	0.26	0.45	1.06	2.69	5.50	9.89	10.21	8.36
CH240-260	J240	0.43	0.62	0.91	1.30	1.88	2.46	2.46	2.22	1.61
Post-development										
R_160-170	J160	0.10	0.14	0.27	1.13	3.23	4.98	5.14	4.62	3.32
CH210-220	J210	0.19	0.28	0.50	1.35	3.15	6.62	10.98	10.97	8.69
CH240-260	J240	0.41	0.60	0.94	1.47	2.15	2.69	2.57	2.28	1.63

% Change in peak flow (Post - Pre) per storm event										
R_160-170	J160	1.05	0.71	15.61	38.50	25.46	15.16	9.22	4.24	3.53
CH210-220	J210	5.71	8.14	12.58	27.72	16.98	20.29	11.02	7.43	3.89
CH240-260	J240	-3.72	-3.23	3.74	13.23	14.41	9.27	4.59	2.79	1.43

Post-development with mitigation										
R_160-170	J160	0.01	0.14	0.27	1.10	3.13	4.85	5.08	4.59	3.31
CH210-220	J210	0.18	0.27	0.46	1.24	2.77	7.11	10.97	10.91	8.67
CH240-260	J240	0.41	0.58	0.89	1.41	2.10	2.67	2.57	2.28	1.63

Change in maximum peak flow

Discharge	Peak flow (m^3/s)		Change	
Point	Pre-dev	Post-dev	(m^3/s)	%
R_160-170	4.71	5.14	0.43	9.2
CH210-220	10.21	10.98	0.77	7.5
CH240-260	2.46	2.69	0.23	9.3

Change in maximum peak flow with mitigation

Discharge	Peak flow (m^3/s)		Change	
Point	Pre-dev	Post-dev	(m^3/s)	%
R_160-170	4.71	5.08	0.37	7.8
CH210-220	10.21	10.97	0.76	7.4
CH240-260	2.46	2.67	0.21	8.5

