

ELEMENT 2030

STORMWATER MASTERPLAN FOR ASHTON



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EXECUTIVE SUMMARY

This report contains the stormwater masterplan for the town of Ashton and Zolani. The project scope entails the compilation of a stormwater infrastructure register and the analysis of existing and proposed stormwater infrastructure within the Ashton urban edge. The methods of data collection are discussed, thereafter the existing stormwater infrastructure is addressed. The stormwater infrastructure was modelled with the EPA SWMM5 software.

The annexures contain all input data and results for the stormwater model, as well as the rainfall distribution curves for the various recurrence intervals which were modelled. The drawings show the existing stormwater infrastructure as well as the infrastructure with insufficient capacities for the 1 in 2 year flood events.

A total of 552 nodes were surveyed consisting of 133 manholes, 343 inlets and 55 wing walls. 16.2km of conduits were surveyed and consisted mostly of pipes, varying from Ø100mm to Ø900mm and concrete channels or culverts.

There are a few areas in Ashton and Zolani which is prone to flooding, either due to insufficient or lack of stormwater infrastructure or inadequate planning.

In order to improve the functionality of the existing stormwater infrastructure, a regular cleaning and maintenance program should be implemented.

STORMWATER MASTERPLAN FOR ASHTON

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1 SCOPE OF WORKS

Element 2030 was appointed by Cape Winelands District Municipality to compile the stormwater masterplan for Ashton. The project scope entails the compilation of a stormwater infrastructure register, and the analysis of stormwater infrastructure within the Ashton urban edge. The infrastructure register and plans are to be compiled from as-built drawings, supplied by the Langeberg Municipality, as well as physical site surveys and investigations.

The scope for this project includes the following:

- Liaison with the interested and affected parties (consultants, working for the Langeberg Municipality within the study area, and municipal officials);
- Data collection and capturing the existing stormwater system and related infrastructure within the study area;
- GPS data collection of the existing system within the Municipal boundary;
- Capturing the surveyed data in AutoCAD Civil 3D;
- Compilation of a stormwater model of the study area in EPA SWMM5;
- Analysis and modelling of the 1 in 2, 1 in 5 and 1 in 10 year return period flood events;
- Report on the results of the stormwater model, flagging the deficiencies and shortcomings of the existing system;
- Upgrade proposals for major stormwater drainage routes to discharge stormwater to the various rivers/streams in and around Ashton;
- Estimated costs of the proposed upgrades.

In this report the existing stormwater system is described, which gives some background to the modelling. The catchment hydrology and the relevant design parameters are discussed. Assumptions are made for modelling and the model is evaluated for various return periods for significant storm events. Upgrades are proposed where existing stormwater infrastructure has insufficient capacity.

2 DATA COLLECTION

2.1 Study Area

The study area was defined as the urban edge of Ashton, refer to the Locality Plan **Drawing 1800561/W/101 in Annexure A**. The cadastral information for Ashton was obtained from CNdV Africa (Pty) Ltd and is shown on **Drawing 1800561/C/104 to 106 in Annexure A**.

2.2 Existing Available Data

No stormwater as-built information was available. Site surveys were required for the entire Ashton area.

Construction drawings was supplied by Aecom for the R62 road and drainage infrastructure which is currently under construction. These were also included in the model as existing infrastructure.

2.3 Site Investigation and Survey

A survey was required of all the existing stormwater infrastructure in the Ashton area. The data required for the node and link tables were captured during these site surveys by means of Yuma GPS instrumentation and Pathfinder software.

The accuracy of this survey information is not the same level of accuracy of a topographical survey, as this survey information was required to compile an asset register and electronic model to simulate stormwater flow.

The survey comprised of nodes and conduits. A total of 552 nodes were surveyed, consisting of manholes, inlets, head walls and a few outlets with no formal structure. The conduits totalled 16.2km and varied from Ø100mm to Ø900mm pipes, culverts and open concrete channels.

