GLENPANEL APPENDICES

MASTERPLAN
INFRASTRUCTURE REVIEW
GEOTECHINCAL REVIEW
TRANSPORTATION REVIEW

Stormwater Infrastructure Assessment

Glenpanel – Special Housing Area

Prepared for

Maryhill Limited

Prepared by

Lowe Environmental Impact

August 2016

Stormwater Infrastructure Assessment

Maryhill Limited

This report has been prepared for the Maryhill Limited by Lowe Environmental Impact (LEI). No liability is accepted by this company or any employee or sub-consultant of this company with respect to its use by any other parties.

Quality Assurance Statement		
Task	Responsibility	Signature
Project Manager:	Rob Potts	
Prepared by:	Cameron Mars	
Reviewed by:	Rob Potts	
Approved for Issue by:	Rob Potts	
Status:	Final	

Prepared by:

Lowe Environmental Impact
P O Box 29288
Christchurch 8440

| T | [+64] 3 359 3059
| E | office@lei.co.nz
| W| www.lei.co.nz

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1	09/08/2016	CM	Initial draft for client comment
2	10/08/2016	CM/RP	Added comments on Council Report - Final

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EXECUTIVE SUMMARY

Lowe Environmental Impact (LEI) has carried stormwater modelling (EPA SWMM) to estimate pre and post development stormwater run-off flow rates/volumes, discharged from the proposed Glenpanel Special Housing Area (SHA). LEI has also modelled the upper hill catchment to estimate the cut-off drain and additional detention requirements, thereby allowing the hillside run-off to be diverted away from the lower platform SHA development area.

Coinciding with the LEI modelling, GeoSolve Limited (GeoSolve) carried out an in-depth soils investigation across the proposed SHA, to define the underlying soil types and potential infiltration and deeper soakage characteristics.

The soils across the SHA were found to be a mixture of silty sands & gravels, and gravelly sands/sandy gravels, down to the inspection depth of 8 m. Overall, sand and gravels were the predominant soil across the SHA; indicating a well-drained environment.

Infiltration testing was carried out within the top soil horizon and within the lower horizon gravels and sands. The top soil infiltration was typical of a silt and on average 12 mm/hr, whilst the lower soils indicate a potential of greater than 1 m/hr, based on raw measured data. However, if calculated based on allowing for lateral seepage from the test bore, could be as low as 200 mm/hr. Overall, it is considered that the underlying soils are suitable for onsite stormwater soakage/disposal applications.

Based on the modelling output and the geotechnical investigation, the following infrastructure options for the reticulation, detention and discharge of the SHA stormwater are considered feasible:

1. Full onsite detention and discharge:
   - Roof run-off to discharge via individual onsite soakage pits;
   - Roading run-off discharging to a series of grassed swales, small basins or raingardens, which in-turn discharge to soakage pits; and
   - Secondary flow beyond the 20-year design infrastructure capacity can be directed to a shallow basin (via overland flow), located within the southern green space area. All 100-year secondary flow can be detained onsite and infiltrated via the underlying soils.

2. Partial onsite detention and discharge:
   - A combination of onsite detention and soakage pit discharges, with an allowance for an offsite discharge to either the Ladies Mile open channel roadside drain or a piped connection to the Queenstown Country Club infrastructure;
   - Onsite detention and disposal infrastructure could cater for some of the design 20-year rainfall run-off and likewise for the 100-year secondary flow. Additional flows beyond the onsite disposal capacity would be discharged off site; and
   - The detention requirement would need to be such to ensure that all offsite discharges were to an acceptably low flow rate to be accommodated by the external infrastructure.

3. Offsite discharge of all Stormwater:
   - All stormwater runoff would be discharged offsite to either the Ladies Mile open drain or to the Queenstown Country Club infrastructure; and
   - Greater volume of onsite detention would be required to ensure the discharge was to an acceptably low level.

The internal reticulation for all three options will primarily be dependent on the civil detailed design requirements. However, at the conceptual design level, the site is suitable for both piped or surface reticulation (e.g. kerb and channels with subsurface pipes or swales, etc.). At this early stage, the preference is for surface flow via a series of kerb and channels feeding into swales or potentially small isolated basins or raingardens (the latter are applicable if onsite soakage is preferred). The site gradient is suitable for infrastructure of this nature and a combination of swales and roading infrastructure can convey secondary flow to a potential detention basin located within the southern green space area of the SHA. The advantage with swales over piped reticulation is reduced capital expenditure, enhanced treatment outcomes, reduced flow velocities, additional onsite storage capacity, as well as being a visually pleasing amenity that will fit in well with the overall site aesthetics.

The upgradient hillside catchment run-off can be diverted around the SHA or down through the development. All hillside infrastructure will have capacity for the 100-year critical duration rainfall event. The following options are considered feasible:

1. Run-off diversion and full onsite detention and discharge:
    - Cut-off drain directing all hillside runoff to an infiltration basin (located in an area of land to the north east of the development boundary);
    - All run-off would be detained and discharged via the basin underlying soils.

2. Run-off diversion and partial onsite detention and discharge:
    - Cut-off drain directing all hillside runoff to an infiltration basin as per Option 1;
    - The basin would partially detain and infiltrate run-off, with run-off volumes beyond the basin capacity would be discharged off site;
    - As per the SHA options, the offsite discharge can be via the roadside open channel or potentially to the Country Club reticulation via a drain down the SHA eastern boundary; and
    - Potentially, the hillside basin (if capacity is limited due to available land area) could discharge to the detention/infiltration basin located within the SHA.

3. No diversion and either full or partial onsite detention:
    - Run-off from the hillside gullies could be channelled through the development area via landscaped drains that would discharge into an onsite detention basin; and
    - The discharge mechanism would be via infiltration/soakage or off site via the roadside open drain or Country Club.

Based on the modelling and known site soils characteristics, Option (1), full onsite detention and discharge, for both the SHA and hillside infrastructure is recommended because the site soils are well drained and appear suitable for onsite soakage. However, all options are feasible.

