Appendix C

Stormwater Assessment

Appendix C

Stormwater Assessment

This appendix contains the Stormwater Assessment, Plan Change 25, Kingston Village Special Zone, structured as follows.

Body of report

Appendix A - Plans

- C100 Stormwater Catchment Areas Rev 1
- C101 Stormwater Scheme Plan Rev 4
- C102 Typical Cross Sections, Sheet 1 of 2 Rev 2
- C103 Typical Cross Sections, Sheet 2 of 2 Rev 1

Appendix B - Supporting Information

- Photos 1 and Photo 2 (refer Drawing C101 for locations)
- HIRDS v2 rainfall data for Kingston area

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Plan Change 25, Kingston Village Special Zone

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Contents

Section **Page**

1. Introduction

1.1 Report Summary

This report details the findings of the stormwater assessment for the Kingston Village Plan Change site (hereafter referred to as the plan change site). It is separated into two main sections: Hydrological Assessment and Hydraulic Assessment. It also identifies the stormwater management options within the plan change site, provides an assessment of the options and makes recommendations based on the options assessment.

The Hydrological Assessment confirms the catchment areas associated with the plan change site and details the methodology used to estimate the flows and flow volumes associated with various significant rainfall events. It then discusses flood mitigation measures and leads into the Hydraulic Assessment section, which details the associated requirements of both the existing and proposed major stormwater components.

The Hydraulic Assessment section also discusses the stormwater management system proposed for the future development of the site.

There are a number of supporting plans for this report, located in Appendix A. Appendix B contains additional supporting information for reference.

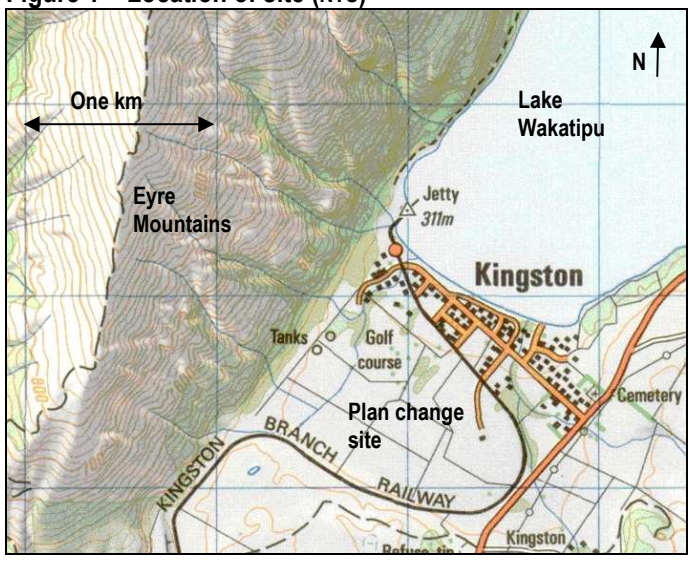
1.2 Site Description

The Kingston Township is at the southern-most tip of Lake Wakatipu, and approximately 50km from Queenstown. The current Township (approximately 208 developed lots) is immediately adjacent to Lake Wakatipu, with the dwellings no more than 500m from the Lake.

The plan change site is approximately 88ha and is bounded by the Kingston Flyer railway tracks on most sides and the Eyre Mountains to the west. It is currently zoned Rural General and is mainly utilised as farm land. For the purposes of this stormwater assessment it has been assumed that the plan change site will provide up to 800 residential lots.

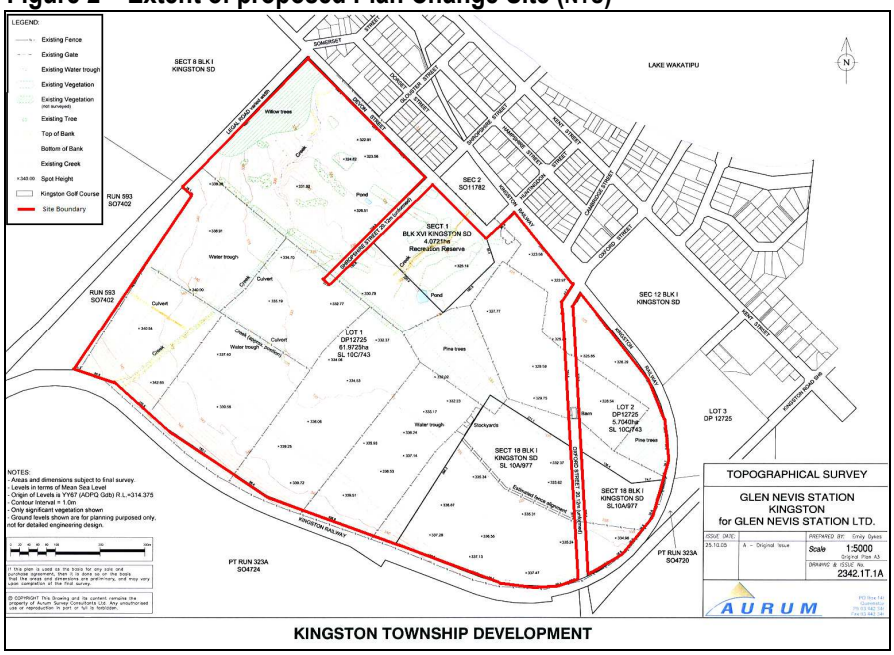
This report has been an input into the iterative master planning exercise for the site. As a result of this report, and others, the masterplan has been amended since this report was first prepared. The changes to the masterplan do not affect the overall recommendations and conclusions provided in this report.

Figure 1 – Location of site (NTS)



The below plan depicts the actual extent of the plan change site (in red).

