Sellicks Beach Structure Plan

Stormwater Management Assessment

City of Onkaparinga

7 September 2020
Ref: 201217R002B

Building exceptional
outcomes together

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Client: City of Onkaparinga
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Appendix A – 1% AEP flood inundation map

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1 Introduction

The City of Onkaparinga (Council) is undertaking master planning for the Sellicks Beach area to inform where, how and when different types of development can occur, and to identify the infrastructure and services required to facilitate this. The masterplan will focus on all remaining areas zoned as 'deferred urban' and 'primary production', outside of the character preservation district (as defined in Council's Development Plan). It is envisaged that the majority of rezoning will be to support residential development.

As part of the master planning works, Tonkin has been engaged to undertake an assessment of the likely stormwater management requirements associated with the future development of the area. The study area for the stormwater management assessment is defined by the Sellicks Beach future urban growth area, which has been identified for urban expansion in the 30 Year Plan for Greater Adelaide (DPTI, 2017). This area is bounded by the orange line as shown in Figure 1.1. The assessment will consider flows within the four watercourses to the west of Main South Road which pass through the study area, in addition to runoff generated from the developable parcels of land.



Figure 1.1 Stormwater management assessment study area

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2 Flood inundation mapping

In order to determine the floodplain extents for the 1% annual exceedance probability (AEP) event, two-dimensional modelling of the four main watercourses passing through the study area has been undertaken using TUFLOW. This section describes the modelling approach and presents the 1% AEP flood inundation map for the existing scenario.

2.1 Hydrological modelling

2.1.1 Estimation of external catchment runoff

The northernmost creek has a rural catchment with an area of approximately 3.77 km^2 upstream of the study area. The runoff hydrographs from this catchment were estimated using RORB, and were used as the inflow hydrographs at the TUFLOW model boundary. The sub-catchments contributing runoff to this boundary are shown in Figure 2.1. Details of the internal drainage configuration of Sellicks Hill Quarry are not known; runoff from this area has been assumed to follow historical flow paths. Catchment delineation was undertaken using available contour data and aerial imagery. The delineation was undertaken in a manner that ensured:

- At least five upstream sub-areas were produced with areas not less than 5% or greater than 30% of the overall catchment.
- Stream lengths were sub-divided to ensure that none were more than 33% of the main stream length.
- Short streams and small sub-areas were removed to avoid over-attenuation of flows.

The adopted RORB model parameters are summarised in Table 2.1. The routing coefficient was estimated using the regional equation for average annual rainfall greater than 500 mm (Kemp, 1993). The initial loss value was obtained from the Australian Rainfall and Runoff (ARR) data hub.

Table 2.1 RORB input parameters

Parameter	Value
Impervious area	0%
Routing coefficient, k_c (k_c = 0.89 x Area$^{0.55}$)	1.88
Storage parameter, m	0.8
Initial loss	24 mm
Continuing loss	3 mm/hr

The RORB modelling estimated a 1% AEP peak flow of 13.9 m^3/s within the creek at the upstream end of the study area, with a critical storm duration of 3 hours.

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Figure 2.1 RORB sub-catchments

2.1.2 Estimation of runoff within the study area

A direct-rainfall approach (‘rainfall on grid’) was used to estimate the runoff generated within the study area. The direct-rainfall approach applies rainfall directly to the grid cells of a two-dimensional hydraulic model and allows the hydraulic model to determine the accumulation of runoff across the catchment.

Rainfall intensity-frequency-duration (IFD) data used in the model were sourced from the Bureau of Meteorology’s design rainfall data system. To understand the variability in catchment response due to varied rainfall inputs, an ensemble of ten temporal patterns was used for each storm duration. The temporal patterns were obtained from the ARR data hub.

2.2 Hydraulic modelling

Hydraulic modelling uses the outputs of hydrological modelling to determine the extent, depth and behaviour of floodwater. The resulting outputs provide an estimate of areas subject to flooding.

The digital elevation model (DEM) supplied by Council covers the full extent of the study area, and was used in the model to represent the existing surface. A computational grid size of 4 m was adopted within the model. Where a higher level of detail was required, such as along watercourses and roads, this was refined to a 2 m grid. A further refinement to 1 m was undertaken within the watercourse immediately downstream of Country Road, to better represent the flow regime across the existing weir.

The bed resistance is a primary determinant of water depth within the model domain. The Manning’s roughness coefficient (n) is used to define the bed resistance. The values adopted in the model are

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shown in Table 2.2. The assumed imperviousness of each land use type within the study area is also presented.

For the direct-rainfall method it is important to vary the roughness coefficient with the depth of flow to ensure a proper representation of the catchment response to rainfall. A high roughness coefficient for shallow flows will allow the resistance experienced by runoff moving past obstacles, such as vegetation, to be replicated. Eventually runoff will accumulate to a sufficient depth that exceeds the height of the majority of obstacles and the resistance to flow will decrease. This process has been replicated in TUFLOW by specifying roughness coefficients for shallow (less than 20 mm) and deep (greater than 100 mm) flow for the grassed areas and channels.

