PAPER 11

Hydrological Study and Hydrodynamic Modelling on Horlosiekloof, De Doorns, Western Cape

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ABSTRACT

An investigation on the flood hydrology and river hydraulics of the Horlosiekloof catchment area (De Doorns in the Western Cape) was carried out with the primary objective to determine firstly if developments in the catchment impacted on the flood peaks and secondly, the possible mitigation measures to decrease the flood risk. This was achieved by performing a flood hydrology analysis of the Horlosiekloof catchment and creating a two-dimensional (2D) model of the catchment area.

A flood analysis of the pre- and post-development of the lower catchment concluded that the agricultural developments had only a marginal impact on the expected flood peaks. However, human interaction resulted in altering the flow path, and presently most of the runoff is diverted into/concentrated in a well-defined stone pitched channel. The 2D modelling indicated that, while the combined capacity of the culverts may be adequate to deal with the 1:2-year flow, supercritical flow conditions exist upstream of the site.

Possible mitigation measures include the upgrading of the capacity of the existing culverts near the stone pitched channel, and that additional culvert may reduce the flood risk. The ground level upstream of all culverts (new and existing) must be lowered to force a hydraulic jump upstream of the site in order to create subcritical flow conditions. A flood attenuation dam was evaluated as an alternative to upgrading the capacities of the culverts.

INTRODUCTION

Flooding of the N1 road occurs near De Doorns in the Western Cape, which might cause risk to normal traffic, as well as storm damage to the N1. Consequently, an investigation on the flood hydrology and river hydraulics of the Horlosiekloof catchment area was carried out.

The primary objectives of the study were:

- Determine the expected flood peaks and hydrographs from the Horlosiekloof catchment area.
- Ascertain the impact that the agricultural development has on the flood regime.
- Perform 2D modelling of the current development of the study area to determine the extent of the flooding that can be expected by using the HecRAS software program.
- Determine possible mitigation measures to decrease the flood risk.

FLOOD HYDROLOGY

The main objective of the flood analysis was to determine the magnitude of the expected flood event at the N1 and to establish the impact of the agricultural developments upstream of the N1 on these flood peaks.

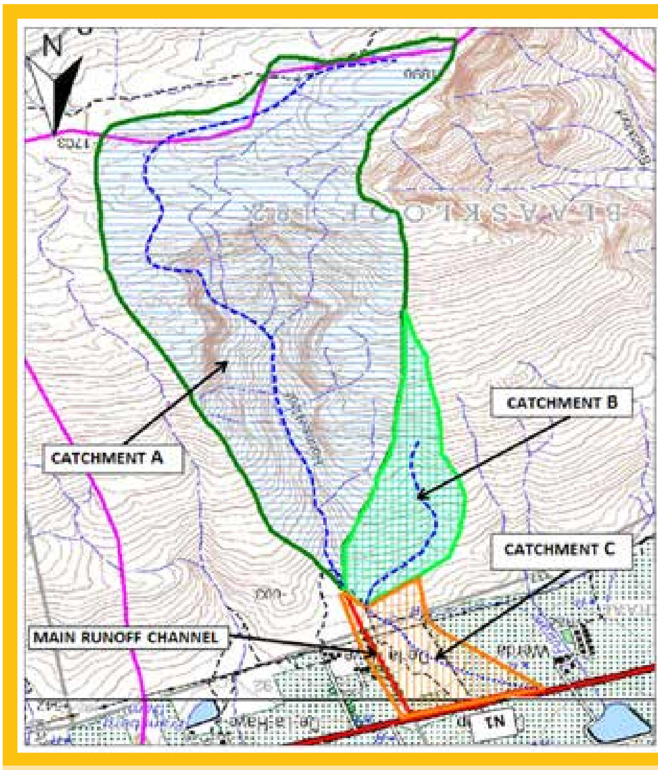

FIGURE 1: Contributing catchments requirements

Catchment characteristics

Various catchments contribute towards the runoff that needs to be dealt with at the N1 crossings. These catchments are shown in Figure 1.