The overall impervious area increase resulting from the SHA is not large and does not result in a significant increase in the post-development run-off rates or volumes. Furthermore, there is sufficient available land area to site one or more infiltration basins.

Should it be deemed during the detailed design phase, that there is not adequate land area to the north east of the SHA to site an infiltration basin with acceptable capacity for the capture of the hillside runoff, a smaller basin can be used, for smaller return period rainfalls, and runoff beyond the basin capacity can be diverted to the SHA system.

In summary, there are numerous viable options for stormwater management for both upgradient hill runoff and the development runoff. These options make use of onsite attenuation and soakage to varying degrees.

1 INTRODUCTION

Lowe Environmental Impact (LEI) has been engaged to assess the stormwater drainage and detention/attenuation options for a proposed Glenpanel Special Housing Area (SHA) development, located on the northern side of Ladies Mile. The site is legally described as Lots 2, 4 and 7 DP 463532 and Sections 42 – 44 Block III Shotover Survey District.

Figure 1.1 provides the site location Plan.

Figure 1.1: Glenpanel SHA Location Plan

The overall area comprises of two principle catchments; the hillside catchment to the north and a flatter platform at the base of the hill to the south (in which the SHA will be sited). It is proposed that the hillside catchment run-off be diverted away from the lower development catchment (via a cut off drain discharging to a detention and/or infiltration basin) and therefore each catchment has been modelled separately.

The hillside and the development catchments have been modelled using EPA SWMM (version 5.1), to define the 20 year and 100 year pre and post development run-off flow/volumes. Coinciding with the modelling, GeoSolve carried out a site investigation (8^{th} & 9^{th} August 2016) to define the underlying soil profiles and infiltration and soakage characteristics.

The modelling and soils investigation have been used to define practical and viable options for the treatment, detention and discharge of stormwater from the SHA.

This report outlines the site characteristics, the stormwater modelling results and provides stormwater infrastructure options.

Appendix A contains the proposed SHA masterplan.
Appendix B contains the EPA SWMM modelling data.
Appendix C contains the results of the GeoSolve Investigation.

2 MODELLING PARAMETERS

All modelling input data is located within Appendix B.

2.1 Area of Investigation

The modelling investigation encompasses the following catchments:

1. Hillside catchment (44.8 ha), directly to the north of the development area.
2. Glenpanel development SHA (20.0 ha), plus an additional area (3.3 ha) of pasture/scrub land just to the north of the development that is not likely to feed into the hillside cut off drain.

The upper (hillside) and lower (SHA) catchment zones have been modelled independently from one another. It is proposed that the hillside catchment run-off will be diverted away from the lower lying development area (as generally occurs currently). Therefore, up to and including the 100-year critical duration, there should be no interaction between the SHA and upper Hill stormwater catchment areas.

The hillside has two gullies sited directly above the SHA and currently run-off is captured by a series of cut off drains discharging to one or more small irrigation ponds. Some sections of the existing cut off drains are not well formed and there is the potential for run-off to percolate or flow down into the lower lying platform. Therefore, as part of the development works, it is considered that the cut off drains will need upgrading. Pending the outcome of the stormwater modelling and geotechnical investigation (discussed in later sections), it is proposed that the cut off drain discharge to a new detention/soakage basin, located to the north east of the SHA.

The majority of the hillside run-off appears to undergo infiltration (during moderate to average rainfall), prior to the irrigation ponds and historically there has been no significant ponding issues within the proposed SHA boundary; thereby suggesting well drained soils. If and when run-off does enter the lower lying areas, it is captured via natural depression storage and infiltrated. This was evident in the site walkover, as there is little in the way of formed channels, indiucating very low run-off rates and low erosion potential soils.

The land formation at the base of the hillside, stretching between the SHA and Lake Hayes, has a naturally low lying depression with a gradient towards the lake. LiDAR indicates that this would be the flow path of secondary flow during an extreme rainfall event e.g. 1 in 1,000 year return period.

The hillside and development catchments, and the secondary flow path are shown in Figure 2.1.

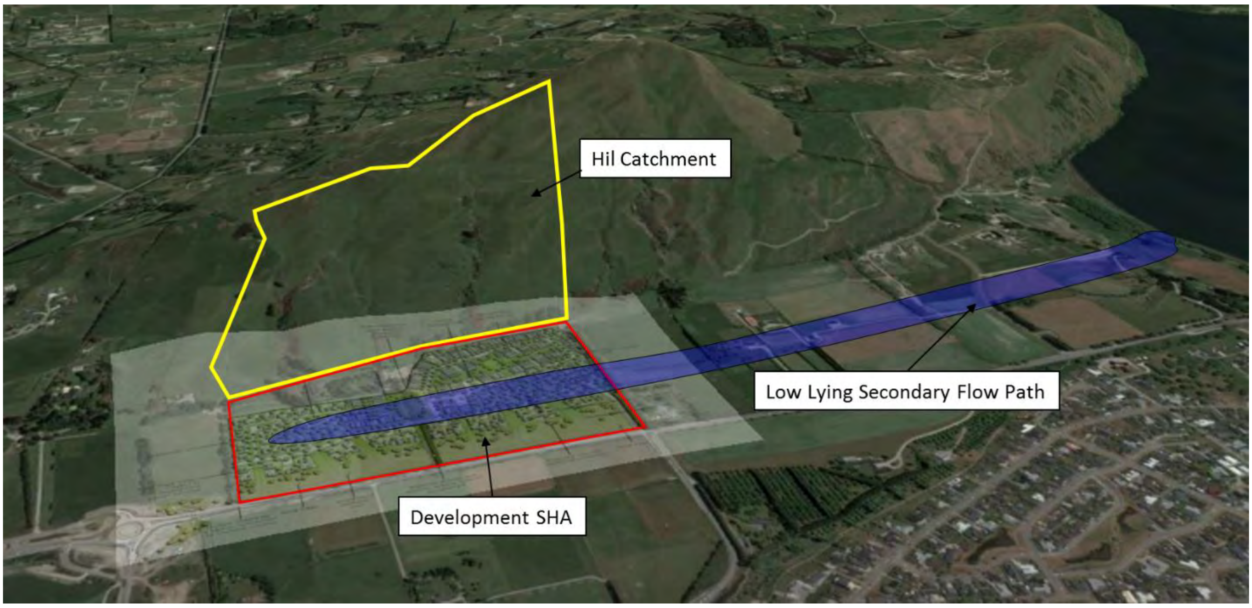


Figure 2.1: Hillside Catchment, SHA Catchment and Existing Secondary Flow Path

The proposed development is around 20 ha, however, there is approximately an additional 3.3 ha of land area (primarily pasture/scrub) between the existing hillside cut off drains and the north western development boundary. This additional area is expected to be downgradient of any future hillside cut-off drain and has a moderate sloping topography towards the development area. Therefore, this area has been included within the development modelling.