Table 2.2 Adopted bed resistance parameters

Land use type	Fraction impervious	Manning's roughness (shallow flow)	Manning's roughness (deep flow)
Grassed areas	0	0.2	0.035
Channels	0	0.2	0.055
Residential	0.6	0.2	0.2
Roads	0.9	0.02	0.02

No hydraulic modelling of the existing underground drainage network (i.e. one-dimensional modelling) within Sellicks Beach was undertaken. However, pipes and culverts were added to the model at key locations, including at creek crossings (such as Country Road and Gulf View Road) to ensure that flows could continue to be conveyed through the watercourses. As the pipes and culverts passing under roads were not in the Council GIS data an approximation of their sizes has been made. Verification of these pipe/culvert sizes would be required to ensure the flood conveyance capacity is accurately captured. Some pipe details were obtained from the "Bluewater @ Sellicks Beach Drainage Works" design drawings.

For this study, one storm frequency (i.e. 1% AEP) was modelled. Six different storm durations (i.e. 1 hour, 1.5 hours, 2 hours, 3 hours, 4.5 hours and 6 hours) were simulated to ensure that the critical storm duration (the duration which produces the maximum flood depth) had been simulated for all areas. Ten distinct temporal patterns were used for each storm duration, resulting in a total of 60 simulations being run to determine the flood inundation extent.

2.3 Flood modelling results

For each model run, the peak flood depth was recorded across the extents of the model. As recommended by ARR, the median depth from the ten temporal patterns was adopted. The results from each storm duration were then spliced together to create a maximum depth envelope for the 1% AEP event. A map showing the extent of inundation is shown in Figure 2.2, and is replicated in Appendix A.

Upstream of Sellicks Beach Road, the profile of Sellicks Creek (Creek 1 – refer label in Figure 2.2) is deep, with depths in the order of 4-7 m. Within this section, runoff from the 1% AEP event is contained within the channel. Flows arriving at Sellicks Beach Road appear to overtop the road (based on an assumed cross culvert size under Sellicks Beach Road of 3 m x 0.75 m). Downstream of the road, the water surface elevation is closer to the top of bank levels, particularly within the wetland's open pond areas. It is likely that the increased runoff associated with future development would worsen flooding issues at Sellicks Beach Road, and potentially exceed the capacity of the downstream reach.

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Seascape View
Maritime Avenue
Seashore Avenue
Luriline Boulevard
Shoreline Avenue
Milford Avenue
Riviera Road
Plaza Avenue
Capri Crescent
Lascala Avenue
Fontaine Drive
Monaco Boulevard
Casino Boulevard
Cannes Drive
Tangier Boulevard
Justs Road
Dundee Street
Stirling Avenue
Armstrong Street
George Parade
Norman Victory Parade
Bay View
Dunrossil Avenue
Aberdeen Street
Sellicks Beach Road
Main South Road
PO_9
PO_8
Herrick Street
Francis Street
Herrick Street
Alexander Street
Arcadia Crescent
Aurora Terrace
Oriana Drive
Virgo Parade
Robert Street
Gisborne Avenue
PO_13
PO_14
PO_15
Dunedin Avenue
Wellington Avenue
Gulf View Road
Country Road
CREEK 1
CREEK 2
CREEK 3
CREEK 4
Napier Avenue
Hastings Avenue

Legend

- Recorded flow location
- Model boundary
- Study area

Depth of inundation

- Less than 0.05m (not shown)
- 0.05m to 0.1m
- 0.1m to 0.2m
- 0.2m to 0.3m
- 0.3m to 0.4m
- 0.4m to 0.5m
- 0.5m to 0.6m
- 0.6m to 0.7m
- 0.7m to 0.8m
- 0.8m to 0.9m
- 0.9m to 1.0m
- 1.0m to 10.0m
- 10.0m to 100.0m

0 200 400 600 m

N

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Revision: A
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Data Acknowledgement:
Aerial imagery from MetroMap, 2017
Roads from Data SA, 2018

CITY OF ONKAPARINGA

SELLICKS BEACH STRUCTURE PLAN
STORMWATER MANAGEMENT ASSESSMENT
1% AEP FLOOD INUNDATION MAP

Figure 2.2

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Runoff within the upstream portion of Creek 2 and Creek 3 is contained within the creek profiles. The DEM shows that these channels terminate at a location approximately 230 m south of Virgo Parade. From this point, runoff travels across the surface towards the north as shallow sheet flow, converging with Creek 4 upstream of Virgo Parade.

The upstream end of Creek 4 does not follow a well-defined flow path, with flows observed to sheet across the surface towards Gulf View Road. Upstream of Gisborne Avenue, runoff from Creek 4 is guided towards a natural gully, from where it continues to travel towards the north in the channel between Alexander Street and Aurora Terrace. Runoff from the residential catchment to the east of Aurora Terrace is discharged to the creek downstream of Arcadia Crescent. Downstream of Sellicks Beach Road, runoff converges with the flows from Creek 1 within the wetland.

The water level within Creek 4 during the 1% AEP event, particularly within the section between Arcadia Crescent and Sellicks Beach Road, is either at or above the top of bank levels. This indicates that the channel is at capacity, and would likely be unable to accommodate un-detained flows from areas of future upstream development.