2.4 Rainfall Data

The 0024110W Ashton (Pol) South African Weather Bureau rainfall station was used for the hydrological calculations. This station was representative of the study area and was used for all the subcatchment areas. The rainfall data of this station is presented in detail in section 5.2 and **Annexure D**.

2.5 Integrated Infrastructure Management Solutions (IMQS)

IMQS Software (Pty) Ltd was appointed to import the existing stormwater infrastructure data into the IMQS Stormwater software.

3 EXISTING STORMWATER INFRASTRUCTURE

The stormwater infrastructure which were surveyed, such as headwalls, manholes, kerb inlets and grid inlets, are represented as nodes in the EPA SWMM5 model. The surveyed pipes and channels were represented as conduits in the model. A detailed list of the parameters of each of these nodes and conduits are presented in **Annexure C**. The corresponding numbering of the nodes are shown on **Drawing 1800561/W/104 to 106** in **Annexure A**.

Overall maintenance and cleaning of the stormwater inlets such as catchpits, grid inlets and wing walls, is required as a moderate amount of these were clogged with litter or silted up. However, a maintenance team was busy cleaning the infrastructure during the survey stage of this project in the Zolani area. See figures 3.1 to 3.4.



Figure 3.1: Damaged Inlet In Mketsu Street (Node D20.25.1)



Figure 3.2: Damaged/Abandoned inlet in Ndinisa Street (Node D21.8.7.1.1)



Figure 3.3: New headwall structure with grid inlet in top slab (Node J2)



Figure 3.4: Grid Inlet in Faure Street (Node D8.9.4)

Maintenance on the conduits such as open channels and culverts seemed good as very few of these were clogged or contained debris. See figures 3.5 to 3.8.



Figure 3.5: Amorfex channel adjacent to the railway (Conduit C7.10)



Figure 3.6: Dish channel between Matotl Avenue and Ntantiso Close (Conduit C21.8.12)



Figure 3.7: Open concrete channel adjacent to R60 (Conduit C20.2)



Figure 3.8: Pipe and stone pitch channel between Tyhalisisu and Maqolo Streets (Conduits C20.5 & C20.4)

Although conduits/stormwater pipes with smaller diameters than Ø375mm are not specifically highlighted, it is not good practice to install smaller pipes than Ø375mm, as part of the minor stormwater system.

4 CATCHMENT HYDROLOGY

The hydrology component of the SWMM software operates on a collection of subcatchment areas, which receive precipitation and generate runoff loads as simulated.

4.1 Catchment Boundary Delineation

Ashton is situated between the Martiensvlei River on the western side, Wildeperdehoek River on the eastern side, Kogmanskloof River on the southern boundary and the Montagu Mountain Nature Reserve on the northern boundary. All stormwater runoff from the Ashton urban area discharges into one of the adjacent rivers.

Zolani is situated between the R60 on the south-western side, the Marloth Nature Reserve on the north and eastern side and then the Sarahs River on the south. All stormwater from the Zolani area discharges into the Sarahs River either directly or by means of furrows and open channels.

The catchment boundary delineation was done by assessing the infrastructure routes of the system, the surveyed invert levels, the contours, and the cadastral layout. Some roads serve as cut-offs and were used to delineate the boundaries of the subcatchments.

The major drainage routes were divided into 28 catchment areas, 19 in Ashton and 9 in Zolani. Most of these catchments were divided into smaller subcatchments for modelling purposes with a total of 137 subcatchments. These subcatchments are shown on **Drawings 1800561/W/102 & 103 in Annexure A**. The runoff from these smaller subcatchments are linked to drain into the identified nodes in the respective subcatchment. The hydrological parameters and sizes of these subcatchments are summarised in **Annexure B**.

4.2 Hydrological Design Parameters

The SCS design runoff method was used to calculate the runoff from the subcatchments in the model. The area, average catchment slope, curve number and lag time for the subcatchments were some of the hydrological parameters required for the SCS method.