2.2 Site Soils

Landcare research S-maps describe the SHA soils as being a moderately deep well drained Barrhill silty loam. The hillside catchment soils are described as being a well drained ArrowBlack loam.

GeoSolve carried out a site investigation of the 8th and 9th August 2016 to define the underlying soil profiles/horizons and infiltration/soakage characteristics. Figure 2.2 shows the testing locations.

At each of the five testing locations, GeoSolve drilled down to a depth of around 8 m and logged the various soil and subsoil horizons. At testing sites 3, 4 and 5, infiltration testing (using rings) was carried out, within the 300 mm topsoil layer. A falling head soakage test was carried out at the 3 m depth and 8 m depth within test site 3. The 3 m depth test was within the lower range permeability silty SAND and the 8 m test depth was within the higher permeability GRAVEL/gravelly SAND.

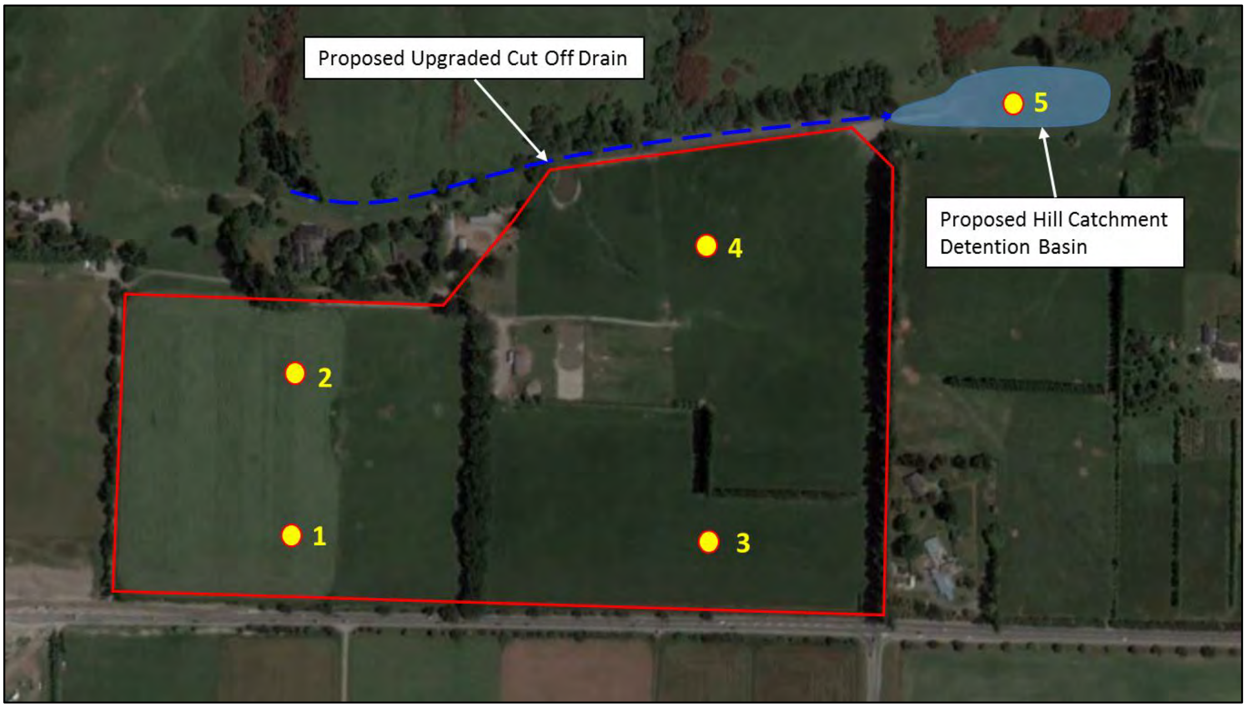


Figure 2.2: Soils Testing Locations

Appendix C contains the soil profiles/horizons and the measured soils infiltration characteristics. within each of the five testing sites shown in Figure 2.2.

Overall, the topsoil infiltration rate is in the range of 5 – 25 mm/hr and has an average infiltration rate of approximately 12 mm/hr. The lower profile sandy gravels and gravelly sands, measured at depths of 3 m and 8 m below ground level (blg), are expected to have higher soakage rates, potentially above 1 m/hr. However, when calculated based on allowing for lateral seepage from the bore, these could be as low as 200 mm/hr (average).

Hortons equation has been used as the infiltration method for the EPA SWMM modelling simulation. The following infiltration rates were used for both the hillside and SHA modelling.

- Maximum infiltration rate = 15 mm/hr
- Minimum infiltration rate = 5 mm/hr
- Decay constant = 1/9 hours
- Drying time = 7 days

It is considered that the modelled maximum and minimum infiltration rates are within the measured range and are considered conservative. The decay constant is an unknown therefore 1/9 hours is based on WWDG (2011) recommendations. A drying time of 7 days is an assumption but has no effect on the modelling outcome. The hillside infiltration rate was not measured, however, the area is mapped as being free draining and due to the lack of drains in the area (indicating low runoff), it has been assumed that the hillside will have similar infiltration characteristics to that of the SHA. We consider this an acceptable assumption for this stage of the investigation, particularly as the modelling results show significant factor of safety regarding areas available for attenuation and soakage should the assumption be less than conservative.

