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SINSKE CONSULT

FLOODLINE INVESTIGATION
VAN WYKS RIVER
PORTION 10 of FARM VAN WYKS RIVER No. 787
PAARL

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1 INTRODUCTION

1.1 Terms of reference

It was requested by Mr Andrew Hood of the Imperial Group (Pty) Ltd, Johannesburg, that a floodline study of the Van Wyks River, a tributary to the Berg River south of Paarl, be conducted and reported on.

The purpose of the study is the estimation of the 50- and 100-year floodline, required for the development of the land on Portion 10 of Farm Van Wyks River No. 787.

1.2 Study Area

Portion 10 of Farm Van Wyks River No. 787 is situated between the National Road N1 and the Regional Road R101. The Van Wyks River crosses the property from west to east. The downstream boundary is the culvert under the Regional Road R101 where the Van Wyks River exits the property; the upstream boundary is ca. 750 m upstream of the culvert.

Relevant maps:
1:10 000 orthophoto maps 3318 DD 4
The study area is shown on an aerial photograph Figure 1, APPENDIX
(Scale 1:10 000)

2 CATCHMENT INFORMATION

2.1 Catchment Area

The Van Wyks River is draining the southwestern flank of the Paarlberg (H = 580 m). The river is flowing in southerly direction towards the National Road N1. After crossing under the N1, the river flows between the N1 and the R101, and after crossing under the R101 further in easterly direction towards the Berg River.

The catchment is situated in Drainage Region G1 and covers 10% sub-region G10C.

Catchment area upstream of R101 culvert: $A = 30{,}8\ km^2$
Length of longest water course $L = 8{,}73\ km$

The catchment is shown (scale 1:50 000), Figure 2, APPENDIX
Source: 1:50 000 Maps 3318 DB and 3318 DD

2.2 Topography

The topographic characteristics of the Van Wyks River upstream of the R101 culvert are calculated and summarised, Table 1.

TABLE 1 Catchment characteristics

Upstream of Catchment area	A (km^2)	R101 culvert 31,0	
Highest point in catchment	RL (m)	580	
Height at catchment outlet	RL (m)	125	
Max. height difference	H_{max}(m)	455	
Longest watercourse	L (km)	8,80	
Distance to centre of catchment	L_C (km)	3,20	
		(1)	(2)
Average height difference	H_{avg} (m)	131	163
Average slope	J (%)	1,49	1,86
Orohydrographic factor	$K= L/\sqrt{J}$	7,20	8,86
Lag time (3)	t_L (min)	89	83
Time of concentration:			
Dyck/Peschke ($t_C = 2t_L$)	t_C (h)	2,97	2,77
Dyck/Peschke ($t_C = 1{,}67t_L$)	t_C (h)	2,47	2,31
Bransby Williams	t_C (h)	2,47	2,36
Kirpich (SCS) formula	t_C (h)	1,78	1,64
Kerby	t_C (h)	2,58	2,45
Kirpich/Kerby	t_C (h)	2,35	

Note:
(1) Average height difference and average slope calculated with equal area method
(2) Average height difference and average slope calculated with 1085 method
(3) Lag time calculated with Ramser/Kirpich/Rowe, Ven Te Chow or Thiele/Euler formulas, depending on K (Sifalda, 1996).

2.3 Land use

Based on slope and distribution of land use, the following weighted averages for the runoff coefficient C and the curve number CN (A.M. Condition II) have been computed:
Source: 1:50 000 Map 3318 DD

C = 0,35 CN = 72,5 (Sinske, 1999)
C = 0,33 (Dept. of Water Affairs Method)

2.4 Rainfall intensity

Centre of catchment: LAT 33° 47' LONG 18° 55'

(a) Point rainfall

Return period		T = 25 yr
Duration	t = 1 h	39 mm
Source:	t =24 h	79 mm

Isohyetal maps (Sinske, 1982, 1993, 1999)

(b) Point rainfall

		T = 20 yr	T = 50 yr	T = 100 yr
Return period		T = 20 yr	T = 50 yr	T = 100 yr
Duration	t = 1 h	30 mm	36 mm	41 mm
	t = 2 h	41 mm	49 mm	56 mm
	t = 4 h	53 mm	64 mm	72 mm
	t = 6 h	62 mm	74 mm	84 mm
	t = 8 h	69 mm	82 mm	93 mm
	t =12 h	80 mm	95 mm	108 mm
Source:	t =24 h	103 mm	123 mm	140 mm

Software (Smithers and Schulze. 2004)

(c) Point rainfall Calculated with Alexander's formula
2-year return period daily rainfall (Adamson, 1981): M = 44 mm
Average number of days per year on which thunder was heard: R = 5
Source: Alexander, 2001

3 FLOOD DETERMINATION

3.1 Empirical Methods

3.1.1 Midgley/Pitman Method

The empirical method (Midgley and Pitman, 1969, 1971) is recommended for catchment areas larger than 100 km^2 (with caution for areas larger than 10 km^2).