The curve number (CN) is a dimensionless number depending on the hydrologic soil group, cover type, hydrological conditions, and antecedent moisture conditions. Table 4.1 gives an indication of the curve numbers for the various soil groups. These weighted CN values are

calculated based on the areas within a subcatchment representing a specific soil type and land use category. The CN values correspond to specific soil types and land uses.

Table 4.1: Assumed Curve Number

CURVE NUMBER	Hydrological soil group						
	A	A/B	B	B/C	C	C/D	D
High density	77	81	85	88	90	91	92
Medium density	57	65	72	77	81	84	86
Low density	51	61	68	75	78	82	84
Open spaces	49	61	69	75	79	82	84
Stormwater body	100	100	100	100	100	100	100

The four soil groups are defined as follows:

- **Group A** (low runoff potential): Soils with high infiltration rates even when thoroughly wetted. These consist mostly of deep, well-drained sands and gravels.
- **Group B** (moderate infiltration rates when thoroughly wetted): These consist of soils that are moderately deep to deep, moderately well drained to well drained with moderately fine to moderately coarse textures. Typically, sandy or sandy loam type soils.
- **Group C** (slow infiltration rates when thoroughly wetted): These consist of soils with a layer that impedes downward movement of water or soils with moderately fine to fine textures. Typically, clayey sand, silts and sandy loam type soils.
- **Group D** (high runoff potential): Soils with very slow infiltration rates when thoroughly wetted. These consist mainly of clay soils with a high swelling potential, soils with a permanent high-water table, soils with a clay layer at or near the surface, and shallow soils over nearly impervious materials.

The soil types throughout Ashton are mostly consistent. It was therefore decided to use the B hydrological soil group throughout the model.

5 STORMWATER MODELLING

5.1 Assumptions

The following assumptions were made for the SWMM model:

- All concrete pipes were assumed with a Manning's roughness value of 0.013.
- All concrete channels were assumed to have a Manning's roughness value of 0.013.
- A Manning's roughness value of 0.025 was used for roads.
- All earth channels and rivers were assumed to have a Manning's roughness value of 0.05.
- The SCS Type II storm distribution was used to distribute the 24h rainfall depth.
- Evaporation was assumed to be zero during storm events.
- All conduits of all systems were assumed to be in serviceable condition for the modelling purposes.

5.2 Rainfall

Rainfall data of the Ashton (0024110W) rainfall station was considered to determine 24-hour rainfall depths for the different recurrence storms. This station is representative of the study area and was used for all the subcatchments in Ashton and Zolani.

Table 5.1: 24 Hour Rainfall Depths

RAINFALL STATION	RECURRENCE INTERVAL (YEARS)					
	2	5	10	20	50	100
Ashton (0024110W)	34.0	50.0	62.0	75.0	96.0	113.0
24-hour depth for model (mm)	37.7	55.5	68.8	83.3	106.6	125.4

SCS Type II distribution with 15 min intervals and a total depth in accordance with the 24-hour depths in Table 5.1 were applied in the modelling. The rainfall distribution graphs are shown in **Annexure D**.

5.3 Drainage System Modelling

The modelling of the existing drainage system was performed in the EPA SWMM5 software and is presented as layout **Drawing 1800561/W/104 to 106** in **Annexure A**, while all input data relating to the nodes and conduits is presented in **Annexure C**. The modelling of the system is discussed below.

5.3.1 Nodes

The nodes for the minor system are located at changes in the type of conduit or at the junction between conduits. Typical nodes are manholes, kerb inlets, grid inlets and wing walls. These nodes have a prefix "J" or "D" to distinguish from other model elements.

The coordinates, depths and cover levels of the nodes were gathered from the survey information and imported into the model. A total of 552 nodes were surveyed and consisted of 133 manholes, 343 inlet structures, 55 wing walls and 21 outlets with no formal structure.