2.3 Design Rainfall

2.3.1 Climate Change

Projection of future climate change is relatively uncertain with many dependencies. However, Otago is expected to become wetter, particularly in winter and spring and rainfall will vary locally within the region. In Queenstown, average annual rainfall is likely to increase by 12% by 2090. Seasonal projections show a 2.2 °C temperature rise in winter and winter rainfall increasing by 29% in Queenstown by 2090. However, there is likely to be little change in summer and autumn rainfall by 2090 (Ministry for the Environment, 2008).

Summer and autumn temperatures are likely to increase by 2.0 °C by 2090, whilst little change is expected in rainfall, evapotranspiration rates will increase.

2.3.2 High Intensity Rainfall Data

Queenstown Lakes District Council (QLDC) Land Development and Subdivision Code of Practice (2015) requires stormwater drainage infrastructure to have capacity for the following return periods:

- Primary reticulation: 1 in 20 year return period (5% AEP); and
- Secondary flow: 1 in 100 year return period (1% AEP).

NIWA's High Intensity Rainfall Data System (HIRDS) Version 3 has been used to estimate the 5% and 1% AEP rainfalls. The HIRDS rainfall is based on an extreme rainfall assessment with climate change, assuming a 2.2 °C temperature rise in winter.

Appendix B provides the HIRDS data used for the modelling simulations.

2.3.3 Rainfall Distribution

Rainfall hyetographs refer to the time distribution of rainfall intensities (or depths) over a water shed and this methodology has been utilised within the EPA SWMM modelling to estimate the pre and post development run-off and the volume of detention required for the Glenpanel SHA development area. Most rainfall events have a time varying pattern, for example, both the Christchurch (CCC) and Dunedin City Council's recommend hyetographs with the peak rainfall twice that of the average, occurring at time 0.7 (70%) of the storm duration. There is no prescribed hyetograph methodology for Queenstown, therefore the modelling simulation hyetographs are based on the recommended methodology stated in the CCC Waterways, Wetlands and Drainage Guide (WWDG, 2011). Appendix B provides the 1 in 100 year rainfall hyetographs used for the SWMM modelling.

2.3.4 Critical Duration Storm

The QLDC Codes of Practice do not specify a design critical duration rainfall, therefore, the critical duration has been assessed based on WWDG (2011) recommendations. It is considered that for the size and scale of both catchments, the critical duration is equivalent to the time of concentration.

The critical durations used for the modelling scenarios, are as follows:

- Hill side catchment = 90 minutes
- Development SHA catchment = 3 hours

Please refer to Appendix B for the critical duration calculations.

2.4 Development Pervious & Impervious Areas

The masterplan for the development (refer to Appendix A) proposes a mix of low, medium and high density residential dwellings over the site. Table 2.1 provides the measured areal coverage of roofing, roading and green space areas.

Table 2.1: Development Impervious and pervious Catchments

Parameter	Impervious (ha)	Pervious (ha)	Percentage of Total Area
Roofing	3.116		
Roading	2.076		
Driveways/miscellaneous impervious	3.0		
Development Green Spaces		11.808	
Other pervious		3.3[(1)]	
Total Impervious	8.2		35%
Total Pervious		15.1	65%

[(1)] Pasture/scrub located to the north of the development which is not likely to be within the hill cut-off drain catchment.

The catchment areas provided in Table 2.1 have been measured directly from the masterplan, located in Appendix A. The average roof area per dwelling is between 120 m^2 to 170 m^2, with the size being dependent on the residential zone. An additional 3 ha was added to the impervious area to account for potential driveways and other miscellaneous impervious areas not necessarily shown on the masterplan; this also allowed for a level of conservatism to be incorporated into the modelling.

Overall, the impervious surfaces account for 35% of the total catchment modelled; however, this also includes the 3.3 ha area to the north of the SHA, which is unlikely to be within the hillside cut-off drain catchment. When considering the 20 ha SHA only, impervious surfaces account for 41% of the total SHA (exact figure will need to be confirmed during the detailed design phase).

3 MODELLING RESULTS

3.1 Hill Catchment

The Hillside Catchment was modelled against the 100 year 90-minute critical duration rainfall to define the runoff volume and potential peak flow rate. Simulations were also carried out for the 30 minute and 1 hour durations to further define potential high flow/low volume scenarios, for which the cut off drain must have adequate capacity.

Table 3.1 provides a summary of the modelling results.

Table 3.1: Hill Catchment Modelling Results

Return Period (year)	Duration (minutes)	Total Runoff Volume (m^3)	Average Flow Rate (m^3/s)	Peak Flow Rate (m^3/s)
100	30	3,512	0.2	3.0
100	60	7,573	0.4	4.5
100	90	10.5	0.6	4.6

The cut of drain was modelled as having the dimensions (roughly based on the existing drain), as follows:

- Bottom width = 1.5 m
- Batter = 1 (vertical): 2(horizontal)
- Maximum depth = 1 m
- Slope = approximately 2% – 3%

The maximum velocity down the full length of the cut off drain ranged between 1.75 m/s – 3.12 m/s with a maximum water depth range of 0.47 m – 0.56 m. Overall, it is considered that a cut-off drain of 1.5 m - 2.0 m in width, with a total depth of 1 m, would be sufficient for diversion of hillside flows during a 100 year return period. Some scour protection is likely to be required, i.e. a gravel invert but this is a detailed design consideration, as the gradient could be flattened.

3.2 SHA Catchment

Pre and post development modelling of the SHA catchment was carried out to define the required detention volume and potential flow rates. The 3-hour critical duration rainfall defines the total detention capacity required, however, modelling was also carried out across shorter duration storm events to ensure the worst case scenario was assessed.

Table 3.2 summarises the pre and post development modelling results.