With mean annual precipitation MAP = 690 mm and coefficients K for Veld type 2 (winter rainfall region), the following flood peaks have been calculated:

Q_{50} = 93 m^3/s Q_{100} = 109 m^3/s

3.1.2 Kovaćs Method

The empirical method (Kovaćs, 1988) is recommended for catchment areas larger than 100 km^2 (with caution for areas larger than 10 km^2). The method is based on the largest recorded flood peaks in a certain flood region. By plotting these flood peaks against the corresponding catchment areas, regional K_e -values are determined to calculate the so-called Regional Maximum Flood (RMF) for the desired catchment area. By means of given ratios, the RMF can be scaled down to the 50-year or 100-year flood.

The catchment of the Van Wyks River is situated in Flood Region $5_{G,H}$ (K_e = 5), bordering on Flood Region 4 (K_e = 4). As the characteristics of the catchment (topography, land use and rainfall intensity) are comparable to those of Flood Region 4, it is recommended to apply K_e = 4 for the flood peak calculation.
The following flood peaks have been calculated:

Q_{50} = 96 m^3/s Q_{100} = 126 m^3/s

3.2 Statistical Method

There are no river gauging stations in the Van Wyks River or in a neighbouring catchment of similar size or characteristics. Therefore a flood frequency analysis (or regional flood frequency analysis) could not be done.

3.3 Deterministic Methods

3.3.1 Standard Design Flood (SDF)

The standard design flood (Alexander, 2002) is the rational method applied with regionalised parameters, depending on the situation and characteristics of the catchment. For this purpose South Africa is divided into 29 drainage basins with unique run-off coefficients and rainfall intensity. Time of concentration, t_C, is calculated by means of the Kirpich formula, using the 1085 method.

The Van Wyks River catchment is situated in drainage basin 17. The following flood peaks have been calculated:

Q_{50} = 68 m^3/s Q_{100} = 85 m^3/s

3.3.2 Rational Method

The rational method is a site-specific method, requiring input on land use and rainfall intensity. The method is generally applied to catchment area smaller than 15 km². The method used hereunder has the following features incorporated (Sinske, 1991, 1999):

- Usually, a rainfall duration $t = t_C$ is used, which results in the maximum flood peak, however, $t \neq t_C$ can be used for hydrograph analysis..
- Time of concentration t_C is calculated using the equal area method. The Kirpich formula usually yields too small values and should be extended with the Kerby formula (Maniak, 1992). Dyck & Peschke (1983) and Sifalda (1996) suggest $t_C = 2 \cdot t_L$ for small rural (natural) catchments and $t_C = 1{,}67 \cdot t_L$ for urban (bare ground) catchments.
- The rainfall intensity during the rainfall event is taken as constant.
- The runoff coefficient C is taken as constant during the rainfall duration. The stated C-value (§ 2.3) is adopted for a 100-year flood. For shorter return periods C is reduced in order to take into account the influence of initial saturation.
- For rainfall durations $t < t_C$ an area correction factor is applied, as all parts of the catchment cannot contribute simultaneously to runoff.

In order to apply the method to catchments larger than 15 km^2 (say up to 100 km^2), the following additional features are incorporated:

- An area reduction factor is applied, depending on catchment area and rainfall duration.
- With increasing catchment area the time of concentration is increasing, and the rainfall intensity cannot be taken as constant any longer. A correction factor is applied that decreases the peak and lengthens the base of the hydrograph accordingly.

Flood peaks are calculated, using point rainfall input according to § 2.4 (a), (b) and (c):

(a) $Q_{50} = 47$ m^3/s $Q_{100} = 61$ m^3/s
(b) $Q_{50} = 41$ m^3/s $Q_{100} = 52$ m^3/s
(c) $Q_{50} = 48$ m^3/s $Q_{100} = 62$ m^3/s

3.3.3 Peak Factor Method (PFM)

The peak factor method (Sinske, 1988,1991,1999) is a modified form of the rational method. For catchment areas $A > 15$ km^2 and/or time of concentration $t_C > 1$h, the method is preferred over the rational method, due to the following modifications:

- Rainfall duration $t = t_C$ as well as rainfall durations $t \neq t_C$ are investigated.
- Time of concentration calculated as above, but for large catchments ($K > 35$) the Kirpich formula is used.
- The rainfall intensity during the rainfall event is not taken as constant, but a non-uniform rainfall distribution is introduced by means of a peak factor, which depends on the time of concentration t_C and the rainfall duration t.
- The runoff coefficient is not taken as constant, but the decrease of infiltration loss with rainfall duration due to saturation is taken into account by means of the curve number (CN) concept.
- Curve numbers can be adjusted for antecedent moisture condition. For the usual return periods (T = 20 to 200 year) the state CN-value (§ 2.3) for condition II is adopted.
- For catchment areas $A > 15$ km² an area reduction factor is applied.
- For rainfall durations $t < t_C$ an area correction factor is applied.