Catchment A represents the main contributing catchment and reflects the catchment area that would contribute all the water that would enter the main runoff channel, leading towards the N1.

Catchment B represents a smaller catchment bordering the lower end (North-West) of catchment A. This catchment does not enter the main runoff channel, which is the main focus area. It does however influence the understanding of how the runoff from these catchment areas were influence over time and specifically the impact on the runoff after the construction of the N1.

Catchment C reflects the catchment area that was impacted significantly over time by agricultural activities. It also represents the lower delta-like catchment area, which provided the natural drainage route of all water from catchment A and B, prior to the redevelopment of the now well-defined runoff channel.

TABLE 1: Catchment characteristics				
Catchment	**Catchment Area (km²)**	**Longest Water-course (km)**	**Average Water Course Slope (m/m)**	**Time of Concentration (hours)**
A	4.685	5.965	0.256	0.44
B	0.477	1.315	0.281	0.13
C	0.569	1.294	0.083	0.21
Full Area (A+B+C)	5.731	7.423 (7.259)	0.231 (0.235)	0.55 (0.53)

The catchment information required by the various flood determination methods are shown in Table 1.

The catchment characteristic values for the full catchment (A+B+C) area shown in brackets in Table 1, represent pre-development values (before the N1 was constructed) and they do differ slightly from the present day situation due to the man-made influences on the runoff path of the stormwater from the catchment area.

Rainfall

Only rainfall data from two monthly rainfall stations were available i.e. De Doorns (NIWW) and Hexvalley PP. De Doorns (NIWW) had a 40 year record length with a Mean Annual Precipitation (MAP) of 311 mm, while Hexvalley had a 9 year record length with a MAP of 406 mm.

No short duration data was available for the catchment. Moreover, since the time of concentration (Tc) for the different catchments to be analysed is less than 1 hour, the design rainfall required for the flood calculations were based on the storm rainfall data as proposed by the software application developed by Smithers and Schulze (2000). Based on this software application, the short duration catchment rainfall, calculated using a Thiessens polygon approach, are shown in Table 2 for catchment A and the full catchment.

TABLE 2: Smithers storm rainfall in mm							
Catchment	**Storm duration**	**Recurrence interval of storm event**					
		2	**5**	**10**	**20**	**50**	**100**
A	15 m	16.8	21.8	25.1	28.2	32.3	35.3
	30 m	23.6	30.8	35.5	39.9	45.7	50.0
	45 m	29.0	37.8	43.6	49.1	56.2	61.5
	1 h	33.7	43.9	50.6	57.0	65.3	71.4
Full	15 m	17.0	22.1	25.5	28.6	32.8	35.8
	30 m	23.5	30.6	35.3	39.7	45.4	49.6
	45 m	28.6	37.2	42.9	48.3	55.2	60.3
	1 h	32.9	42.9	49.4	55.6	63.6	69.5

TABLE 3: Station H2H005: Flood calculation: Q (m³/s)						
Methodology	**Recurrence interval: T(a)**					
	1:2	**1:5**	**1:10**	**1:20**	**1:50**	**1:100**
Probabilistic approach	8	11	14	16	19	23
Rational: Smithers MAP	37	51	62	74	89	102
Rational: De Doorns MAP	27	38	46	55	66	76

Flood frequency analysis

Three methods were used in calculating the flood peaks of the different catchments under evaluation, i.e the Rational, SCS-SA and SDF methodology.

Rational method

This is the most used method in South Africa and the Horlosiekloof catchment size falls well within the application range of the Rational method. The method depends on the MAP to guide the selection of the runoff coefficients used in the method and was developed with a coaxial diagram, using MAP to determine storm rainfall (Van Dijk, Van Vuuren & Smithers 2015). The MAP at De Doorns and Hexvalley are 311 mm and 406 mm respectively. These values were used as indicative values for the catchment in the Rational method, to determine the runoff coefficients.

The critical MAP ranges for the selection of appropriate runoff coefficients within the Rational method is categorised in three groups i.e.: below 600 mm/a, between 600 and 900 mm/a and above 900 mm/a. The software application (Smithers and Schulze 2000) used to calculate the storm rainfall can also provide the MAP values. These values were however suspiciously high. To verify the selection of appropriate runoff coefficients using the catchment MAP, the closes available river flow station's annual maximum peak flow series (AMS) were analysed, using a probabilistic approach.