Table 3.2: SHA Catchment Modelling Results

Return Period (year)	Duration (minutes)	Run Off Volume (m^3)		Peak Run Off Flow Rate (m^3/s)	
		Pre Development	Post Development	Pre Development	Post Development
20	10	Minimal	530	0.006	1.8
20	60	1,500	3,020	0.8	1.8
100	10	140	960	0.2	2.7
100	60	3,590	5,300	1.9	3.2
100	180	7,890	10,210	1.8	2.2

The overall detention capacity based on the 100 year 3-hour critical duration equates to 2,320 m^3, i.e. the difference between pre and post development. There is sufficient land area within the southern part of the SHA to easily accommodate the detention requirement.

4 INFRASTRUCTURE OPTIONS

Based on the modelling output and the geotechnical investigation, the following infrastructure options for the reticulation, detention and discharge of the SHA and hillside catchment stormwater are considered feasible.

4.1 SHA Catchment Infrastructure Options

Option 1: Full Onsite Detention and Disposal

Roof run-off can discharge via individual onsite soakage pits with capacity for the 20-year rainfall. Roading run-off can discharge via kerb and channel or directly to a series of swales, small basins or raingardens, which in turn would discharge to soakage pits. Secondary flow beyond the 20-year design infrastructure capacity can be directed to a shallow infiltration basin (via overland flow), located within the southern green space area. All 100-year secondary flow can be detained onsite and infiltrated via the underlying soils.

Option 2: Partial Offsite Discharge

A combination of onsite detention and soakage pit discharges, with an allowance for an offsite discharge to either the Ladies Mile open roadside drain or a piped connection to the Queenstown Country Club infrastructure. Onsite detention and disposal infrastructure could cater for all or some of the 20-year design storm and likewise for the 100-year secondary flow. Additional flows beyond the onsite capacity would be discharged offsite. The detention requirement would need to be such to ensure that all offsite discharges were to an acceptably low flow rate to be accommodated by the external infrastructure.

Option 3: Full Offsite Disposal

All stormwater run-off would be discharged off site to either the Ladies Mile open drain or to the Queenstown Country Club infrastructure. A significantly greater volume of onsite detention would be required to ensure the discharge was to an acceptably low level.

Drainage Reticulation

The internal reticulation for all three options will primarily be dependent on the civil detailed design requirements. However, at the conceptual design level, the site is suitable for both piped or surface reticulation (e.g. kerb and channels with subsurface pipes or swales etc.). At this stage, the preference is for surface flow via a series of kerb and channels feeding into swales or potentially small isolated basins or raingardens (the latter are applicable if onsite soakage is preferred). The site gradient is suitable for infrastructure of this nature and a combination of swales and roading infrastructure can convey secondary flow to an infiltration/detention basin located with the southern part of the SHA. The advantages of swales over piped reticulation is reduced capital expenditure, enhanced treatment outcomes, reduced flow velocities, additional onsite storage capacity and they can be a visually pleasing amenity that will fit in well with the overall site aesthetics.

4.2 Hillside Catchment Drainage Options

Hillside run-off can be either be diverted around the SHA or through the development. All hillside infrastructure will have capacity for the 100-year critical duration rainfall. The following options are considered feasible:

Option 1: Run-Off Diversion, Full Onsite Detention and Disposal

Cut-off drain directing all hillside runoff to an infiltration/soakage basin (located within an area of land to the north east of the development boundary). All run-off would be detained and discharged via the basin underlying soils.

Option 2: Run-Off Diversion, Partial Onsite Detention and Disposal

Cut-off drain directing all hillside runoff to an infiltration basin (as per Option 1). The basin would partially detain and infiltrate stormwater. Run-off volumes beyond the basin capacity would be discharged off site. As per the SHA options, the offsite discharge can be via the roadside open channel or to the Country Club reticulation. Another sub-option if the hillside basin capacity is limited due to available land area), would be discharge to the detention/infiltration basin located within the SHA. There is sufficient land for this.

Option 3: No Diversion and SHA Detention

Run-off from the hillside gullies could be channelled through the development area via landscaped drains that would discharge into an onsite detention basin. The discharge mechanism would be via infiltration/soakage or off site via the roadside open drain or Country Club.

4.3 Summary

Based on the modelling and known site soils characteristics, Option (1), full onsite detention and discharge, for both the SHA and hillside infrastructure appears to be the best option because the site soils are well drained and appear ideal for onsite soakage. The overall impervious area increase is not large and does not result in a significant increase in the pre development run-off volumes. Furthermore, there is sufficient available land area to locate the infiltration basins. Should at a later date, during the detailed design phase, it be deemed that there is not adequate land area to the north east of the SHA to site an infiltration basin of acceptable capacity for the capture of the hillside runoff, then a smaller basin can be constructed and runoff beyond the basin capacity can be diverted to the SHA system.