Figure 2 – Extent of proposed Plan Change Site (NTS)



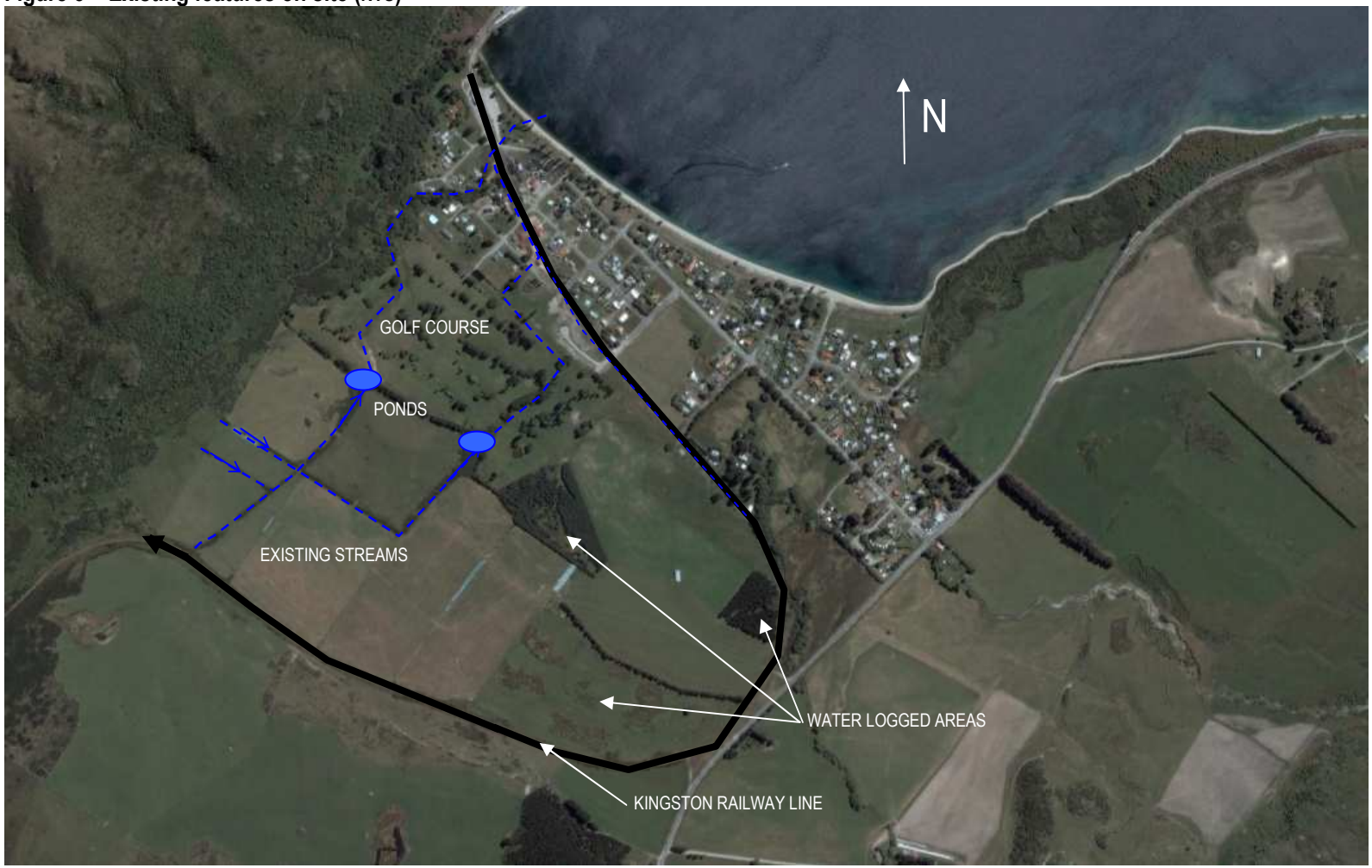
The plan change site comprises relatively planar ground gently falling about 1m vertically over a 40 to 50m length (1V:50H) from about RL340m on the southern boundary down to RL320m on the north boundary. The site mainly consists of well sorted gravel and sand in alluvial fans, with isolated areas of clay. The soil is generally silt loam, and is considered to drain moderately well. The vegetation is mainly pasture in average condition. The isolated areas with underlying clay tend to be water sodden with tussock vegetation or planted with pine trees. During the lower rainfall summer months it appears much of the rainfall within the plan change site infiltrates into the ground.

There are several drainage courses running down the western Eyre mountainside; approximate locations are on the catchment plan in Appendix A (C100). These drainage courses flow in existing deeply incised drainage channels across the western part of the plan change site (refer Figure 3 on the following page), before forming small ponds at the entrance to the golf course. At the main pond entering the golf course the stream has been filled in and piped with a 300mm diameter pipe for a 100m section. The intake is a simple pipe weir within the pond. The stream then exits the pipe and meanders through the golf course, through several culverts and across the western side of the existing Kingston Township before discharging into Lake Wakatipu.

The other, smaller stream to the east side of the golf course also crosses the golf course, runs along several drainage channels and through the existing Kingston Township along a drainage channel adjacent to the Kingston railway line. It then joins up with the larger stream to the west (described above) at the junction of Kent St and Somerset St, just prior to a triple 900mm diameter culvert. This culvert was previously a single 900mm diameter culvert with a 750mm x 750mm box culvert outlet, a new double barrelled 900mm diameter culvert has recently been constructed adjacent to this culvert.

For the rest of the plan change site, that rainfall that is not infiltrated currently sheets across the site and into a drainage channel that runs alongside the Kingston railway line. This drainage channel then converges with the channel described above between Shropshire and Gloucester Sts.

Figure 3 – Existing features on site (NTS)



[Google Earth Image]

2. Hydrological assessment

2.1 Catchment areas

Catchment areas associated with the plan change site are shown on the attached catchment plan (C100, Appendix A). The hydrological parameters are discussed in this section and summarised in Table 1.

As per the site description (Section 1.2 above), all catchment sub-areas eventually discharge into the stream on the northwestern end of the plan change site (at the junction of Kent St and Somerset St). This stream passes through a 900mm diameter culvert and a triple barrelled 900mm diameter culvert (one of which narrows to a 750mm x 750mm box culvert outlet) and then under a bridge before discharging into Lake Wakatipu. As discussed in Section 1.2, two of the 900mm diameter culverts were recently installed.