The closes available river flow station is the Rooi Elskloof flow station (H2H005), situated just north to the Horlosiekloof catchment. While it is recognised that this catchment might receive different storm rainfall than the Horlosiekloof catchment, the general catchment characteristics in terms of slope and vegetation were judged to be able to produce similar runoff coefficients as what can be expected from the Horlosiekloof catchment. The following procedure was followed to verify the MAP values:

- Do a probabilistic analysis of the AMS of river flow station H2H005 and determine the flood peaks for the 1:2 to 1:100 year recurrence interval (RI).
- Calculate the Rooi Elskloof (H2H005) catchment MAP and design rainfall using the software application.
- Apply the Rational method to calculate the 1:2 to 1:100 year RI flood peaks.
- Compare the results with the results obtained with the probabilistic approach.
- Change the MAP category and determine the impact of a different MAP category on the associated runoff coefficients.
- Based on this finding, determine the most likely MAP category to be used to provide realistic flood peak results for the H2H005 catchment and assume that the same MAP category can be used for the Horlosiekloof catchment.

The procedure above confirmed that, using the software application to calculate the catchment MAP, produce flood peaks that overestimate the flood peak values compared to the probabilistic approach. The selection of runoff coefficients associated with a catchment MAP values below 600 mm/a provided results, when using the Rational method, which was deemed more appropriate if compared with the results obtained from the analysis of the observed AMS, than using a MAP category of more than 600 mm/a as suggested by the software for the H2H005 catchment. Following this observation, it was assumed to be also applicable to the

Horlosiekloof catchment areas and runoff coefficients associated with a catchment MAP of 600 mm/a or less was used in all further analysis.

The results of the three different methods for the flood calculation for station H2H005 are summarised in Table 3.

Rational method: Flood peaks before and after development of N1

The N1 road was developed along its present alignment between 1942 and 1962 and the catchment area upstream of the N1 was at the time undeveloped and consists of natural vegetation.

It is therefore reasonable to assume that the storm water drainage along the N1 was designed, using runoff coefficients associated with an undeveloped catchment. The storm rainfall used for the design was in all probability also those values obtained from the old coaxial diagrams (Van Dijk, Van Vuuren & Smithers 2015). For a comparison between a pre- and post-N1 scenario, the catchment characteristics for the full catchment as highlighted in Table 1 (values in brackets) were used in conjunction with the storm rainfall from the coaxial diagrams to calculate the flood peaks for different return periods. A MAP of less than 600 mm/a was again assumed for the selection of the runoff coefficients.

For the post-N1 scenario, the delta discharge drainage area was considered as almost fully developed. The Rational method was again applied with the corresponding new runoff coefficients, using the same coaxial diagram-based storm rainfall and catchment characteristics from Table 1.

In evaluating the results presented in Table 4, it is clear that the development of the delta discharge drainage area did not have any impact on the flood peaks. This is due to a very small percentage of area developed, while the runoff path (longest water course) was artificially lengthened at the same time.

TABLE 4: Sational method: Pre & Post – N1 flood peaks (m^3/s)

Methodology	Recurrence interval: T(a)					
	1:2	1:5	1:10	1:20	1:50	1:100
Rational: Pre-N1	15	22	29	39	53	68
Rational: Post-N1	15	22	29	38	53	68

Rational method: Present day flood peaks

For the calculation of the present day flood peaks to be dealt with, given the problems experience at the N1, two factors need to be considered:

- Improved storm rainfall is available through the software application, and
- a newly defined catchment exists as a result of the well-defined man-made runoff channel, which only deals with the runoff from catchment A (see Figure 1). The water from catchment B does not enter this channel, but also need to pass underneath the N1.

The present-day flood peak calculations using the catchment characteristics for catchment A and the storm rainfall from the software application and runoff coefficients associated with a MAP less than 600 mm/a is presented in Table 5.