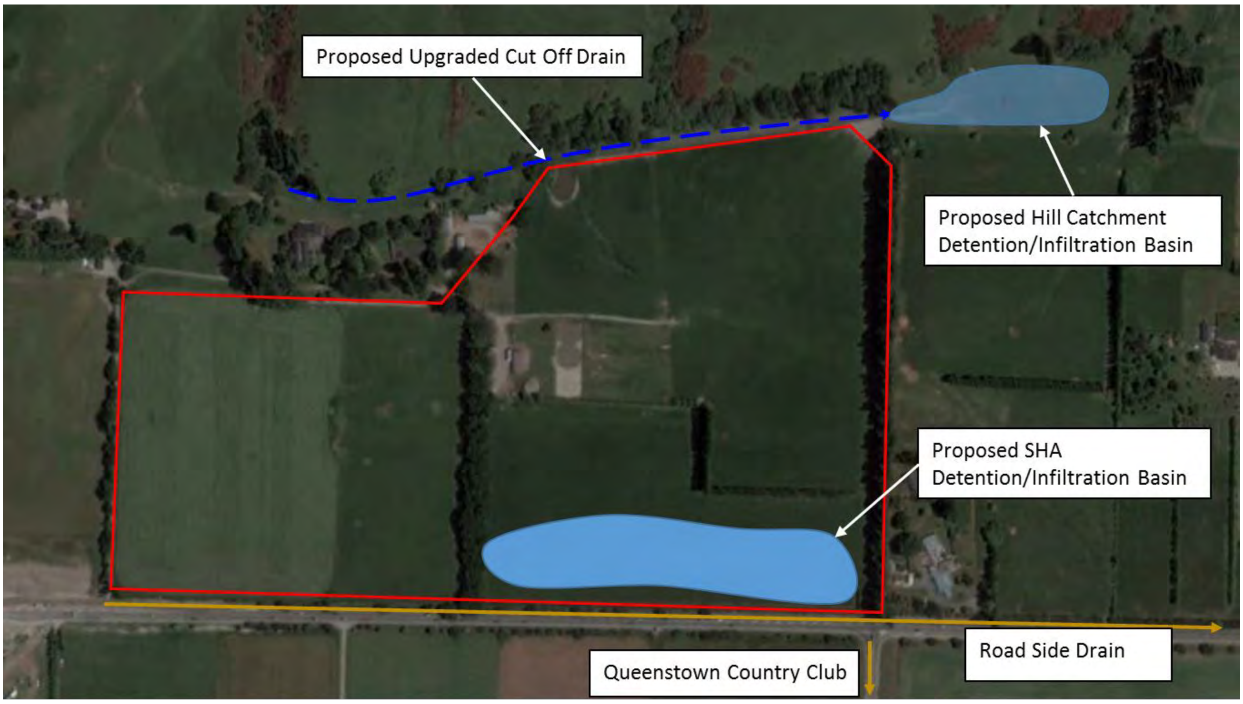


Figure 4.1: Proposed Stormwater Infrastructure

5 CONCLUSIONS AND COUNCIL REPORT

The Glenpanel SHA has several options for the detention and discharge of stormwater, ranging from full onsite disposal for both primary and secondary flows, partial disposal offsite or full disposal off site. Potentially, offsite disposal may require a larger onsite detention capacity, depending on the allowed discharge flow rate (this may be lower than the pre development rate) to the downstream infrastructure.

Modelling of the Glenpanel SHA and onsite soils testing indicate that the site has adequate soils infiltration capacity to allow for the use of soakage pits for roof and roading stormwater disposal, and an infiltration basin for the onsite discharge of secondary flow. Reticulation infrastructure can comprise of a mixture of kerb and channel or swales. It is considered that the preferred reticulation methodology should be to avoid piped reticutlion because the site layout and gradient allows for grassed swale flow, allowing for a level of treatment, reduced capital expenditure, improved aesthetics and avoiding the need for subsurface reticutlion.

Modelling of the upper hillside catchment indicates that a cut-off drain, similar in width to the existing, will have capacity to divert run-off during the 100-year critical duration return period. However, the existing cut-off drain arrangement will require upgrading and possibly scour protection.

The ideal scenario is for the cut-off drain to discharge via an infiltration/detention basin, located within an area of land to the north east of the SHA. The modelling and topographical maps indicate that the area of land is large enough to site a basin allowing full capture and detainment of secondary flow (with no external discharge off site). However, there is the option of discharge from the hillside detention basin to the SHA basin or to discharge directly off site via the roadside drain.

In summary, there are numerous viable options available for managing the onsite and offsite stormwater without putting any stress on infrastructure outside the SHA area.

5.1 Comments on Council Report

Paragraphs 46 – 57 cover stormwater infrastructure. Issues raised were identified by the Council reviewers – Holmes Consulting Group (HCG) – these were:

1. Non-conservatism in runoff calculations regarding impervious areas;
2. No downstream network being identified for discharge;
3. Breakout risk from the hillside cut-off drain;
4. No soakage tests having been carried out;
5. Lack of conservatism in the infiltration rates;
6. Offsite seepage effects and a Stormwater Management Plan; and
7. Lack of pretreatment to reduce sediment to protect infiltration basins.

We generally agree with the HCG concerns and we have covered all but three points in the report above.

Points' 1 and 2 are to do with runoff calculations and soakage assumptions. The runoff calculations provided in the Clarke Fortune MacDonald report used a simple but widely accepted formula that generally results in conservative runoff rates. However, it cannot be used to route flood events through basins to test mitigation methods, such as attenuation basins and soakage systems. It also uses simplified methods to predict infiltration and runoff. It is very widely used in Australia but in NZ, it is generally not used for catchments above 20 ha. The EPA SWMM model used removes all the short comings of the Rational Method described above.

Points 4 and 5 are to do with infiltration and soakage. These have now been measured by GeoSolve and the infiltration rates used in runoff modelling. The test pits and soakage tests show sandy gravels with significant hydraulic conductivity that is suitable for soaking the entire 100 yr event in suitably sized basins. These will be 300 – 400 mm open green spaces that will provide adequate sedimentation, landscaping, picnic areas, etc. with overflows via scruffy domes into deeper stone filled soakage systems.

Point 7 is to do with pretreatment. This has not been covered in this modelling report but would be covered in a later investigation as there are numerous methods that can be used to protect the integrity of the soakage system, such as; swales, small check dams in the swales if gradients are too steep, first flush forebays, raising the scruffy domes to allow settling in the infiltration basin, etc. The selected method could be a combination and there is sufficient room to fit them into the greenspace area.

Point 6 is to do with offsite effects such as seepage from terraces and springs due to runoff being put into ground soakage. This is possibly an issue for Otago Regional Council in consenting the discharge. However, based on the lack of drainage from assessing plans and the site walkover, it is very apparent that most predevelopment runoff currently soaks into ground. The proposal to continue this will not alter the status quo. Where runoff enters the ground may be in more concentrated areas that may temporarily mound groundwater but this unlikely to alter groundwater flow direction, spring locations or flows.

Point 3 is to do with the structural integrity of the cut-off drain. This is a detail for later investigation and most likely detailed design. Mentioned above is the possibility of requiring scour protection of the drain invert, such as gravel chip and possibly small rock check dams. Other methods are also available depending on the soils encountered, such as lime or cement stabilisation, lining the channel, providing flatter gradients and batters.