Note it is intended that following the full development of Area D (which is mainly made up of the plan change site), stormwater runoff of sub-areas D2 and D3 will be discharged through a new culvert under the Kingston Railway and a new downstream drainage pipe that will connect into an existing discharge to the Lake. This has been recommended to facilitate the management of the stormwater flows both within the development and within the catchment as a whole. This is further discussed in Section 3, but has been discussed here to clarify the splitting of catchment area D within the hydrological assessment (refer Table 1, page 5).

2.2 Infiltration loss

Calculation of the run-off volume is based on the SCS Unit Hydrograph Method.

The catchment has roughly 2 distinctly different soil types and geological conditions:

- **Hillside catchments**
  The hillside catchments consist of undifferential volcanoclastic sandstone or siltstone (soil type C), the catchment is very steep (average slopes approximately 30% - 40%) and the vegetation is considered to be in reasonably poor condition. Subsequently the infiltration of rainwater is expected to be poor; a CN (curve number) of 86 has been assumed for the assessment of infiltration loss. Refer to the Connell Wagner Rockfall / Debris Flow Hazard Appraisal for discussion on the stability of this catchment.

- **Flat catchments**
  The catchments on the flat land areas consist mainly of well sorted gravel and sand in alluvial fans and the soil is generally silt loam (soil type B). The vegetation is pasture in average condition. The soils are considered to drain moderately well; a CN of 69 has been assumed.

Infiltration loss also depends on the percentage of impervious area of the catchment. The following has been assumed within the plan change site (based on the full 800 lots master planning scenario, as this will generate the greatest proportion of impervious area):

- Existing town area (including road) 40% *(percent impervious area)*
- Road reserve 60%
- Low density residential 30%
- Medium density residential 35%
- High density residential: 40%
- Employment use: 70%

- School / education use: 30%
- Golf course: 5%

The average impervious area for each catchment is shown in Table 1.

2.3 Run-off transformation

The transformation of excess run-off has been assessed using the Clark Unit Hydrograph Method, which requires the input of the time of concentration (T_c) and storage coefficient (R).

The time of concentration is based on a combination of the New Zealand Institution of Engineers guidelines (for flat area run-off and developed catchments) and the Bransby-Williams formula (for stream catchments). The results of the time of concentration analysis are shown in Table 1.

The following ratio values have been assumed for the calculation of the storage coefficient (Note that a large ratio volume results in a slower (i.e. less peaky) runoff hydrograph):

- Steep hill catchments Ratio = 0.45
- Rural flat land areas Ratio = 0.60
- Developed high storage residential areas Ratio = 0.35

The storage coefficient (R) is calculated by:

$$Storage\ coefficient = R = \frac{(T_C \times Ratio)}{(1 - Ratio)}$$

The results of this calculation are shown in Table 1, page 5.

2.4 Rainfall

Rainfall data has been derived from HIRDS v2 published by NIWA. The HIRDS information for this site is attached in Appendix B.

An allowance for climate change impact on the rainfall intensities has been included in the final rainfall assessment. This is in accordance with Ministry for the Environment guidelines. Assuming an average temperature rise of 2° by the end of this century, the expected rainfall intensity increase varies from approximately 10% - 15% (depending on rainfall probability and storm duration). The allowance within this assessment is 15%.

Hydrological simulation runs have been carried out for 24 hour design storms (nested storm) with an Annual Exceedence Probability (AEP) of 1%, 2% & 10% (ARI100y, 50y & 10yr storm events respectively).

2.5 Model sensitivity

We note that the hydrological analysis is a model, and as such has the same limitations associated with any model of a complex system. The accuracy associated with this system is in the order of plus or minus 20 to 40 percent.

Table 1 Catchment areas and hydrological results

Catchment	Area (ha)	Curve Number (CN)	Time of Concentration (Tc) (hr)	Ratio	Storage Coefficient (R) (hr)	Impervious Area (%)	Peak Flow		Run-off Volume	
							AEP 10% (ARI 10yr) (m^3/s)	AEP 1% (ARI 100yr) (m^3/s)	AEP 10% (ARI 10yr) (1000 m^3)	AEP 1% (ARI 100yr) (1000 m^3)
Existing Scenario										
A	66.4	86	0.67	0.45	0.55	0 %	2.2	5.5	32	68
B	35.4	86	0.50	0.45	0.41	0 %	1.3	3.4	17	36
C	46.5	86	0.50	0.45	0.41	0 %	1.7	4.5	23	48
D1	50.0	69	0.33	0.60	0.50	10 %	1.2	3.4	18	40
D2	38.0	69	0.33	0.60	0.50	0 %	0.8	2.4	11	28
D3	18.0	69	0.33	0.60	0.50	0 %	0.4	1.2	5	13
Junction*	254.3						7.4	19.8	107	232
Developed Scenario										
A	66.4	86	0.67	0.45	0.55	0 %	2.2	5.5	32	68
B	35.4	86	0.50	0.45	0.41	0 %	1.3	3.4	17	36
C	46.5	86	0.50	0.45	0.41	0 %	1.7	4.5	23	48
D1	50.0	69	0.25	0.35	0.13	32 %	2.4	6.5	23	47
Junction*	198.3						6.7	17.0	95	199
D2	38.0	69	0.25	0.35	0.13	37 %	1.9	5.1	19	37
D3	18.0	69	0.25	0.35	0.13	36 %	0.9	2.4	9	17
Total*	56.0						9.1	23.6	123	253

*Note: The maximum peak flow is not equal to the summation of the peak flows of individual sub-catchments, due to differences in timing of the run-off peak.
In the existing scenario the flows at the junction is the combined flows of all sub-catchments, while in the developed scenario flow of area D2 & D3 are diverted through the new proposed pipe directly into the Lake.

2.6 Flood mitigation measures

The hydrological assessment shows that the catchment for the plan change site is large and that an increase of total peak flow and run-off volume is expected as a result of the development associated with the plan change. It is noted that:

- Over half of the flow and run-off associated with the catchment is discharged from the hillside sub-catchments outside the plan change site. The hillside sub-catchments peak flows and runoff volumes are not affected by the plan change as they are outside of the plan change site.
- The increase in flows from sub catchment D due to the development associated with the plan change is approximately a doubling of the peak flow and a 25% to 50% increase in run-off volume.
- Over the entire catchment, the increase in total run-off volume for a 24 hour design storm is expected to be approximately 20,000 m^3.