It is clear that there is only a marginally difference in the flood peaks if the full catchments are used as opposed to only using catchment A, which is the only catchment producing runoff that flows in the man-made runoff channel.

TABLE 5: Rational method: Present day floods (m^3/s)

Methodology	Recurrence interval: T(a)					
	1:2	1:5	1:10	1:20	1:50	1:100
Rational: Catchment A: present day	25	34	42	50	60	70
Rational: Full Catchment: present day	27	38	46	55	66	76

TABLE 6: Summary: Catchment A: Present day flood peaks (m^3/s)

Methodology	Recurrence interval: T(a)					
	1:2	1:5	1:10	1:20	1:50	1:100
Rational	25	34	42	50	60	70
SCS-SA	26	46	62	81	107	131
SDF	11	26	39	53	73	90

SCS-SA method

The SCS-SA methodology is specifically adapted to suit South African conditions and is suitable for small catchment taking different development zones within one catchment area into consideration. Given the simplicity of the specific catchment, it was necessary to identify only two different homogenous development zones, reflected as curve numbers (CN). One zone, representing 37% of catchment A's area, was considered to have a high runoff potential (CN = 84) which was associated with the steep mountain slopes, while the second area, representing 63% of catchment A's area, was considered to have a lower runoff potential (CN= 80).

The storm rainfall that is used with the SCS-SA method is based on a 1-day storm rainfall event which can be obtained from either the SCS-SA own dataset, the TR102 dataset or a user defined 1 day storm rainfall analysis. The first approach (using the SCS-SA dataset), which is similar to the software applications storm rainfall data, provided results way in excess of the results obtained with the Rational and SDF methods. It was therefore decided to use the 1-day storm rainfall as provided in TR102. This approach provided results comparable to the other two alternatives approaches used.

SDF method

The SDF methodology is the most recent developed flood determination methodology in South Africa, developed specifically to allow for less engineering judgement in the application process of the methodology. The method makes use of a pre-selected SDF basin (Basin 18 in this case) and calibration coefficients. Some serious shortfalls have been identified in recent research (Gericke and Du Plessis 2012) with the development of the runoff coefficients and a need for the re-calibration of some of the coefficients used in the method was highlighted. The method is however simple to apply and is always used as a benchmark value for road drainage designs.

Summary of flood peaks

With the focus on the problems experienced with the runoff in the well-defined runoff channel when it reaches the N1, the results of the different flood determination methods for only catchment A, are presented in Table 6.

The three approaches used to calculate the expected flood peaks to be dealt with in the main runoff channel provide reasonable similar results, with the Rational method providing reasonable results for the smaller return periods and slightly lower values for the higher return periods. It was therefore decided to use the results from the Rational method as representative of the Horlosiekloof catchment in all further analysis.

RIVER HYDRAULICS

Hydrodynamic Model Setup

A detailed 2D hydrodynamic modelling (HecRAS 5.0.3. (Brunner 2016)) simulation was conducted on the catchment.

A LIDAR Survey was carried out by aircraft to cover the area and 1 m interval contour lines were made available for the modelling. The topographical survey data on a grid of 2 x 2 m was read into the model to create the model bathymetry.

The hydraulic roughness was specified in the model to compensate for the effects of vegetation and a Manning n-value of 0.055 was used (Chadwick et al. 2013). The N1 road, crossing the study area, was also included with the culverts.

The upstream boundary of the model is located at the start of the gorge, while the downstream boundary was placed several hundred meters (on average) downstream of the study area so that the downstream boundary does not affect the main model domain and flow patterns. At the upstream inflow boundary, a typical flow hydrograph shape was specified for the various recurrence interval floods. At the downstream boundary, normal flow depth was specified. The model simulates the hydrodynamics in a fully hydrodynamic mode, with time steps of the unsteady simulation typically less than 1 second.

Culvert Discharge Capacities

The total combined discharge capacity of the existing culverts is 30.7 m^3/s, which is only marginally greater than the 2-year peak flood of 27 m^3/s (for full catchment). The 2-year flood peak of 25 m^3/s associated with the flow from catchment A, following the well-defined man-made flow path, is slightly less than the calculated culvert capacities.