We therefore consider that the points raised in the Planning Officers Report that need to be addressed, have been addressed.

6 REFERENCES

Christchurch City Council (2011). *Waterways, Wetlands and Drainage Guide (WWDG).* Christchurch, New Zealand.

7 APPENDICES

Appendix A	Glenpanel Conceptual Layout Plan
Appendix B	EPA SWMM Modelling
Appendix C	Soils Characteristics Results

APPENDIX A

Glenpanel Concept Masterplan

APPENDIX B

EPA SWMM Modelling Parameters

Intensity + 2.2 ^{o}C Climate Change (mm/hr)

ARI (y)	AEP	10m	20m	30m	60m	90m	2h	3h	6h	12h	24h	48h	72h
1.58	0.633	21	16.5	14.2	10.8	9.3	7.8	7.00	4.6	3.3	2.3	1.4	1
2	0.5	23.4	18	15.2	11.7	10	8.3	7.45	4.9	3.5	2.5	1.5	1.1
5	0.2	30.6	23.1	20	15.4	13.1	10.8	9.65	6.2	4.4	3.1	1.9	1.4
10	0.1	36	27.9	23.8	18.4	15.6	12.8	11.45	7.4	5.2	3.6	2.2	1.6
20	0.05	42.6	32.7	28.2	21.8	18.5	15.2	13.55	8.6	6	4.2	2.5	1.9
30	0.033	46.8	36	31	24.1	20.45	16.8	14.98	9.5	6.6	4.6	2.7	2
40	0.025	49.2	38.4	33.2	25.8	21.85	17.9	15.93	10	7	4.8	2.9	2.1
50	0.02	52.2	40.5	35	27	22.9	18.8	16.73	10.5	7.3	5	3	2.2
60	0.017	54.6	42.3	36.4	28.2	23.85	19.5	17.35	10.9	7.5	5.2	3.1	2.3
80	0.012	58.8	45.3	39	30.2	25.5	20.8	18.48	11.5	7.9	5.5	3.3	2.4
100	0.01	61.2	47.7	41	31.9	26.9	21.9	19.43	12	8.3	5.7	3.4	2.5

CRITICAL DURATION ANALYSIS

Time of Concentration (ToC) = Critical Duration

Time of overland flow = $\frac{100nL^{0.33}}{S^{0.2}}$ = minutes

where:
n = Horton's value for surface roughness.
L = Length of overland flow in metres.
S = Slope as a percentage (e.g. 2%).

Table 21-2: Horton's n roughness values for overland flow.

Surface Type	n
Asphalt/concrete	0.010–0.012
Bare sand	0.010–0.060
Bare clay/loam	0.012–0.033
Gravelled surface	0.012–0.030
Short grass	0.100–0.200
Lawns	0.200–0.300
Pasture	0.300–0.400
Dense shrubbery	0.400

(WWDG, 2011)

Time of channel flow = Table 21-4 (WWDG, 2011)

There is insufficient data available to calculate the time of open channel flow and therefore Table 21-4 (WWDG, 2011) has been used.

Table 21-4: Approximate natural stream velocities.

Catchment Description	Grade	Velocity (m/s)
Flat	flat to 1 in 100	0.3 - 1.0
Moderate grade	1 in 100 to 1 in 20	0.6 - 2.0
Hillside	1 in 20 or steeper	1.5 - 3.0

HILLSIDE CRITICAL DURATION

Approx. hill slope (average) = 15%
Length of overland flow prior to gully entry = approx. 100 m
Horton's n = pasture = 0.3

Time of overland flow = 100 x 0.3 x $100^{0.33}/15^{0.2}$ = 80 minutes

Gully slope steeper than 1 in 20
Gully (channel) velocity = approx. 2 m/s
Gully length = 230 m

Time of gully channel flow = 2 minutes (note: longest flow path is being considered and therefore only western most gully is considered).

Cut off drain slope = moderate grade.
Cut off drain velocity = approx. 1 m/s – 1.5 m/s
Cut of drain length = approx. 500 m

Time of cut off drain flow = 8 minutes

Time of Concentration = Critical Duration = 80 + 2 + 8 = 90 minutes

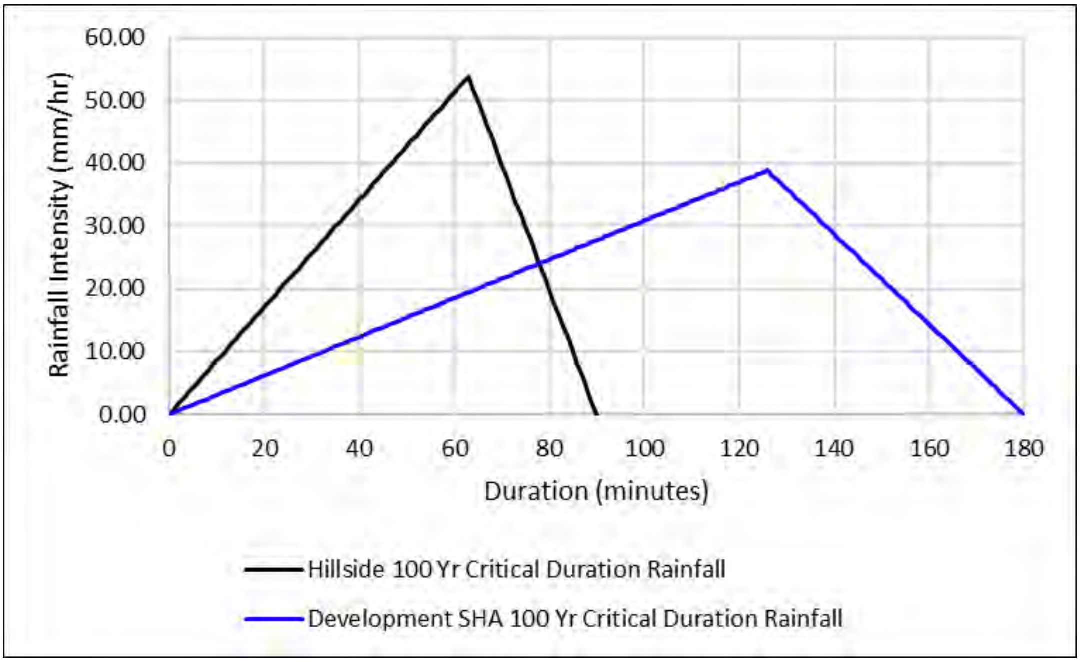
SHA DEVELOPMENT CRITICAL DURATION

Approx. development slope (average) = 2.5%
Length of overland flow prior to gully entry = approx. 620 m
Horton's n = pasture = 0.25