The following sections discuss the various sub-catchments and the proposed flood mitigation methods associated with the plan change. Following this, Section 3 Hydraulic Assessment discusses the various stormwater management measures proposed within the plan change site.

2.6.1 Southern hillside catchments

Following development of the plan change site the flow from the hillside catchments will account for approximately half of the total runoff within the catchment (currently the hillside catchments account for over 65% of the runoff). The hillside catchments are very steep with reasonably poor vegetation; infiltration on the hillsides is poor. As described in Section 1.2 Site Description, the north western corner of the plan change site is a golf course; there is an existing water course that meanders through this area (western side of the golf course). South of this the hillside catchments currently drain via multiple drainage channels to this location.

However, in the analysis of the stability of the hillside area (refer to the Connell Wagner Geotechnical and Contamination Hazard Appraisal), it was recognised that the hillside was of limited stability. There was concern regarding the construction of structures near the potentially unstable hillside. To enable full utilisation of the plan change site for development in the future it was recommended that a bund be installed along the western boundary of the site, to provide protection should a debris flow type event occur on the steep hillside.

With the addition of a channel to the immediate west of the debris flow protection bund the bund could also be utilised as part of a stormwater conveyance system. Refer C101 for the location of this bund. The construction of this channel and bund (which would be designed to managing a 100 year return period flow, AEP 1%) would greatly assist in managing the impact of extreme events within the plan change site through the isolation of the hillside catchments from the areas of development within the plan change site. The stormwater flow would then be directed into the existing stream that meanders through the golf course. The development of a channel and bund system along this length would also decrease the height of the bund necessary for providing debris flow protection, as the substantial channel and bund would work together to provide the necessary protection. The channel would also provide a good immediate source of fill for the bund.

The preferred option in this area is to utilise the channel and bund system described above to isolate the southern hillside catchments from the plan change site. Refer to C102 in Appendix A for a typical cross section of this channel / bund.

2.6.2 Plan change site

The plan change site is approximately 88ha (Area "D" also includes the golf course and a section of the existing Township). As described in Section 1.2 Site Description the site currently drains towards the north, with the western side of the site directed into two streams and the eastern side either eventually infiltrating into the ground (there are several waterlogged areas of underlying clay) or running off and being collected in a drainage channel which runs alongside the railway and drains to the northwest, connecting with the main western stream which runs through the golf course. The main stream passes through multiple existing culverts. Section 3 Hydraulic Assessment details the capacity of the multiple downstream culverts, they are under capacity for the current pre-development Q_{10} design flow. With the whole 255ha catchment draining to this area and the proximity to the existing houses it will be difficult to mitigate flooding in this area.

It is therefore proposed that the catchment be split and a new stormwater discharge facility be constructed for the site. It is proposed that the run-off from sub-areas D2 and D3 be diverted in a culvert under the existing Kingston Railway and via a new stormwater pipe along Oxford St. This flow will connect into an existing discharge on the eastern side of the Kingston Township, immediately prior to Lake Wakatipu. This existing discharge currently appears undersized and in need of upgrade – this could be completed as part of the upgrade to take the additional flow from sub-areas D2 and D3). Plan C101 depicts this proposed new system, it is further discussed in Section 3. Note that the short stream upgrade would be designed to ensure the extra discharge does not have an adverse effect on the flooding risk of the stream (i.e. the discharge location would be close to the existing outlet and the outlet would be upgraded).

The splitting of sub-catchment D would result in a decrease in total peak flow within the stream to the west of the plan change site.

2.6.3 Stormwater attenuation

To assist with the management of the peak stormwater flows, stormwater attenuation will be included within the plan change site. The total increase in run-off volume for a 24 hour design storm is expected to be approximately 20,000 m^3. Current good practice is to design on-site attenuation to ensure pre-development flows do not exceed post-development flows. However, this site is very close to Lake Wakatipu, which, at 30,000ha, effectively acts as a large stormwater buffer zone. That is, in an extreme event the total runoff from the plan change catchment will make a negligible difference in Lake level. In terms of water quality, water polishing swales and streams are proposed within the plan change site, which decreases the need to use attenuation to aid in water quality.

As in Section 2.6.2 above, it is proposed to split the flows within the development site, resulting in a decrease in total peak flow within the stream to the west of the plan change site, which passes by multiple existing dwellings. The splitting of this flow means the increase in runoff associated with the plan change is already mitigated, further decreasing the need for on-site stormwater attenuation such that pre-development flows do not exceed post-development flows.

Given the above, an attenuation volume of 10,000 m^3 is suggested in this situation, as depicted on C101. This attenuation is depicted solely within the existing golf course area, in the vicinity of the western stream. This will further reduce the peak flows associated with the catchment and minimise the upgrade works required to the existing western stream. As above, with the splitting of the flow the peak flows associated with this western stream will actually decrease with the development of the plan change site.

3. Hydraulic assessment

3.1 General

This section includes a summary of the proposed stormwater management within the plan change site, depicted on drawings C101, C102 and C103. It should be noted that the cross sections in Appendix C depict the engineering requirements only, for the actual cross sections proposed within the site (incorporating environmental and aesthetic inputs) refer to the Woods Bagot plans.

It also specifically discusses areas where the flow will be restricted, that is, the various culverts and connections within the stormwater areas. The following components of the existing downstream stormwater network (to the west of the plan change site and through the existing Kingston Township) have been assessed on their current flow capacity. This capacity has then been compared with the existing and developed design flows.

- Existing stream (refer Section 3.2)
- Existing series of culverts, including a triple 900mm diameter culvert (refer Section 3.3)

The design flows and necessary capacity for the proposed new stormwater pipe under the railway embankment and along Oxford St have also been assessed (refer Section 3.4).