Council has indicated that development will not occur within 40 m of each of these creeks (20 m each side of the creek centreline). Review of the flood map confirms that development outside of this buffer zone will not be located within the extents of the existing major flood flow paths.

The DEM shows that there is a small gully within the undeveloped catchment to the east of Oriana Drive. Runoff from this catchment appears to result in some minor inundation of private properties within Star Princess Court and Oriana Drive.

Flooding within the open land to the north of Sellicks Beach Road appears to be minor in nature, draining in a north-westerly direction towards Justs Road.

Peak 1% AEP flows were recorded at a number of locations within the study area, denoted by a 'PO' number in Figure 2.2. A summary of these flow rates is provided in Table 2.3.

Table 2.3 Peak 1% AEP flows

Location	Flow rate (m^3/s)
PO_8	4.4
PO_9	13.6
PO_13	3.7
PO_14	0.96
PO_15	1.1

In addition to the depth of inundation, TUFLOW was used to model the velocity of flows within the study area for the 1% AEP event. The results of the velocity modelling are shown in Figure 2.3. Flood velocity in non-creek areas is relatively low, generally less than 0.1 m/s. Some areas of high velocity are expected within Creek 1, particularly within the section downstream of Country Road, where modelled velocities are as high as 3.6 m/s and scouring of the channel is known to be an issue.

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Seascape View
Maritime Avenue
Seashore Avenue
Luriline Boulevard
Shoreline Avenue
Milford Avenue
Riviera Road
Plaza Avenue
Capri Crescent
Lascala Avenue
Fontaine Drive
Monaco Boulevard
Casino Boulevard
Cannes Drive
Tangier Boulevard
Justs Road
Bay View
Norman Victory Parade
Dundee Street
Armstrong Street
George Parade
Dunrossil Avenue
Aberdeen Street
Stirling Avenue
Main South Road
Sellicks Beach Road
Herrick Street
Francis Street
Herrick Street
Alexander Street
Arcadia Crescent
Aurora Terrace
Oriana Drive
Virgo Parade
Robert Street
Gisborne Avenue
Dunedin Avenue
Wellington Avenue
Country Road
CREEK 1
CREEK 2
CREEK 3
CREEK 4
Gulf View Road
Main South Road
Napier Avenue
Hastings Avenue

Legend

- Model boundary
- Study area

Velocity

- 0 m/s
- 0.4 m/s
- 0.8 m/s
- 1.2 m/s
- 1.6 m/s
- 2 m/s
- 2.4 m/s
- 2.8 m/s
- 3.2 m/s
- 3.6 m/s

0 200 400 600 m

N

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SELLICKS BEACH STRUCTURE PLAN
STORMWATER MANAGEMENT ASSESSMENT
1% AEP FLOOD VELOCITY MAP

Figure 2.3

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3 Stormwater management

This section outlines the stormwater infrastructure planning assessment that has been undertaken for Sellicks Beach. Based on a review of available contour data, the study area has been separated by discharge location into three regions. The boundaries of each region and discharge location are shown in Figure 3.1. The existing stormwater infrastructure data provided by Council is also shown for reference.

3.1 Hydrological modelling

Hydrological modelling of the catchments shown in Figure 3.1 was undertaken to provide estimates of pre- and post-development peak flows generated by each catchment (as opposed to the peak flows within the creeks, described in Section 2). The modelling was undertaken using DRAINS (version 2020.036) with the same ensemble of ten rainfall patterns as that used in the TUFLOW model.

An ILSAX-type hydrological model was used to simulate runoff from the study area. The hydrological parameters used in the model vary between the pre- and post-development scenarios, as the site will be changing from rural, open land to a developed precinct. The parameters are summarised in Table 3.1. The pre-development storm losses (initial and continuing) were obtained from the ARR data hub. The post-development grassed depression storage is based on research undertaken by Kemp (1999). The post-development continuing loss allows for changes in the pervious area once the site is developed. It should be noted that runoff from pervious areas will not be the primary contributor of flows following development of the site; post-development flows will be predominantly governed by runoff from impervious areas.

Table 3.1 DRAINS model parameters

Parameter	Pre-development	Post-development
Paved area depression storage (equivalent to an impervious area initial loss)	1 mm	1 mm
Grassed area depression storage (equivalent to a pervious area initial loss)	24 mm	45 mm
Continuing loss	4.6 mm/hr	3 mm/hr

Similarly, the catchment imperviousness differs from pre- to post-development. For the existing scenario, aerial imagery was used to define the impervious fraction. The DPTI hydrology master specification (DPTI, 2019) was used as a guide to define the post-development imperviousness, assuming the catchment would predominantly be characterised by residential allotments with an average area of 700 m^2. For comparison, a secondary assessment was undertaken assuming an average lot size of 450 m^2. The catchment impervious areas adopted in the model are shown in Table 3.2.