Time of overland flow = 100 x 0.25 x $620^{0.33}/2.5^{0.2}$ = 174 minutes = approx. 3 hours

RAINFALL HYETOGRAPHS

The chart below provides the rainfall Hyetographs used for the modelling of both hillside and development area critical duration rainfalls for the 1 in 100 return period.



HILLSIDE CATCHMENT MODELLING CHARACTERISTICS

Gully Sub Catchment 1

Property	Value
Name	H3
X-Coordinate	1356.375
Y-Coordinate	5258.065
Description	
Tag	
Rain Gage	100yrRainfall
Outlet	J1
Area	20.4
Width	2040
% Slope	15
% Imperv	0
N-Imperv	0.24
N-Perv	0.24
Dstore-Imperv	5.08
Dstore-Perv	5.08
%Zero-Imperv	0
Subarea Routing	OUTLET
Percent Routed	100
Infiltration	HORTON
Groundwater	NO
Snow Pack	
LID Controls	0
Land Uses	0
Initial Buildup	NONE
Curb Length	0

Gully Sub Catchment 2

Property	Value
Name	H4
X-Coordinate	4587.448
Y-Coordinate	5512.399
Description	...
Tag	
Rain Gage	100yrRainfall
Outlet	J2
Area	24.4
Width	2440
% Slope	20
% Imperv	0
N-Imperv	0.24
N-Perv	0.24
Dstore-Imperv	5.08
Dstore-Perv	5.08
%Zero-Imperv	0
Subarea Routing	OUTLET
Percent Routed	100
Infiltration	HORTON
Groundwater	NO
Snow Pack	
LID Controls	0
Land Uses	0
Initial Buildup	NONE
Curb Length	0

Infiltration Editor

Infiltration Method	HORTON

Property	Value
Max. Infil. Rate	15
Min. Infil. Rate	5
Decay Constant	9
Drying Time	7
Max. Volume	0

Maximum rate on the Horton infiltration curve (in/hr or mm/hr)

SHA CATCHMENT PRE DEVELOPMENT MODELLING CHARACTERISTICS

Pre Development

Subcatchment PreDev

Property	Value
Name	PreDev
X-Coordinate	3886.329
Y-Coordinate	5639.401
Description	
Tag	
Rain Gage	Rainfall
Outlet	Outfall
Area	23.2
Width	2320
% Slope	2.5
% Imperv	0
N-Imperv	0.15
N-Perv	0.15
Dstore-Imperv	5.08
Dstore-Perv	5.08
%Zero-Imperv	0
Subarea Routing	OUTLET
Percent Routed	100
Infiltration	HORTON ...
Groundwater	NO
Snow Pack	
LID Controls	0
Land Uses	0
Initial Buildup	NONE
Curb Length	0

Post Development

Subcatchment Development

Property	Value
Name	Development
X-Coordinate	3911.291
Y-Coordinate	5670.123
Description	
Tag	
Rain Gage	Rainfall
Outlet	Outfall
Area	23.2
Width	4660
% Slope	2.5
% Imperv	35
N-Imperv	0.011
N-Perv	0.15
Dstore-Imperv	1.27
Dstore-Perv	5.08
%Zero-Imperv	50
Subarea Routing	OUTLET
Percent Routed	100
Infiltration	HORTON ...
Groundwater	NO
Snow Pack	
LID Controls	0
Land Uses	0
Initial Buildup	NONE
Curb Length	0

Hortons Infiltration Characteristics are as per the Hillside Catchment

APPENDIX C

GeoSolve Soils Characteristics

Summary Tables

Table A.1: Soils Characteristics

Site 1		Site 2		Site 3		Site 4		Site 5	
Depth	Soil Type	Depth	Soil Type	Depth	Soil Type	Depth	Soil Type	Depth	Soil Type
0-0.25	Topsoil	0-0.3	Topsoil	0-0.3	Topsoil	0-0.4	Topsoil	0-0.3	Topsoil
0-25-0.9	SILT	0.3-1.8	SILT and silty SAND	0.3-0.6	SILT	0.4-1.4	Silt with fraction of sand	0.3-1.4	SILT and minor fine sand
0.9-1.5	silty SAND and SILT	1.8-8.0	sandy GRAVEL	0.6-1.6	sandy GRAVEL/gravelly SAND	1.4-1.7	sandy GRAVEL	1.4-8.0	silty SANDS, sandy GRAVELS/gravelly SANDS
1.5-5.0	gravelly SAND/ sandy GRAVELS			1.6-2.0	sandy SILT	1.7-1.9	sandy SILT		
5.0-6.0	sandy SILT and silty SAND			2.0-2.7	gravelly SAND/sandy GRAVEL	1.9-8.0	sandy GRAVEL/gravelly SAND some pocket of sandy SILT		
6.0-8.0	sandy GRAVEL			2.7-4.0	silty SAND (sand is fine)				
				4.0-8.0	sandy GRAVELS/gravelly SANDS some silt				

Table A.2: Measured Infiltration Rate

Test Depth	Soil Type	Infiltration Rate (Range)		
		Site 3	Site 4	Site 5
0.3 m	Silty top soils	5.4 mm/hr	11 – 25 mm/hr	7 – 16 mm/hr
3.0 m	Silt		175 – >1,000 mm/hr	
6.0 m	Silty sand			> 105 – >2,000 mm/hr
8.0 m	Sandy gravel		255 – >2,000 mm/day