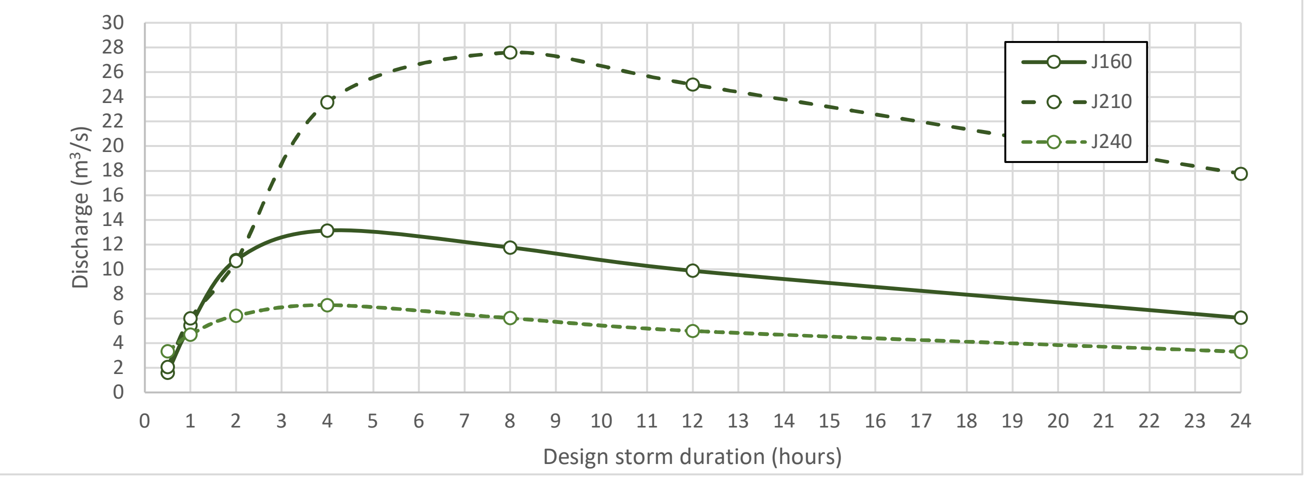
Roan PV SWMP

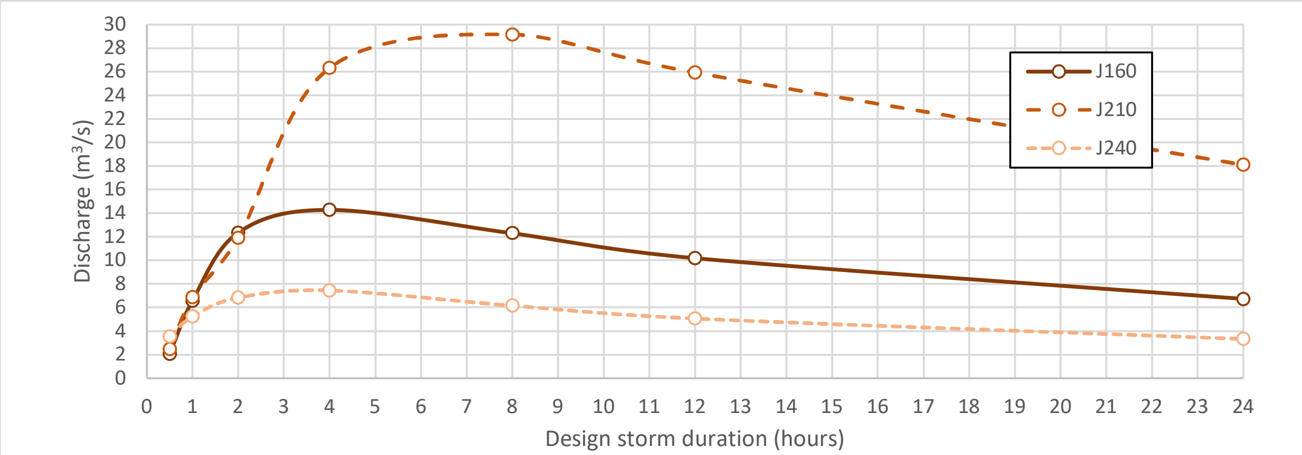
SWMM summary results (Runoff)

Rev 01 08/02/2022

Peak runoff

Sub-Catchment	Discharge point	Peak runoff (m^3/s) for Return Period T = 50 years								
		Design storm duration (hours)								
		0.17	0.25	0.5	1	2	4	8	12	24
Pre-development										
S01	J110	0.85	1.17	1.61	2.27	3.02	3.44	2.93	2.43	1.61
S02	J110	0.44	0.60	0.83	1.18	1.57	1.80	1.55	1.29	0.85
S03	J140	0.64	0.89	1.22	1.72	2.28	2.59	2.20	1.82	1.20
S04	J150	0.36	0.50	0.68	0.95	1.21	1.30	1.04	0.84	0.55
S05	J160	0.38	0.52	0.73	1.05	1.45	1.78	1.64	1.42	0.97
S06	J130	0.70	0.96	1.29	1.72	2.06	2.00	1.48	1.17	0.75
S07	J120	0.44	0.60	0.83	1.15	1.49	1.63	1.33	1.09	0.71
S08	J240	1.75	2.42	3.33	4.70	6.23	7.09	6.03	4.99	3.30
S10	J170	0.57	0.80	1.11	1.62	2.29	2.94	2.89	2.58	1.83
S11	J180	0.82	1.14	1.59	2.32	3.28	4.23	4.17	3.73	2.65
S12	J200	1.05	1.45	2.00	2.83	3.78	4.34	3.72	3.10	2.06
S13	J190	1.80	2.46	3.32	4.49	5.52	5.53	4.19	3.33	2.14
S14	J210	0.70	0.97	1.36	2.01	2.89	3.89	4.07	3.77	2.80
Post-development										
S01	J110	0.99	1.41	2.00	2.82	3.63	3.91	3.15	2.55	1.66
S02	J110	0.49	0.70	0.98	1.38	1.80	1.98	1.63	1.34	0.87
S03	J140	0.69	0.97	1.34	1.89	2.47	2.74	2.27	1.86	1.22
S03	J150	0.43	0.60	0.83	1.14	1.41	1.43	1.10	0.87	0.56
S05	J160	0.38	0.52	0.73	1.05	1.45	1.78	1.64	1.42	0.97
S06	J130	0.70	0.96	1.29	1.72	2.06	2.00	1.48	1.17	0.75
S07	J120	0.51	0.72	1.00	1.39	1.75	1.81	1.41	1.13	0.73
S08	J240	1.83	2.54	3.51	4.94	6.50	7.30	6.13	5.05	3.33
S10	J170	0.61	0.87	1.24	1.82	2.55	3.20	3.05	2.68	1.86
S11	J180	0.89	1.26	1.81	2.66	3.72	4.67	4.45	3.90	2.71
S12	J200	1.13	1.57	2.18	3.08	4.05	4.55	3.83	3.16	2.08
S13	J190	2.00	2.76	3.74	5.01	6.01	5.81	4.31	3.40	2.17
S14	J210	0.73	1.03	1.48	2.20	3.15	4.16	4.26	3.90	2.85