5.3.2 Conduits

The conduits in the minor system are mainly pipes, whereas that of the major system are open channels and roads. The dimensions of the minor and major system conduits were obtained from the survey information.

The conduits which were surveyed consisted mostly of pipes, varying from Ø100mm to Ø900mm, culverts and open concrete channels. The total length of the conduits in Ashton are 16.2km. All conduits have a "C" prefix.

5.3.3 Dividers

Most of the nodes in the model were defined as dividers. Flow Dividers are drainage system nodes that divert inflows to a specific conduit in a prescribed manner. For this project, all dividers were classified as overflow dividers, which will divert inflow above the flow capacity of the minor system to the overland conduit, for example streets and natural watercourses. All dividers have a "D" prefix.

5.3.4 Diverted Conduits

For the overflow system, it was assumed that the stormwater will overflow into the roads to the downstream node. The overflow conduit for the diverted conduit of the model was defined as a wide and shallow rectangular channel (to simulate a road). The overflow conduits were defined with a depth of 200mm and the road width.

To prevent losses, the width of the overflow conduits was increased as required to accommodate the overland flow in the major storms. All overflow conduits have an "O" prefix before the route number to make a distinction between a regular conduit and an overflow conduit. These conduits were not included in the registers in the Annexures.

6 HYDRAULIC ANALYSIS RESULTS

The model was simulated for the 1 in 2, 1 in 5, 1 in 10, 1 in 20, 1 in 50 and 1 in 100 recurrence intervals. Since the status reports from these model runs are comprehensive, it was decided not to include it in this report, but rather have it available in electronic format. The summary of the results of these runs are summarized in **Annexure E**.

The 1 in 2 year flood event is normally seen as the minor flood event and the 1 in 50 flood event as a major flood event. According to the "Guidelines for Human Settlement Planning and Design", the minor stormwater system should provide sufficient capacity for up to the 1 in 2 year flood peak runoff in residential areas. Therefore, the results of the 1 in 2 minor flood event are critical for the analysis of the minor stormwater system.

The links with insufficient capacities for the 1 in 2 year flood event was highlighted on **Drawings 1800561/W/107 to 109 in Annexure A**.

Areas in Ashton which were identified where flooding possibly occurs on a regular basis due to insufficient stormwater infrastructure are the following:

- The stormwater system in Vygie and Dahlia Street, in the north-western part of Ashton, has insufficient capacity for the 1:2 year flood runoff. However, the additional inlets in Magnolia Street will alleviate/prevent flooding of residential erven. An Ø600mm pipe is proposed.
- The existing Ø600mm midblock stormwater system between Coronation and Disa Streets (north of the sport field) have insufficient pipe capacity for the 1:2 year flood runoff. It is proposed that the system be upgraded to a Ø750mm pipe or be re-routed down Wilger Street.
- The existing Ø675mm pipe, adjacent to the nursery has insufficient capacity for the 1:2 year flood runoff. **The existing Ø675mm pipe can be either replaced by a Ø1200mm pipe or an additional parallel pipe can be constructed.**
- The existing Ø600mm stormwater pipe adjacent to the railway, between the canning factories, have insufficient pipe capacity for the 1:2 year flood runoff. It is proposed that the system be upgraded to a Ø1200mm pipe. However, there is sufficient overland escape route towards the Kogmanskloof River for runoff to drain freely.
- The existing Ø375mm stormwater pipe in Barry Street have insufficient pipe capacity for the 1:2 year flood runoff. It is proposed that the system be upgraded to a Ø450mm pipe.

- In Roodewal Street the existing Ø375mm stormwater pipe have insufficient capacity for the 1:2 year flood runoff. However, there is sufficient overland escape route towards the stream for runoff to drain.
- In Station Road the existing Ø375mm stormwater pipe have insufficient capacity for the 1:2 year flood runoff, there is however adequate overland escape routes for the runoff to prevent erven being flooded.