Table 3.2 Catchment impervious percentage

Parameter	Pre-development	Post-development (lot size 700 m^2)	Post development (lot size 450 m^2)
Paved area	0	50	70
Supplementary paved area	0-5	15	10
Grassed area	95-100	35	20

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Button Road
Justs Road
Clark Court
Dundee Street
Armstrong Street
Lanark Street
George Parade
Norman Victory Parade
Bay View
Dunrossil Avenue
Aberdeen Street
Cannes Drive
Tangier Boulevard
Riviera Road
Plaza Avenue
Lascala Avenue
Fontaine Drive
Monaco Boulevard
Casino Boulevard
Maritime Avenue
Seascape View
Seashore Avenue
Lurline Boulevard
Shoreline Avenue
Milford Avenue
Sellicks Beach Road
Herrick Street
Francis Street
Alexander Street
Arcadia Crescent
Oriana Drive
Virgo Parade
Robert Street
Gisborne Avenue
Dunedin Avenue
Wellington Avenue
Hastings Avenue
Napier Avenue
Ranfurly Avenue
Country Road
Gulf View Road
Main South Road
Cat 1
Cat 2
Cat 3
Cat 4
Cat 5
Cat 6
Cat 7
1
2
3

Legend

- Discharge location
- Box culvert
- Contour 5 m
- Watercourse

Discharge region

- Region 1
- Region 2
- Region 3
- Storage

Stormwater pipe

- 150
- 225
- 300
- 375
- 450
- 525
- 600
- 675
- 750
- 825
- 900

0 200 400 600 m

N

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SELLICKS BEACH STRUCTURE PLAN
STORMWATER MANAGEMENT ASSESSMENT
DISCHARGE LOCATIONS

Figure 3.1

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Estimates of the surface flows from each of the catchments are provided in Table 3.3.

Table 3.3 Modelled catchment 1% AEP peak flow rates

Catchment	Catchment area (ha)	Pre-development flow rate (m^3/s)	Post-development flow rate (lot size 700 m^2) (m^3/s)	Post-development flow rate (lot size 450 m^2) (m^3/s)
Cat 1	30.9	1.15	5.08	7.11
Cat 2	26.2	0.86	4.05	5.68
Cat 3	29.7	1.27	4.88	6.83
Cat 4	8.7	0.15	1.49	2.08
Cat 5	3.4	0.12	0.61	0.85
Cat 6	10.2	0.38	1.75	2.45
Cat 7	14.3	0.49	2.35	3.29

3.2 Regional scale detention basins

With the increased imperviousness following development of the area, the resulting increase in peak flows is significant. To manage this, it is proposed that detention storages be constructed to detain post-development peak flows from the 1% AEP event to the pre-development level, such that the carrying capacity of the downstream systems is not exceeded. Two options have been investigated. Option 1 is with the basins kept off-line (refer Section 3.2.1), while option 2 is to have them on-line (refer Section 3.2.2. Based on the fewer basins involved for maintenance purposes, the recommended option is to have the basin on-line.

3.2.1 Basins within each catchment

The DRAINS model was used to size regional scale stormwater infrastructure to facilitate the required reduction in peak flows. Basins are proposed at the downstream end of each catchment, with all runoff from the developed catchment directed to a basin via a new piped stormwater system. The basins have been located as far downstream as possible to minimise the erosion risk along the existing watercourses. Positioning the basins any further upstream would make it difficult for the downstream portions of the site to drain via gravity. Additionally, the basins have been located off-line to the existing watercourses such that their water quality functions are not diluted by the large upstream catchments.

The modelling has been undertaken for the 700 m^2 allotment size scenario. A summary of the basin properties is provided in Table 3.4. The basin volumes have been calculated assuming a regular (i.e. rectangular) shape, with batter slopes of 1V:5H. Where this cannot be achieved, the basin configurations (i.e. depth and surface area) may differ, however the volume should be maintained.

Given the fall of the terrain, it may be possible to balance the cut and fill of the basins, with material won from excavation of the upstream segment used to form the downstream edges, which would sit above natural surface level. The estimated basin footprints, which include an additional allowance for the fall across the terrain, are shown in Figure 3.2.

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BASIN OUTLET TO CONNECT TO EXISTING ø525 DRAIN WITHIN JUSTS ROAD

FORMALISE WATERCOURSE TO BETTER GUIDE RUNOFF TO THE NORTH-WEST

FORMALISE WATERCOURSE (OR CONVERT TO PIPE)

Button Road
Justs Road
Clark Court
Dundee Street
Armstrong Street
Lanark Street
George Parade
Dunrossil Avenue
Aberdeen Street
Norman Victory Parade
Bay View
Cannes Drive
Tangier Boulevard
Riviera Road
Maritime Avenue
Seascape View
Seashore Avenue
Lurline Boulevard
Shoreline Avenue
Milford Avenue
Plaza Avenue
Lascala Avenue
Fontaine Drive
Monaco Boulevard
Casino Boulevard
Sellicks Beach Road
Herrick Street
Francis Street
Alexander Street
Arcadia Crescent
Orlando Drive
Virgo Parade
Robert Street
Gisborne Avenue
Dunedin Avenue
Wellington Avenue
Country Road
Gulf View Road
Main South Road
Ranfurly Avenue
Cat 1
Cat 2
Cat 3
Cat 4
Cat 5
Cat 6
Cat 7

Legend

- Basin outlet pipe
- Watercourse
- Basin footprint

Discharge region

- Region 1
- Region 2
- Region 3

0 200 400 600 m

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SELLICKS BEACH STRUCTURE PLAN
STORMWATER MANAGEMENT ASSESSMENT
INDICATIVE BASIN FOOTPRINTS