The modelling indicated that, while the combined capacity of the culverts may be adequate to deal with the 1:2-year flow, supercritical flow conditions exist upstream of the N1 due to the steep flow path slope. Supercritical oncoming flow must be allowed to pass through the culverts virtually undisturbed (Rooseboom & Van Vuuren 2015), otherwise considerable damming would occur upstream of the N1, that would flood the road surface.

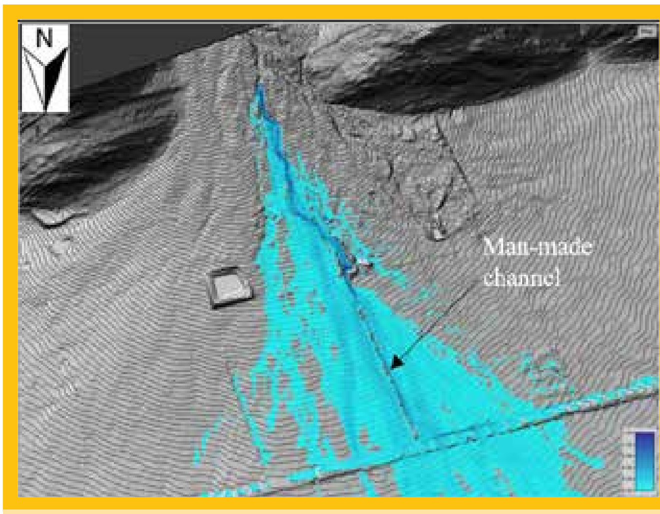

FIGURE 2: Simulated study area flow depths (m) at peak of the 20-year flood (50 m^3/s)

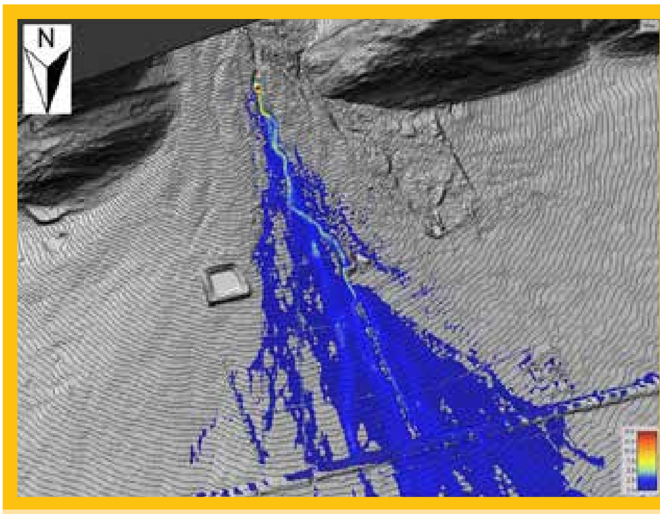

FIGURE 3: Simulated study area flow velocities (m/s) at peak of the 20-year flood (50 m^3/s)

Simulation results on current development

The simulation results for the 20-year flood (current scenario) are illustrated in Figure 2 and Figure 3. The simulated flow depths are shown in Figure 2. The maximum flow depth just upstream of the N1 is 1.2 m and is caused by the road as an obstruction across the floodplain. Spillage occurs over the road to a maximum flow depth of 0.4 m. The damming causes flooding upstream of the road over a distance for about 1.8 km along the N1, and spillage over the N1 over a distance of approximately 700 m.

The simulation results indicate that most of the storm runoff flows down the catchment along and near the man-made channel as seen in Figure 2. The limited capacity of the waterway underneath the N1 at the point where the channel reaches the N1 still results in the overtopping of the N1.

The simulated flow velocities for the 20-year flood peak are shown in Figure 3. The flow velocities just upstream of the N1 road are below 2.6 m/s due to the damming. Higher velocities up to 2.8 m/s were simulated just downstream of the N1 road. However, higher flow velocities up to 21.6 m^3/s higher up in the catchment were simulated, due to the steep slope of the catchment. Supercritical flow conditions occur upstream of the N1 road.