Flood peaks are calculated, using point rainfall input according to § 2.4 (a), (b) and (c):

(a) $Q_{50} = 48$ m^3/s $Q_{100} = 68$ m^3/s
(b) $Q_{50} = 48$ m^3/s $Q_{100} = 63$ m^3/s
(c) $Q_{50} = 50$ m^3/s $Q_{100} = 69$ m^3/s

3.4 Discussion of Results and Design Flood Peaks

Discussion:

(1) Empirical method: The results of the empirical methods are too high. There are ca. 40 farm dams in the catchment with a total surface area of ca. 0,6 km^2, causing a lag, which will be reflected in the concentration time. The empirical methods do not take concentration time into account and will not considered for further evaluation.

(2) SDF, rational method and PFM: The results of these three methods do not differ very much. The SDF yields the highest peaks, because the concentration time cannot be adjusted for lag. In case of the other two methods, a t_C-value near the upper limit has been selected from the spectrum of possible concentration times to compensate for lag (cf. Table 1).

Recommendation:

Three of the farm dams (total area 0,3 km^2) are situated in the main stream of the Van Wyks River, causing an attenuation of the flood peaks by 6 to 7% (SANCOLD, 1991). Taking the attenuation effect into account, the following design flood peaks are recommended for the hydraulic calculation:

Design Flood Peaks: Q_{50} = 45 m^3/s Q_{100} = 57m^3/s

4 FLOOD PROFILE COMPUTATION

4.1 River Cross-sections

Herman Gelderblom, Biff Lewis Geomatics, Professional Land Surveyors, Cape Town, provided nine river cross-sections:

Section [1]: Directly downstream of the R101 culvert
Section [2]: Directly upstream of the R101 culvert
Section [3]: Upstream of R101 culvert, east of Van Wyks River
Section [4] to [9]: Upstream of R101 culvert, along Van Wyks River in upstream direction

The alignment of the cross-sections is shown on the contour plan.
(Scale 1:4 000). Fig. 3, APPENDIX

4.2 Stream Channel Characteristics

4.2.1 Channel Morphology

After passing through the N1 culvert, the Van Wyks River is flowing between the N1 and the R101 in a natural meandering course with dense vegetation on the riverbanks. Towards the upstream boundary of the study area (section [9] to [8), the river adopts a wetland type character. From section [8], however, the original course of the river must have been altered in the past, as can be seen on aerial photographs. The river now flows in a straight course along the R101 towards the R101 culvert, where it exits the study area with a sharp right bend.
The average slope of the riverbed within the study area is 0,25%, increasing in front of the culvert.

4.2.2 Channel Roughness

Channel roughness is determined depending upon whether the cross-section is treated as a compact or a compound section.

In case of a compact section a compound roughness is computed, based on roughness values k_S (mm) determined for channel bed and riverbanks, depending on the bed material (sand, cobbles, boulders) and low- to medium-size vegetation cover (grass, weeds, brush), to which up to 50 mm could be added for non-uniformity (BWK, 1999).

In case of a compound section, i.e. in the presence of floodplains, or where the river-banks are covered with large-size vegetation (tress, bush, reed), the stream channel is vertically divided into a main channel and a left and/or right channel (separated from the main channel by an interface). For the main channel the compound roughness is computed as above, but taking the interface roughness into account. For the left and/or right channel a friction factor λ is computed by taking into account the roughness k_S of the ground and the flow resistance parameters of large-size vegetation. Interface roughness and large-size vegetation roughness is determined based on techniques developed by Mertens (1989, 1994, 1997) and documented in DVWK (1991), BWK (1999).
The following basic roughness values are adopted:
Main channel: k_S = 5000 mm (reeds and sedges)
Floodplain: k_S = 300 mm to 350 mm for ground cover (grass, weeds, brush)
ω = 0,01 to 0,05m^2/m^3 for large vegetation (single trees and bush)

Photographs: Plates 4 to 12, APPENDIX

4.3 Hydraulic Capacity of R101Culvert

Herman Gelderblom, Biff Lewis Geomatics, measured the dimensions of the culvert.
Description:
The culvert under the R101 is a major culvert, consisting of two parallel rectangular barrels in a normal crossing. The Van Wyks River enters the culvert in a sharp right-hand bend.