Peak flow

Conduit	Discharge point	Peak flow (m^3/s) for Return Period T = 50 years								
		Design storm duration (hours)								
		0.17	0.25	0.5	1	2	4	8	12	24
Pre-development										
R_160-170	J160	0.40	0.59	1.60	5.44	10.75	13.15	11.76	9.88	6.06
CH210-220	J210	0.75	1.11	2.07	6.01	10.68	23.57	27.59	25.00	17.76
CH240-260	J240	1.75	2.42	3.33	4.70	6.23	7.09	6.03	4.99	3.30
Post-development										
R_160-170	J160	0.41	0.60	2.07	6.57	12.34	14.28	12.30	10.18	6.72
CH210-220	J210	0.79	1.19	2.46	6.86	11.90	26.32	29.15	25.92	18.10
CH240-260	J240	1.70	2.41	3.53	5.23	6.83	7.44	6.15	5.06	3.33

% Change in peak flow (Post - Pre) per storm event										
R_160-170	J160	0.74	1.69	28.74	20.81	14.75	8.59	4.61	3.06	10.89
CH210-220	J210	5.31	7.11	19.09	14.07	11.42	11.65	5.64	3.70	1.90
CH240-260	J240	-3.03	-0.54	5.95	11.23	9.70	4.95	2.06	1.38	0.85

Post-development with mitigation										
R_160-170	J160	0.41	0.61	1.90	5.86	10.95	12.80	11.53	9.81	6.69
CH210-220	J210	0.75	1.10	1.98	4.87	12.06	24.49	26.74	24.79	18.04
CH240-260	J240	1.64	2.27	3.20	4.87	6.58	7.37	6.14	5.06	3.33

Change in maximum peak flow

Discharge Point	Peak flow (m^3/s)		Change	
	Pre-dev	Post-dev	(m^3/s)	%
R_160-170	13.15	14.28	1.13	8.6
CH210-220	27.59	29.15	1.56	5.6
CH240-260	7.09	7.44	0.35	5.0

Change in maximum peak flow with mitigation

Discharge Point	Peak flow (m^3/s)		Change	
	Pre-dev	Post-dev	(m^3/s)	%
R_160-170	13.15	12.80	-0.35	-2.7
CH210-220	27.59	26.74	-0.85	-3.1
CH240-260	7.09	7.37	0.28	4.0
AGGREGATE	47.83	46.91	-0.92	-1.9

Roan PV SWMP

SWMM summary results (Storage)

Rev 03 10/02/2022

Detention pond storage requirements

Outlet Node	Pond Name	Depth and storage volume for Return Period T = 50 years								
		Design storm duration (hours)								
		0.17	0.25	0.5	1	2	4	8	12	24
Pond depth (m)										
J110	ST108	0.21	0.25	0.33	0.48	0.66	0.86	0.84	0.69	0.42
J120	ST118	0.24	0.33	0.52	0.59	0.80	1.05	1.10	1.01	0.58
J200	ST198	0.36	0.41	0.46	0.82	1.20	1.52	1.50	1.30	0.77
J230	ST228	0.36	0.42	0.47	0.51	0.55	0.50	0.39	0.34	0.26
Pond storage (x 10^3 m^3)										
J110	ST108	0.01	0.03	0.13	0.52	1.35	2.80	2.57	1.55	0.32
J120	ST118	0.18	0.30	0.47	1.01	2.33	4.60	5.23	4.20	0.94
J200	ST198	0.03	0.13	0.66	2.45	6.69	13.21	12.92	8.47	2.10
J230	ST228	0.19	0.34	0.50	0.64	0.78	0.60	0.26	0.14	0.04