Figure 3.2

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Table 3.4 Basin properties required to detain 1% AEP peak flows to pre-development levels

Catchment	Storage volume (m^3)	Estimated footprint (m^2)	Storage depth (m)	Outlet pipe size (mm)	Discharge rate (pipe) (m^3/s)	Discharge rate (overflow) (m^3/s)
Cat 1	10,700	8,840	2	375	0.40	0.62
Cat 2	8,430	7,150	2	375	0.40	0.38
Cat 3	9,650	8,060	2	375	0.40	0.72
Cat 4	3,400	3,250	2	225	0.13	0
Cat 5	900	1,950	0.7	300	0.12	0
Cat 6	1,600	2,600	1	375	0.26	0.70
Cat 7	3,240	3,120	2	375	0.39	0

There is an existing 525 mm diameter pipe located along the western side of Justs Road. It is proposed to connect the outlet of the basin within Catchment 1 into this pipe. Hydraulic modelling of this existing network has not been undertaken; confirmation that there is capacity within the pipe to accommodate the flows from the basin outlet is required. Overflows from the basin should be directed to the north/north-west along existing flow paths.

Each of the remaining six basins will discharge directly into the adjacent watercourses. Aside from the basin outlet pipes, no additional piped drainage outfalls will be required.

Stated basin volumes are likely to increase by approximately 30-40% if 450m^2 lot sizes are used in the modelling.

3.2.2 Combined basin configuration

It is acknowledged that maintaining seven individual basins may not be practical or the most efficient way for Council to achieve the required storage volumes. Investigation into whether the basins can be combined has been undertaken.

The external catchment contributing runoff to Sellicks Creek is large (approximately 3.77 km^2). Merging the basins within Catchments 2 and 3 would require construction of an on-line basin; this merged basin would accept flows from the developed catchments as well as the large external catchment. Similarly, merging the basins within Catchments 4-7 would result in an on-line basin which also receives flows from the contributing catchment upstream (1.53 km^2).

Basin sizing has been undertaken to limit the post-development flow rate from the combined catchments to pre-development levels. While runoff from the external catchments will pass through each basin, the sizing of detention storage has not considered the magnitude of these external flows, which will overtop the basins and continue along existing flow paths.

A summary of the combined basin properties is provided in Table 3.5, while the estimated footprints are shown in Figure 3.3.

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Button Road
Justs Road
Clark Court
Dundee Street
Armstrong Street
Lanark Street
George Parade
Dunrossil Avenue
Aberdeen Street
Norman Victory Parade
Bay View
Cannes Drive
Tangier Boulevard
Riviera Road
Maritime Avenue
Seascape View
Seashore Avenue
Lurline Boulevard
Shoreline Avenue
Milford Avenue
Plaza Avenue
Lascala Avenue
Fontaine Drive
Monaco Boulevard
Casino Boulevard
Cat 1
Cat 2
Cat 3
Cat 4
Cat 5
Cat 6
Cat 7
Sellicks Beach Road
Herrick Street
Francis Street
Herrick Street
Alexander Street
Arcadia Crescent
Orienta Drive
Virgo Parade
Robert Street
Gisborne Avenue
Dunedin Avenue
Wellington Avenue
Hastings Avenue
Napier Avenue
Ranfurly Avenue
Country Road
Gulf View Road
Main South Road
BASINS LOCATED ONLINE TO UPSTREAM WATERCOURSES

Legend

- Basin outlet pipe
- Watercourse
- Basin footprint

Discharge region

- Region 1
- Region 2
- Region 3

0 200 400 600 m

N

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SELLICKS BEACH STRUCTURE PLAN
STORMWATER MANAGEMENT ASSESSMENT
INDICATIVE COMBINED BASIN FOOTPRINTS

Figure 3.3

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Table 3.5 Combined basin properties

Catchment	Storage volume (m^3)	Estimated footprint (m^2)	Storage depth (m)	Outlet pipe size (mm)	Discharge rate (pipe) (m^3/s)	Spillway capacity (m^3/s)
Cat 2-3	18,460	14,200	2	375	0.41	14.0
Cat 4-7	12,080	9,800	2	375	0.41	9.0

3.3 Retention of runoff from frequent events

The detention basins described in Section 3.2 will mitigate the flood risk from rare (i.e. 1% AEP) storm events. For more frequent events, during which the existing catchment would be unlikely to generate any runoff, the post-development flow rate will exceed the pre-development flow rate, even following detention of flows within a basin.

To offset the increased peak flows during frequent rainfall events, it is recommended that a retention component (i.e. no outlet) be incorporated within the base of each detention basin. The basins should be unlined, allowing runoff to infiltrate into the soil below. Given the sandy soil type within the area (Geoscience Australia, 2012), it is expected that infiltration rates will be high, and hence runoff will drain quite quickly. Additional losses will also occur due to evaporation.