Results for the 1:50 year flood simulation are similar to the 1:20 year flood simulation results.

The results from the hydrodynamic modelling indicated that the limited capacity of the culverts cause damming upstream of the road. The flow velocities in the catchment are high with supercritical flow conditions upstream of the road.

The best way to reduce the damming would be to construct a bridge(s) or culverts on the road in order to allow the supercritical flow to traverse the road undisturbed, but the road is low and it is doubtful whether sufficient freeboard is available. A flood attenuation dam could also be constructed to attenuate the flood to a river flow discharge capacity equal to the combined culvert capacity at the N1 road.

POSSIBLE MITIGATION MEASURES TO DECREASE FLOOD RISK

SANRAL proposed that for the future flood remedial measures for the Horlosiekloof catchment area, the 20-year (50 m^3/s) flood recurrence should be used in designs.

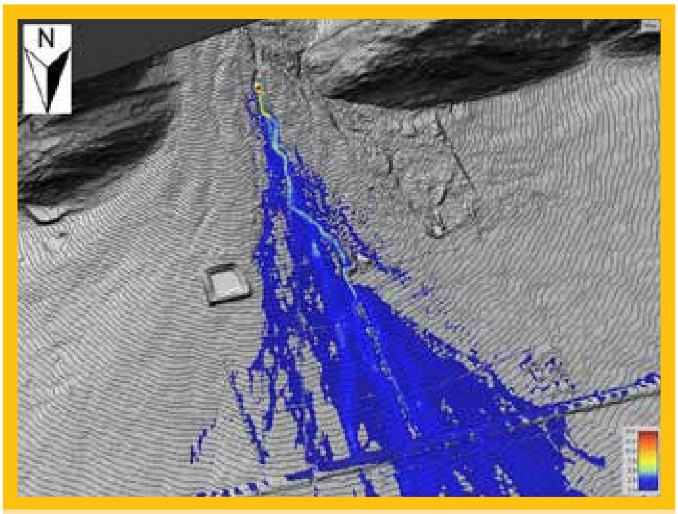

FIGURE 4: Proposed attenuation dam site

The following long-term flood mitigation measures were considered:

- Upgrade of N1 culvert discharge capacity.
- Flood attenuation dam.

Upgrade of N1 culvert discharge capacity

It is proposed that the discharge capacity of the existing culverts near the man-made channel be upgraded, and that additional culverts be constructed in certain areas to convey the 1:20 year flood (50 m^3/s).

All the culverts (existing, as well as additional culverts) have to be designed with intake structures and USBR type stilling basins at the upstream and downstream end respectively.

The natural ground levels upstream of all the culverts has to be lowered so that a hydraulic jump would form upstream of the N1, forcing the supercritical flow to return to subcritical flow conditions. The discharge capacity of the culverts has to be adequate so that the hydraulic jump would remain upstream of the N1, thus preventing flooding of the road. The damming height upstream of the N1 has to be at least 10% greater than the required water depth in order to sustain a stable hydraulic jump.

Flood attenuation dam

The intension with a flood attenuation dam is that it is used to attenuate the flood to the available culvert discharge capacity at the N1. The dam is normally empty and has a bottom outlet.

TABLE 7: Flood attenuation dam required heights

Flood attenuation dam characteristic with 1:20 year inflow hydrograph		
Flood peak target	**2 year**	**5 year**
Maximum outflow	26.8	35.1
Culvert height (d) (m)	1.8	1.4
Culvert width (b) (m)	1.6	1.6
Number of culverts	1	2
Bottom dam elevation (masl)	521	
Dam height (h) (excl. freeboard) (m)	8.3	6
Dam crest (excl. freeboard) (masl)	529.3	527
Total wall length (m)	354	324

Problems associated with attenuation dams are bed aggradation upstream and degradation downstream of the dam, as well as attenuation of small floods, not only large floods. The sinuosity of the main channel of the river downstream of the dam may also change as the river has less sediment to transport (initially following dam construction) and would experience smaller floods. The possible dam site that was investigated is shown in Figure 4.