3.2 Stormwater management within the plan change site

As previously discussed it is proposed that the stormwater within the plan change site is split into two separate areas, catchment A-B-C-D1 and catchment D2-D3. Within this, catchments A, B and C are separated from the Kingston Village development through a new stormwater channel and bund and a new stream which then connects into the existing stream which takes the flow from the whole catchment (A, B, C and D1) out to Lake Wakatipu. Refer to Section 2.6 Flood Mitigation Measures for further detail regarding this channel / bund. Note that as discussed in Section 2.1 this existing stream is currently the outlet for the entire catchment (sub-catchments A, B, C and D).

Sustainable urban drainage systems (SUDS) are proposed within the plan change site to provide both source control of stormwater quality and quantity (flow rates). The features included are infiltration trenches and stormwater polishing swales, very limited or no use of kerb and channel and a system of open watercourses throughout the development. The stormwater will be managed with a network of minor swale drains connecting to stormwater pipes, which then discharge into a series of major streams and swales around the site. C101 depicts the major system of swales and streams (the minor networks will be detailed once the development master plan has been confirmed). Drawing C101 is linked to drawings C102 and C103, which illustrate typical cross sections of both the proposed swales, streams and stormwater bunds noted on C101. Note that the typical cross sections shown on C102 and C103 also detail the proposed freeboard levels associated with the system of swales and streams. Due to the sloping nature of the site and setback from Lake Wakatipu a specific minimum building level is not applicable.

It is noted that the significant event within the Lake Wakatipu catchment in 1999 (recorded as a 1 in 150 year event) resulted in flooding to the level of 312.78m RL; the plan change site is at least 320m RL. As part of the development of the plan change site the waterlogged areas within the site will be drained (and in some cases relocated) through earthworks and the above-described swale network.

3.3 Existing stream

The existing stream to the west of the site has been assessed on site where it crosses through the existing Kingston Township. In general the existing stream bed is approximately two metres wide, with side slopes of approximately 1H : 1V (1 horizontal : 1 vertical). The height of the stream bank varies

and is assumed to be approximately 0.9 m. The longitudinal gradient of the stream is averaged at approximately 3% (1 in 33).

The stream capacity (prior to overflow of the bank, without allowing for a freeboard) is approximately 8 m^3/s (assuming a Manning's n value of 0.04, as could be expected in this type of stream). This capacity was derived utilising the Manning's formula. If 300 mm freeboard is considered, the capacity reduces to 4 m^3/s.

The design peak flow for the existing and future scenarios are presented in Table 2.

Table 2 – Existing stream (SW 1) hydraulic assessment

Scenario	Capacity	Peak Flow (m^3/s)		Flood Depth (m)	
		AEP 10%	AEP 1%	AEP 10%	AEP 1%
Existing	8.0 m^3/s	7.4 m^3/s	19.8 m^3/s	0.9 m	1.5 m
Developed	8.0 m^3/s	6.7 m^3/s	17.0 m^3/s	0.9 m	1.4 m

Note: For flood depths exceeding stream bank level (i.e. 0.9m) it is assumed that the stream bank continues at 3H : 1V

We note that in a 100 year return period event (AEP 1%) the stream would not be contained within the banks. There are multiple houses close to the existing stream (as close as 20m offset from the stream). The assumption for calculating the flood depth once the stream banks are overtopped (refer note above) is conservative as it is expected that the cross-sectional area will be higher due to flatter slopes above the top of stream bank.

Under the current conditions it is expected that the stream is just able to cope with the Q_{10} design flow (if no allowance is included for freeboard).

The performance of the existing stream (which is not made worse by the development of the plan change site) will no doubt be further assessed as part of the Stormwater Management Plan that is to be completed by Queenstown Lakes District Council (QLDC) for the wider Kingston area.

3.4 Existing culverts

The existing stream passes through several culverts just prior to discharging into Lake Wakatipu, including a recently upgraded triple 900mm diameter culvert under the Kingston Flyer railway line (one of which has a 750mm x 750mm box culvert outlet). Assuming that the culverts are discharging peak flow under inlet controlled conditions the design capacity of the culverts (i.e. head water not exceeding soffit* of the culvert inlet) is approximately 3 m^3/s. Considering surcharge (based on the survey in the area) this increases to approximately 5.5 m^3/s.

Considering the current Q_{10} design flow of approximately 7 m^3/s, these culverts are below current design standards and will require upgrade. The upgrade options will be reviewed as part of detailed design and incorporate the effect of the proposed attenuation within the plan change site. Upgrade options could include an increase in headwall height, breaking out of the current 750mm x 750mm box culvert outlet and so on.

** the soffit of the culvert is the top of the pipe (internal)*

3.5 New proposed discharge pipe

A new stormwater pipe is proposed as shown on C101, to manage the run-off from areas D2 and D3. This new stormwater pipe will cross under the existing railway in a culvert and continue along Oxford St to an existing discharge to Lake Wakatipu. The design flows for this stormwater pipe (from areas D2 and D3) are:

- Q_{10} = 2.8 m^3/s
- Q_{100} = 7.5 m^3/s

It is expected that this would require a 1500mm diameter drainage pipe.

It is likely that the existing open channel discharge into Lake Wakatipu, which this pipe will connect into, is undersized. The catchment area for this discharge extends into the adjacent eastern hills and is quite large. The current performance of the existing open channel will likely be further assessed as part of the Kingston area Stormwater Management Plan. If the existing open channel is undersized it may need to be upgraded to ensure the stormwater in this area can be managed in peak flow conditions and also to prevent outlet control conditions limiting the capacity of the new pipe.

3.6 Summary

For the proposed development the following flood management and stormwater drainage facilities are recommended.