Provision of an infiltration device sized to accept the first 15 mm of rainfall will reduce peak flows from frequent storm events, hence reducing the frequency of discharge to the marine environment. This is consistent with recommendations from the Department for Water (2012). Based on a connected paved area of 50% (700 m^2 allotments), the additional retention volumes required to achieve this retention target are summarised in Table 3.6. These volumes are typically in the order of 20-30% of the detention volumes listen in Table 3.4. As such, incorporation of retention will likely result in the basin footprints increasing. Once the volume of the retention component is exceeded, flows will continue to discharge through the detention outlet pipe. Stated volumes would increase by approximately 30-40% if 450 m^2 allotments are used in the modelling.

Table 3.6 Additional storage volume required to retain 15 mm of rainfall

Catchment	Retention volume (m^3)
Cat 1	2,320
Cat 2	1,960
Cat 3	2,230
Cat 4	650
Cat 5	250
Cat 6	770
Cat 7	1,070
Combined Cat 2-3	4,190
Combined Cat 4-7	2,740

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3.3.1 Reduction in infiltration potential

Studies show that groundwater recharge rates in urban areas still remain poorly understood and largely under-researched, and knowledge about these rates and their expected changes under increasing urbanisation is therefore of primary importance. In cases where groundwater is shallow and soils are highly permeable, research shows that there is a strong positive correlation between groundwater recharge rates and the extent of the urban area. Based on studies and on water budget calculations within these conditions, the transformation of natural landscapes into impervious areas leads to an increase in groundwater recharge rates due to the reduction of evapotranspiration that more than compensates for the increase in runoff. Furthermore, water main leakages contribute to an increase in recharge rates. Overall, studies tend to demonstrate that a better understanding of groundwater recharge changes in urban areas is required to move towards sustainable water management.

The geology of the investigation area consists of Pleistocene age alluvial fan sediments derived from the erosion of the adjacent hills. These sediments consisting of sand or gravel plains of quartz sand sheets with local clay, calcrete, laterite, silcrete, silt are likely to be relatively porous in nature given the presence of sands, gravels and silts. These alluvial sediments are in turn underlain at depth by the older fractured rocks of the Normanville Formation.

Regional groundwater onsite however appears to be deep and lies within the underlying fractured rock of the Normanville Formation. Standing water level readings across the development area range from 30-48 m below ground level, although it is possible that shallower discontinuous (perched) groundwater occurrence may exist within the shallower alluvial sediments. Groundwater recharge of the aquifer beneath the site predominantly occurs to the east within the hills, where the fractured rock outcrops at the surface and is exposed to meteoric waters. Given the depth of groundwater beneath the site it is unlikely that significant recharge would occur within the development area.

Based on this information, it is deemed by Tonkin that the reduction of surface water infiltration resultant from the proposed development would not have an adverse impact on the recharge of the underlying aquifer.

3.4 Water quality

Water quality modelling of the study area has not been undertaken, however a qualitative analysis is provided. The primary aim of any water quality improvement devices should be to ensure that the quality of runoff discharged from the site is not worsened following development. Water quality measures should be considered at the whole of catchment scale, as well as at the individual lot scale.

3.4.1 Detention/retention basins

The proposed basins at the downstream end of each sub-catchment will provide water quality benefits due to the slow velocities through them, which facilitate vegetative filtering and some settling of particulate pollutants. Incorporation of the retention component described in Section 3.3 will reduce the volume of runoff discharging from the site (particularly for frequent events), thereby providing further improvements to the water quality of surface runoff. The water quality benefit provided by them will be somewhat diluted if the basins are on-line and also have to treat flows from the upstream undeveloped catchments.

3.4.2 Gross pollutant traps

The installation of gross pollutant traps (GPTs) upstream of each basin should be considered in order to alleviate the total mass of pollutants discharging into waterways. This will reduce the residual load of gross pollutants that are discharged to the receiving waters. The maximum removal of gross pollutants will be dependent on the selected GPT and maximum treatable flow rate.

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3.4.3 Raingardens

Raingardens are typically shallow, planted depressions that can provide water quality improvement benefits via biofiltration mechanisms. Raingardens can be implemented at a range of scales from individual residential blocks up to the treatment of whole of catchment flows. Raingardens can reduce the quantity of sediment and nutrients exported to receiving waters.

Raingardens should be considered within the study area in order to provide improved stormwater quality. Typically constructed within verges or roads, streetscape raingardens receive gutter flows via gaps in the kerbing. Flows are then allowed to pond and infiltrate. A high level overflow/outlet may be provided to discharge flows exceeding the storage capacity of the raingarden into the underground drainage network.

3.5 Water reuse

3.5.1 Managed aquifer recharge

Consideration has been given to the benefits of incorporating a managed aquifer recharge (MAR) stormwater harvesting scheme. Council currently operates a number of MAR schemes as part of the Waterproofing the South initiative, seeking to offset the mains water demand with harvested stormwater. The Hart Road MAR scheme is located approximately 4.5 km north of the study area, and receives runoff from the surrounding urban catchment. As part of the Hart Road scheme, treatment of stormwater runoff within a wetland is undertaken prior to injection into the underlying Port Willunga Formation aquifer. A similar approach could be undertaken within the Sellicks Beach study area.