	Span: 2 x 4,75 m	Width between abutments:	10,5 m
		Total opening area:	15,5 m^2
Levels:	Head wall 126,93 m	Length:	29,0 m
(upstream)	Deck 126,50 m (average)	126,40 m (lowest point on road)	
	Soffit 126,11 m		
	Bottom 124,48 m (average)		

Photographs: Plates 1 to 4, APPENDIX

Hydraulic Calculation
Hydraulic calculation (Franke, 1970/71; Hager and Wanoscheck, 1986; Hamill, 1999, Zanke, 2002) shows that for a 50- year flood the flow through the culvert will be in an instable state, between free surface and submerged flow. For a 100-year flood the flow will be submerged, with flow over the road.
In addition to the culvert, ca. 135 m in easterly direction, there is a d = 0,5 m pipe under the road embankment with an additional inlet between the two lanes.

The following water levels have been calculated, assuming the culvert opening is free of debris or other obstructions.

	Q_{50}	Q_{100}
Approach to culvert:	126,25 m	126,50 m
Over road:	----	126,45 m

4.4 Flow Profile Calculation

4.4.1 Method

The flow profile is calculated by non-uniform flow calculation (standard step method). The computation is done using the method developed by Naudascher (1956, 1987) and recommended (DVWK, 1991) for natural streams with floodplains or riverbanks with large vegetation. The Darcy-Weisbach equation is preferable to the GMS flow formula, especially when channel roughness consists of different roughness producers, such as sand roughness and vegetation (friction factors λ may be added, Manning's n not), Zanke, (2002). The Keulegan equation to compute λ is used instead of the Colebrook-White equation, as recommended by Mertens (1989, 1994).

4.4.2 Control Sections

The flow profile computation starts at a section where the relationship between flow depth and discharge can be determined, a so-called control. This can be natural or artificial cross-structure (e.g. a barrier, a weir), a contraction (caused by a culvert or a bridge), and finally a river reach with uniform flow, where the resistance due to channel roughness acts as control. From the control section the computation proceeds in upstream or downstream direction to the next cross-section.

4.5 Result

The flow profile for the 50- and 100-year flood, calculated for the surveyed river cross-sections is listed in Table 2.

TABLE 2 Result of flow profile calculation (Levels in [m] above mean sea level)

Cross-Section	Chainage (m)	Channel Bottom	Water Level 50-yr flood	Water Level 100-yr flood
[1]	0	124,40	125,90	126,00
[Culvert – Exit]	6	124,45		
[Culvert – Entr]	35	124,48		
[2]	39	124,50	126,30	126,50
[4]	62	124,65	126,30	126,40
[5]	249	125,10	126,90	127,00
[6]	426	125,50	127,10	127,20
[7]	612	125,90	127,50	127,65
[8]	699	126,10	127,75	127,90
[9]	805	(127,00)	128,50	128,65

Note:
Chainage in upstream direction:

Cross-section [1]: downstream of R101 culvert
Culvert (see § 4.3)
Cross-section [2]: upstream of R101 culvert
Cross-section [8]: downstream of boundary of property
Cross-section [9]: upstream of boundary of property (approx. water level, survey incomplete)

5 FLOODLINE DETERMINATION

5.1 Contour Plan

Herman Gelderblom, Biff Lewis Geomatics, Cape Town, provided the contour plan for the site (0,5 m contour interval).

5.2 Drafting of Floodline

The floodline on the Van Wyks River between successive river cross-sections has been drawn according to the contours of the site plan and features taken from the aerial photograph.

5.3 Representation of Floodline

The 50-year and 100-year floodline have been drawn on the provided contour plan on layers FLOODLINE50A and FLOODLINE100A within the boundaries of the property

Attached to this report is a printout of the site plan. Figure 3, APPENDIX
(Scale 1:4 000)
50-year floodline (blue solid line)
100-year floodline (blue dashed line)

The floodline is also shown on the aerial photograph. Figure 4, APPENDIX
(Scale 1:5 000)

5.4 Comment

At the upstream boundary of the property, between cross-sections [9] and [8], the river has a wetland-type character. Only an approximate floodline is shown here, due to incomplete survey.

Dr. B.H. SINSKE, Pr.Eng.

Stellenbosch,
September 2008

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APPENDIX

Figures

Photographs Courtesy of Mr H. Gelderblom and Dr A. Sinske (Aug. 2009)