- The addition of a channel / bund along the western boundary of the plan change site, that extends just north of the residential area located in the south west corner of the golf course, to separate existing flows from the Eyre mountains from the development.
- Upgrade of the existing stream (and associated culverts) through the Kingston Township area that are currently under capacity (this stream is currently being assessed as part of the Stormwater Management Plan being completed by QLDC for the Kingston area).
- Attenuation of approximately 10,000m^3 of stormwater runoff on site (within the golf course) to assist with the management of peak stormwater flows that discharge into an existing stream.
- Sustainable urban drainage systems (SUDS) are proposed to provide both source control of stormwater quality and quantity (flow rates). The stormwater will be managed with a network of minor swale drains connecting to stormwater pipes, which then discharge into a series of major streams and swales around the site.
- Drainage of the major swales described above into the existing stream through the golf course or to a new 1500mm diameter drainage pipe along Oxford St that is proposed as part of the development. This pipeline would discharge at the existing Kingston Stream outlet into Lake Wakatipu, with the stream outlet being upgraded as part of the works.
- Drainage (and in some areas relocation) of wet areas through earthworks and the swale network as above.

Management of stormwater within the plan change site is proposed such that the development will effectively be stormwater neutral to downstream watercourses. This is achieved through the swale system as described above to assist in the management of peak flows, incorporating stormwater attenuation in certain areas and splitting the stormwater flows within the plan change site. With the management measures proposed and the proximity of Lake Wakatipu the development of the plan change site will have minimal downstream impact on the existing Kingston Township stormwater infrastructure and on the water quality of Lake Wakatipu.

Otago Regional Council has been consulted on the proposed stormwater solutions for the development of the plan change site, and has agreed in principle to the concepts presented in this report.

Appendix A

Plans

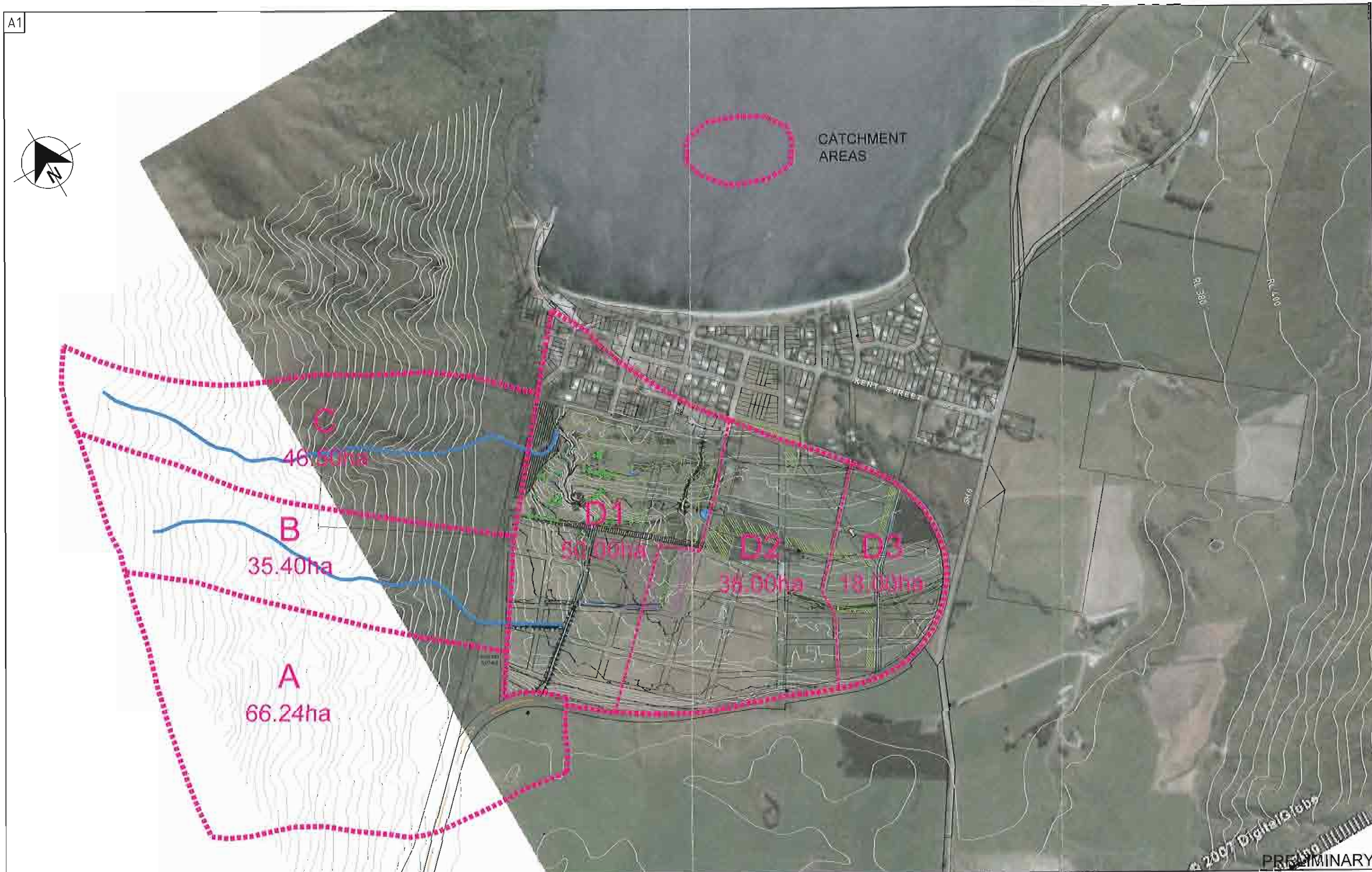
Appendix A

Plans

The following plans are appended to this report:

Dwg No.	Title	Revision
C100	Stormwater Catchment Areas	1
C101	Stormwater Scheme Plan	4
C102	Typical Cross Sections, Sheet 1 of 2	2
C103	Typical Cross Sections, Sheet 2 of 2	1

A1



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Designed CMS		04/12/07
Verified MMF	MMF	04/12/07
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Drawing Title: STORMWATER CATCHMENT AREAS

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Scale 1:5000 (AT A1) 1:10000 (AT A3)