Stormwater harvesting schemes rely upon a supply of catchment flows. The sub-catchments within the study area are relatively small in terms of harvesting potential (maximum sub-catchment area of 31 ha). A more reliable supply of catchment flows would be provided if the runoff is collected from the entire study area (135 ha) and the external upstream catchments (approximately 540 ha). Given that there are three primary discharge locations, consideration would need to be made as to whether treatment of runoff would occur at each of these locations, or if flows are directed to a single, centralised wetland for treatment. As development of the area would be occurring within greenfield land, a localised recycled water distribution network could be relatively easily accommodated. Alternatively, the area could be connected to existing schemes further to the north, such as the Hart Road scheme.

Further investigations to assess the harvestable volume of runoff from the study area should be undertaken to determine whether construction of a MAR scheme is economically viable. The outcomes of this study could require an increase in the recommended size of basins to allow for adequate water quality treatment. Alternatively, the treatment system could be via a more linear system along the watercourses.

3.5.2 Rainwater tanks

Rainwater tanks can be used to encourage the on-site reuse of stormwater runoff. Council should encourage the installation of rainwater tanks which, at a minimum, are plumbed into the hot water service and toilet to supplement mains water usage. The volumes of reuse achieved will be dependent on the following factors:

- the area of roof plumbed into the rainwater tank
- the size of the tank
- the daily water demands for rain water.

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3.5.3 Infiltration systems

A range of passive infiltration systems will facilitate water recharge into the groundwater system close to the location where runoff was first generated. The systems described below can be used to promote infiltration (which in turn also provides improvements to water quality). Given the sandy soils in the area these measures are likely to be quite effective.

Raingardens

Raingardens can allow for soakage of runoff that is diverted into the raingarden.

Permeable paving

Permeable paving, also known as porous paving, is a load bearing pavement structure which can be used on trafficable surfaces including roads and driveways with low traffic volumes, carparks and pedestrian areas. It is best suited to areas that are relatively flat (DPLG, 2010).

Permeable paving typically comprises a permeable surface layer overlying an aggregate storage layer and provides many runoff management benefits including:

- Reduction in peak discharges and volumes.
- Increased groundwater recharge.
- Water quality improvement as a result of infiltration.

Council should consider permeable paving in lieu of other footpath pavement options across the study area.

Tree pits

Tree pits typically involve the construction of an opening in the kerb to divert low gutter flows into infiltration pits behind the kerb. The primary objective of the pits is to provide passive irrigation for street trees, with associated amenity and cooling benefits. However, the pits also provide a reduction in stormwater volumes and pollutant loads discharged to receiving environments.

On-site soakage systems

Allotment-scale infiltration systems could be an effective strategy in managing runoff from development in the area, given the sandy soils in the area, which will aid infiltration. These are typically in the form of soakage trenches.

3.6 Watercourse management

Council has identified that Sellicks Creek (Creek 1) is heavily eroded between Country Road and Sellicks Beach Road. So as not to exacerbate existing erosion issues, the outlets from Catchment 2 and Catchment 3 should be located as far downstream as possible. These outlets should be the only points of discharge from the respective catchments to the creek. Appropriate scour protection should be provided at these (and all other) basin outlets to reduce the velocity of flows to non-erosive rates. These measures will assist in minimising the risk of worsening the erosion within the creek.

Council should consider undertaking remediation works to reinstate the bank slopes of Sellicks Creek to match the original profile, and installing structures, such as gabions or reno mattress, at key areas to protect the banks from erosive flows originating upstream of the study area. Council has previously undertaken rehabilitation works of the section of creek downstream of Sellicks Beach Road, including construction of linear ponds for detention and retention. A similar strategy could be investigated for the upstream segment. Given the depth of watercourse, these works could require a relatively wide strip of land, depending on the types of treatments used.

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As described in Section 2.3, Creek 2 and Creek 3 terminate at a location approximately 230 m south of Virgo Parade. Review of the floodplain mapping (Figure 2.2) confirms that downstream of this location, runoff does not follow the alignment displayed by the watercourse layer in Figure 3.2, but rather sheets across the surface towards the north through an area likely marked for development. It is recommended that formalisation of these watercourses be undertaken such that runoff is directed towards the north-west, where it will merge with Creek 4, rather than sheeting to the north. Alternatively, the flows could be piped through development in the area. However, an overland surface flood flow path would still be recommended for flows in excess of the pipe capacity. Similarly, Creek 4 is not well defined in the vicinity of Gulf View Road. This section of the watercourse should also be formalised or piped.

Stabilisation works will also be required for each of these three creeks.

3.7 External flow management

Review of the contours upstream of the study area indicates that runoff from catchments to the south-east currently passes through the study area. Where not conveyed within a defined watercourse, these flows are likely to be in the form of shallow sheet flows across the surface. Development of the area will result in changes to the flow paths of these external flows. Consideration of how external flows will be managed is therefore required.

A bund located just upstream of the study area could be used to intercept the flows and direct them to a watercourse, allowing them to be conveyed through the site along a defined flow path. Alternatively, runoff could be collected via pits and conveyed within an underground drainage network, discharging to one of the three outlet locations.