Table 7 summarises the dam characteristics for different outflows. The proposed peak release discharges investigated were 25 m^3/s (2-year flood peak) and 34 m^3/s (5-year flood peak).

For the scenario where the outflow hydrograph peak is equal to the 2-year flood peak (25 m^3/s), the natural ground levels upstream of all the N1 culverts has to be lowered to form a hydraulic jump upstream of the road, forcing the supercritical flow from the attenuation dam to return to subcritical flow conditions. However, for the scenario where the outflow hydrograph peak is greater than the 2-year flood peak, the joint capacity of the existing N1 culverts near the man-made channel must be upgraded, and additional culverts are required in order to allow the outflow from the attenuation dam to traverse the N1 undisturbed. Concomitant with the upgrading of the culverts, the ground level upstream of all the N1 culverts have to be lowered to obtain subcritical flow conditions. For both scenarios, the capacity of the culverts has to be adequate so that the hydraulic jump would remain upstream of the N1 and not flood the road. The damming height upstream of the N1 has to be at least 10% greater than the required water depth in order to sustain a stable hydraulic jump. All culverts underneath the N1 must be designed with a USBR type stilling basin at the downstream end.

The simulation results for the study area with an attenuation dam with the 20 year flood as the inflow hydrograph and the 2 year flood peak (25 m^3/s) as the outflow hydrograph are illustrated in Figure 5 (flow depths) and Figure 6 (flow velocities). The number of culverts and culvert dimensions underneath the N1 road, as well as the ground levels upstream of the road were not amended for the simulations done on the study area with an attenuation dam.

Figure 5 to Figure 6 indicate that the supercritical flow downstream of the dam (2-year flood peak) still cause spillage over the road. If the ground levels upstream of all the culverts is lowered to form a hydraulic jump to create subcritical flow conditions for this scenario, the flood risk might be reduced.

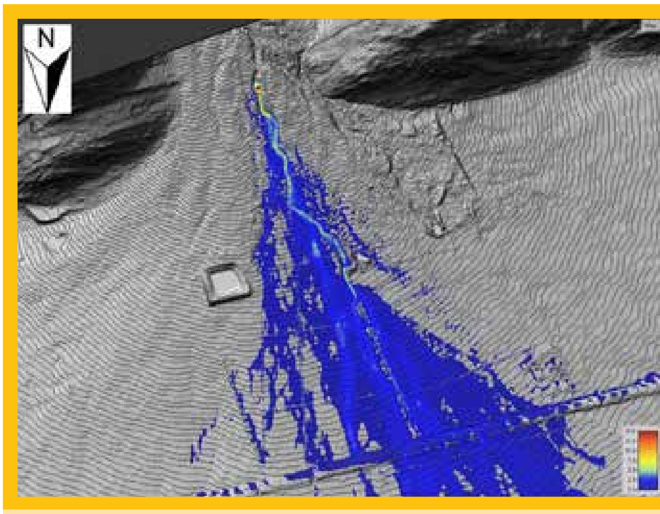

FIGURE 5: Simulated attenuation dam flow depths (m) at 20-year inflow peak (50 m^3/s) and 2-year peak outflow (25 m^3/s)

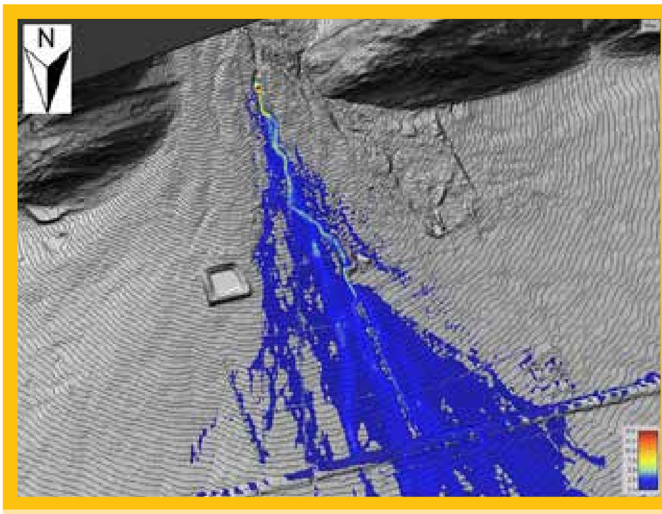

FIGURE 6: Simulated attenuation dam flow velocities (m/s) at 20-year inflow flood peak (50 m³/s) and 2-year peak outflow (25 m³/s)