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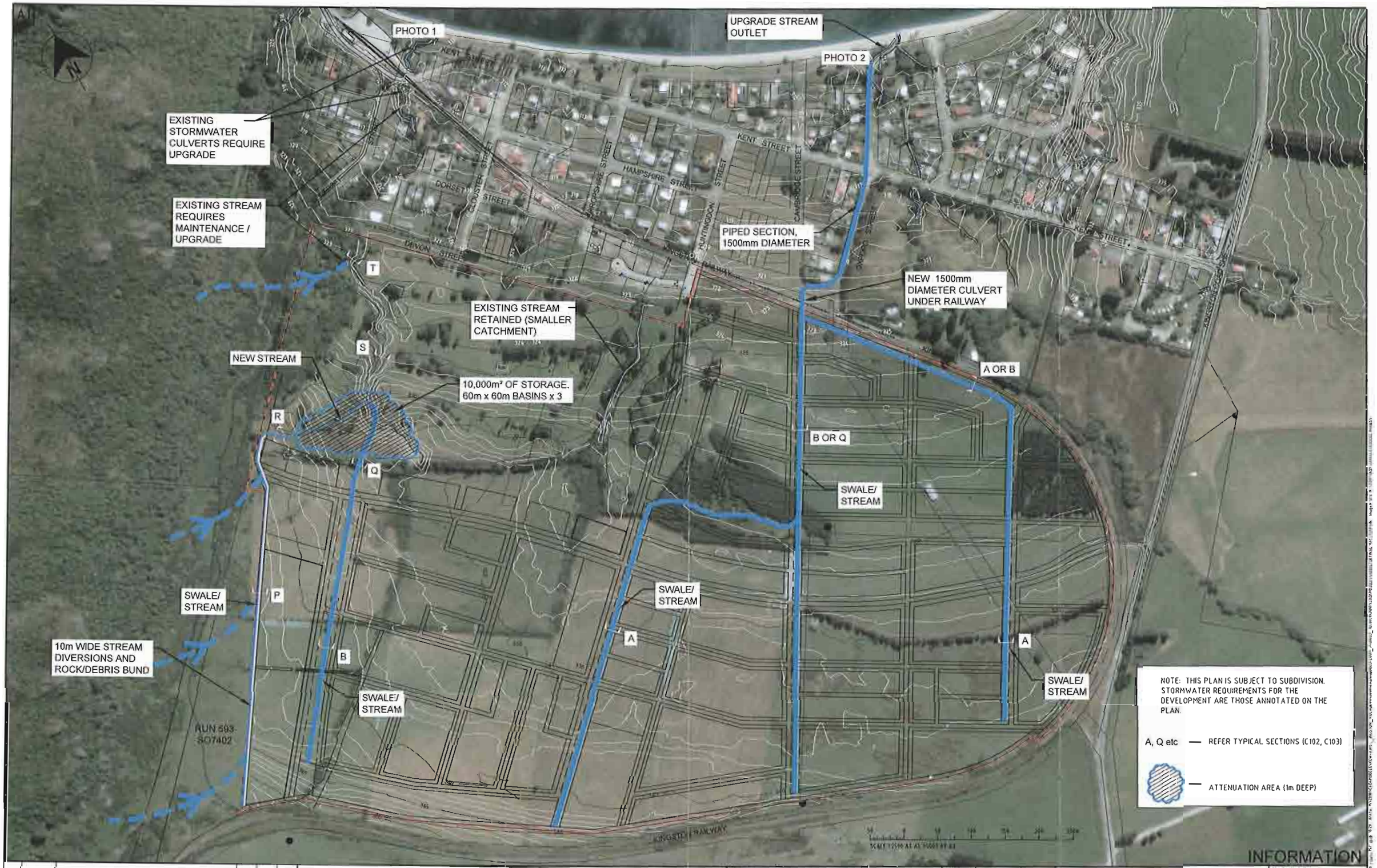

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4	13/10/08	RE-ISSUE FOR INFORMATION	CMS	MMF	MMF
3	22/02/08	RE-ISSUE FOR INFORMATION	CMS	MMF	MMF
2	17/12/07	RE-ISSUE FOR INFORMATION	CMS	MMF	MMF
1	04/12/07	ISSUE FOR INFORMATION	CMS	MMF	MMF

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Drawn	IH	Signed	Date 04/12/07
Designed	CMS	Signed	Date 04/12/07
Verified	MMF	Signed	Date 04/12/07
Approved	MMF	Signed	Date 04/12/07