Areas which are likely to experience flooding/ponding due to the lack of infrastructure or town planning are the following:

- In Orgidee Crescent at node D3.3.1.2 where the overland drainage route towards the river runs through the properties (see **Drawing 1800561/W/107 in Annexure A**)
- In Narsing Crescent (node 4.6.3) and Madeliefie Close (node 4.6.4), where the urban layout is forcing overland stormwater to run into the properties (see **Drawing 1800561/W/107 in Annexure A**)

Areas in Zolani which were identified where flooding possibly occurs on a regular basis due to insufficient stormwater infrastructure are the following:

- In Mketsu street three sections were identified where the existing stormwater system do not have sufficient capacity for the 1 in 2 year flood runoff. Upgrades are proposed for these sections of pipes. One area was identified which could be prone to regular ponding or flooding.
- In Tshoto Street and Dudumashe Avenue sections of the existing Ø375mm stormwater pipe have insufficient capacity for the 1:2 year flood runoff. Upgrading to Ø450mm stormwater pipes are proposed for this system.
- In Motasi Avenue an area was identified as a local low point which might cause flooding to residential erven should the inlets be blocked, or a major storm event occur.
- The other areas adjacent to the Sarahs River which are shown on **Drawing 1800561/W/109 in Annexure A** have potential ponding or flooding areas. During flood events the residential erven might be prone to flooding.

These potential flooding areas are mainly due to limited capacity of infrastructure and might result in erven being flooded, and roads being inaccessible which cause inconvenience to residents.

7 RECOMMENDED UPGRADES AND COSTINGS

Since the 1 in 2 year minor flood event is critical for the analysis of the normal functioning of the minor stormwater system, insufficient capacities in drainage systems in this analysis needs to be addressed.

Hence, the upgrades proposed were sized to provide capacity for the 1 in 2 year storm runoff. An EPA SWMM model was set up and all the proposed upgrades were included into the model to ensure that the proposals had sufficient capacity. These proposals are summarized in **Annexure F**.

The following table presents a high order cost estimate should all the proposed upgrades be considered. A detail design with detailed estimate is recommended before the upgrading continues to the construction stage.

Table 7.1: Cost Estimate for Pipe Minor System Upgrades

Description		Amount
Preliminary and General (20%)		R 1 006 000
Stormwater Drainage		R 5 027 700
Pipes	R 3 763 700	
Manhole	R 340 000	
Headwalls	R 30 000	
Grid /Kerb inlet	R 204 000	
Reinstate Road surface	R 690 000	
Sub-total		R 6 033 700
Contingencies (10%)		R 603 370
Sub-total		R 6 637 070
VAT (15%)		R 995 561
TOTAL		R 7 632 631

8 CONCLUSIONS

The existing stormwater infrastructure was surveyed and verified. The major and minor stormwater infrastructure for Ashton and Zolani were modelled with EPASWMM software for the 1 in 2, 1 in 5 and 1 in 10 peak runoffs.

A total of 552 nodes and 16.2km of conduits were surveyed consisting of 133 manholes, 343 inlet structures, 55 wing walls and 21 outlets. 16.2km of conduits were surveyed and consisted mostly of pipes, varying from Ø100mm to Ø900mm, culverts and open channels.

A moderate amount of the conduits of the minor system have insufficient capacity for the 1 in 2-year peak runoff. Several areas were identified as low-points without an overland escape route as well as conduits with insufficient capacity for the 1 in 2 year flood runoff.

The major problem area was identified as the existing Ø675mm pipe, adjacent to the nursery which has insufficient capacity for the 1:2 year flood runoff. **The existing Ø675mm pipe can be either replaced by a Ø1200mm pipe or an additional parallel pipe can be constructed.**

Upgrading proposals and cost estimates were presented for the conduits with insufficient capacities.

APPENDIX A - DRAWINGS