FIGURE 7: Sam water levels (masl) at 20-year inflow flood peak (50 m³/s) and 2-year peak outflow (25 m³/s)

Figure 7 depicts the water levels through the attenuation dam with the 20-year flood peak as inflow hydrograph and the 2-year flood peak as the outflow hydrograph.

A flood attenuation dam has the benefit that local stormwater downstream of the dam could drain towards the man-made channel as seen in Figure 5 and Figure 6. The capital cost of an attenuation dam concomitant with the upgrade of the culvert discharge capacities traversing the N1 would be more costly than only upgrading the total culvert capacities to pass the 1:20 year flood (50 m³/s) underneath the N1. Moreover, an attenuation dam may lead to fluvial morphological impacts upstream and downstream of the dam. Therefore, a flood attenuation dam does not result into a viable option to reduce the flood risk.

Other short-term measures to consider

Some alternative short-term measures could also reduce the flood risk:

- Clear the area of boulders, alien vegetation, fallen and excessive trees planted on the floodplain that could block the culvert entrances. A riparian ecologist specialist should coordinate this so that ground erosion is not increased.
- Clear flow paths through the culverts and streamline the approaching flow conditions.
- The stability of the erosion protection along the floodplain should be monitored.

CONCLUSION

The flood analysis concluded that the agricultural developments did not have an impact on specifically the flood peaks. The developments did however change the flow regime. A probabilistic analysis of the AMS of H2H005 catchment confirmed that the Smithers software application, in this case, provided MAP values in excess of what is reasonable to expect in the Horlosiekloof catchment. The Rational method proved to be the more appropriate approach to be used.

A detailed 2D hydrodynamic modelling (HecRAS 5.0.3) simulation was conducted on the catchment. The modelling indicated that, while the combined capacity of the culverts may be adequate to deal with the 1:2-year flow, supercritical flow conditions exist upstream of the N1.

It was found that upgrading the discharge capacity of the existing culverts near the channel, and that additional culverts might reduce the flood risk. The lowering of the ground level upstream of all culverts will result into the formation of a hydraulic jump upstream of the N1 which will create subcritical flow conditions. The damming height upstream of the N1 has to be at least 10% greater than the required water depth in order to sustain a stable hydraulic jump.

A flood attenuation dam was evaluated as an alternative to upgrading the capacities of the culverts, but was found to be more costly than the proposed solution. Moreover, an attenuation dam may lead to fluvial morphological impacts upstream and downstream of the dam.

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REFERENCES

Brunner GW 2016. *HEC-RAS River Analysis System, 2D Modelling User's Manual*. Version 5.0, USA: US Army Corps of Engineers.

Chadwick A, Morfett J & Borthwick M 2013. *Hydraulics in Civil and Environmental Engineering*, ISBN 978-0-415-6245-0. CRC Press. 5th Edition.

Gericke OJ & Du Plessis JA 2012. Evaluation of the standard design flood method in selected basins in South Africa.

Rooseboom A & Van Vuuren SJ 2015. *Drainage Manual 6th Edition*. Chapter 7 – Lesser culverts (and stromwater conduits). The South African National Road Agency SOC Ltd Drainage Manual, ISBN 978-0-620-55428-2. Pretoria.

Van Dijk SJ, Van Vuuren SJ & Smithers JC 2015. *Drainage Manual 6th Edition*. Chapter 3 – Flood calculations. The South African National Road Agency SOC Ltd Drainage Manual, ISBN 978-0-620-55428-2. Pretoria.

Smithers JC & Schulze RE 2000. Long duration design rainfall estimates for South Africa. *Report No 811/1/00*, Pretoria: Water Research Commission.