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4 Summary

A stormwater management assessment of the Sellicks Beach future urban growth area has been undertaken. A summary of the key outcomes of the assessment is provided below:

- Development should not occur within the extents of the existing major flood flow paths.
- The watercourses downstream of the proposed development area are likely at capacity, and would be unable to accommodate un-detained flows from the development.
- Regional scale detention basins are proposed to limit the 1% AEP peak flows from the study area to pre-development conditions. Options have been shown for both off-line and on-line basin options. Council has a preference for the on-line option as it reduces the total number of basins required, even though they will need to mange with flows from the undeveloped upstream portion of the catchment.
- The basin within the northernmost catchment will discharge to the existing pipe network within Justs Road. The remaining basins will discharge directly into the adjacent watercourses through short sections of pipe. No additional piped drainage outfalls will be required.
- The location of the basin outlets will be such that the outflows will not cause additional impacts to the eroded gully south of Sellicks Beach Road.
- An additional retention volume should be incorporated within the basins to allow runoff from frequent events to infiltrate.
- The basins will provide water quality improvement through infiltration, vegetative filtering and settling of pollutants.
- GPTs and raingardens should also be considered within the study area to provide further improvements to water quality.
- Rainwater tanks and infiltration systems should be incorporated to facilitate water reuse. A MAR scheme has also been considered, but further investigations would be required to determine whether this would be economically viable.
- Formalisation of the watercourse to the south of Virgo Parade is required to ensure that runoff does not spill across the developable area.
- Shallow bunds or inlet pits will be required at some locations to intercept external sheet flows that drain towards the upstream boundary of the proposed areas of future development.

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5 References

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Appendix A – 1% AEP flood inundation map

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Disclaimer

This map has been prepared to a standard of accuracy sufficient for broad scale flood risk management and planning. The flood extents are not based on actual historical floods. The map does not increase the risk or affect the level of flooding over an area or property. The limit of flooding shown on this map is not a boundary between flood prone and flood free land. Land outside the flood extent shown on this map could be affected by:

- Floods with a different Annual Exceedence Probability (AEP).
- Blockage in drainage systems, creeks or culverts caused by vegetation or other debris carried by floodwaters.
- Further development, earthworks and other changes to the catchment that alter the actual flood extents.

The flood extents shown are a prediction of land subject to a specific level of flood risk and do not necessarily indicate a threat to buildings located on that land. Confidence in the prediction is reduced in areas affected by flood depths less than 0.1 m, due to the effects of fences, walls, buildings and landscaping which affect the flow of floodwaters. Such effects, which require detailed modelling, are beyond the capabilities of the modelling process. Flood assessment for particular sites will require more detailed interpretation, survey and analysis by qualified and experienced persons.

This map is provided on the basis that those responsible for its preparation and publication do not accept any responsibility for any loss or damage alleged to be suffered by anyone as a result of the publication of the map, and the notations on it, or as a result of the use or misuse of the information provided herein.

More detail can be found in the report associated with this study.

Clark Court
Dundee Street
Dee Street
Armstrong Street
Stirling Avenue
Perth Street
Don Street
Greenock Street
Lanark Street
George Parade
Pier Street
Bay View
Norman Victory Parade
Dunrossil Avenue
Firth Place
Gordon Place
Aberdeen Street
Forth Street
Sandano Avenue
Old Sellicks Hill Road
Seven Hills Heights
Jetty Road
Cannes Drive
Tangier Boulevard
Riviera Road
Lucerno Crescent
Samarra Drive
Naples Place
Fontaine Drive
Monaco Boulevard
Lasseta Avenue
Capri Crescent
Piazza Avenue
Duncan Avenue
Casino Boulevard
Martinos Avenue
Seascape View
Seashore Avenue
Lurline Boulevard
Shoreline Avenue
Milford Avenue
Sellicks Beach Road
Bluewater Boulevard
Hendrick Street
Francis Street
Hanck Street
Alexander Street
Arcadia Crescent
Arion Way
Ortana Drive
Caronia Cove
Amore Terrace
Crystal Harmony Court
Silver Cloud Grove
Virgo Parade
Robert Street
Gisborne Avenue
Dunedin Avenue
Country Road
Main South Road
Gulf View Road
Neasby Avenue
Napier Avenue
Napier Place
Penrhyn Avenue
Wellington Avenue
Hastings Avenue
Nelson Avenue
Cactus Canyon Road

Legend
- Model boundary
- Study area

Depth of inundation
- Less than 0.05m (not shown)
- 0.05m to 0.1m
- 0.1m to 0.2m
- 0.2m to 0.3m
- 0.3m to 0.4m
- 0.4m to 0.5m
- 0.5m to 0.6m
- 0.6m to 0.7m
- 0.7m to 0.8m
- 0.8m to 0.9m
- 0.9m to 1.0m
- 1.0m to 10.0m
- 10.0m to 100.0m

tonkin

Job Number: 201217
Filename: 201217GQ001RevA
Revision: A
Date: 2020-07-28
Drawn: MM

0 100 200 300 m

Data Acknowledgement:
Aerial imagery from MetroMap, 2017
Roads from Data SA, 2018

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CITY OF ONKAPARINGA

SELLICKS BEACH STRUCTURE PLAN
STORMWATER MANAGEMENT ASSESSMENT
1% AEP FLOOD INUNDATION MAP
EXISTING SCENARIO

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