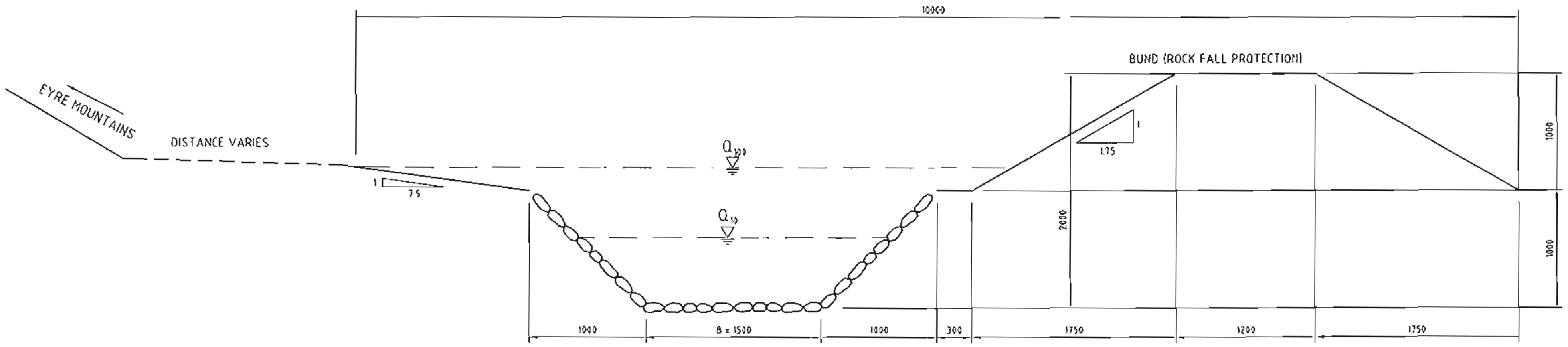
Drawing Title: STORMWATER SCHEME PLAN

CW Project No.: 209F

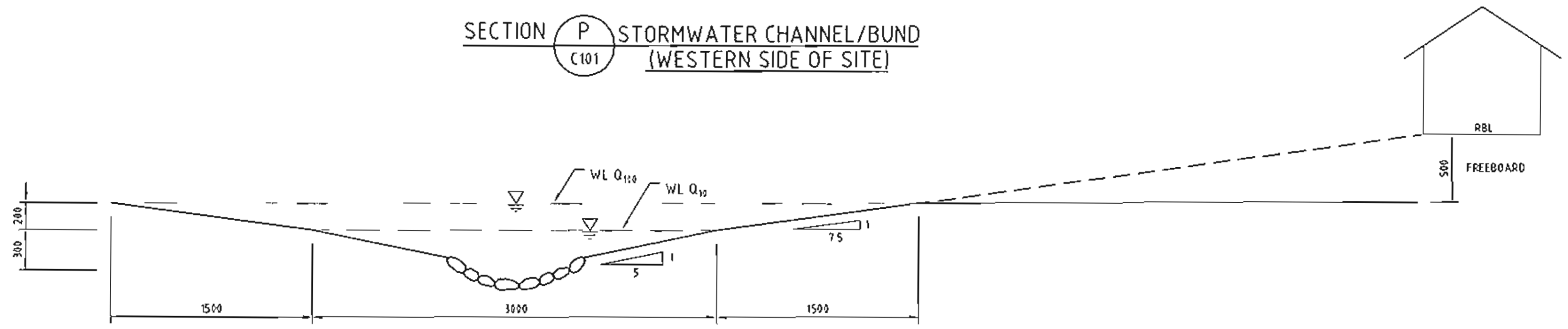
Scale: 1:2500 (AT A1) 1:5000 (AT A3)

Drawing No.: C101 Rev.: 4

A1



SECTION P C101 STORMWATER CHANNEL/BUND (WESTERN SIDE OF SITE)



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Q_{100} (max) = 3.4m³/s (V = 2.8 m/s)

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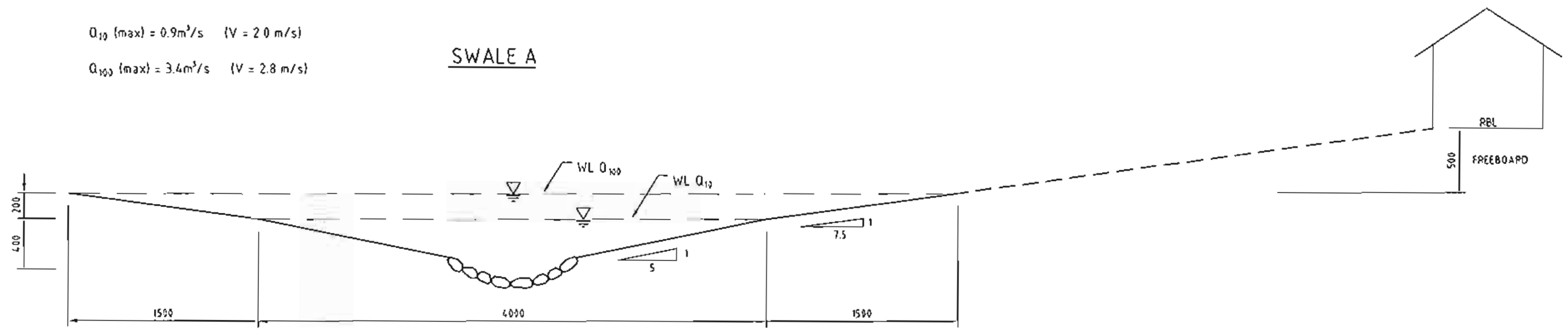

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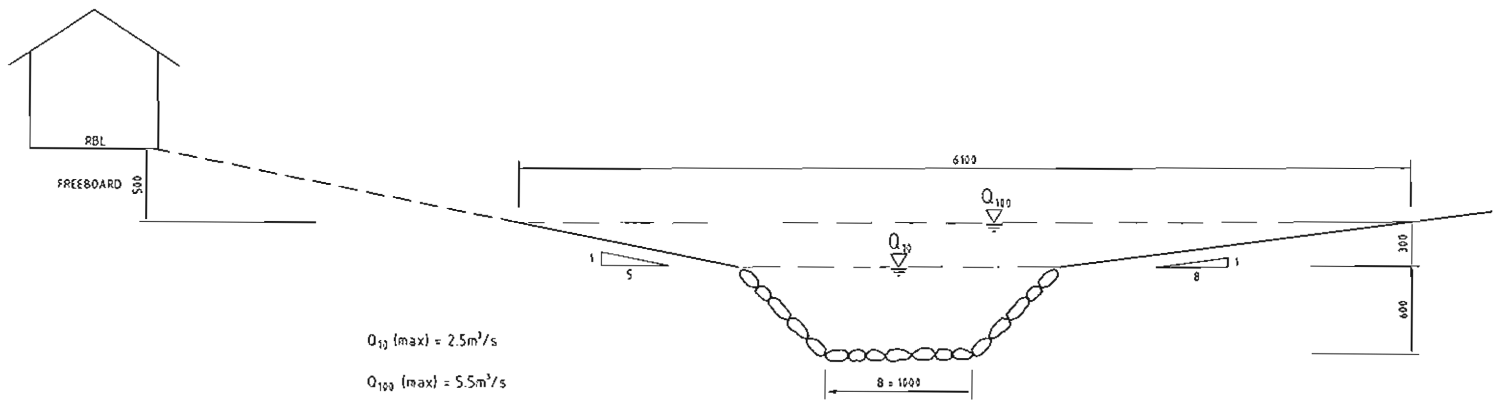

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Q_{100} (max) = 5.5m³/s

STREAM Q

INFORMATION

Rev.	Date	Revision Details	By	Ver	App.
2	04/12/07	ISSUE FOR INFORMATION	CMS		
1	26.11.07	ISSUE FOR INFORMATION	CMS		

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		Signed	Date
Drawn	IH		26.11.07
Designed	CMS		26.11.07
Verified	RVL		26.11.07
Approved	MMF		26.11.07

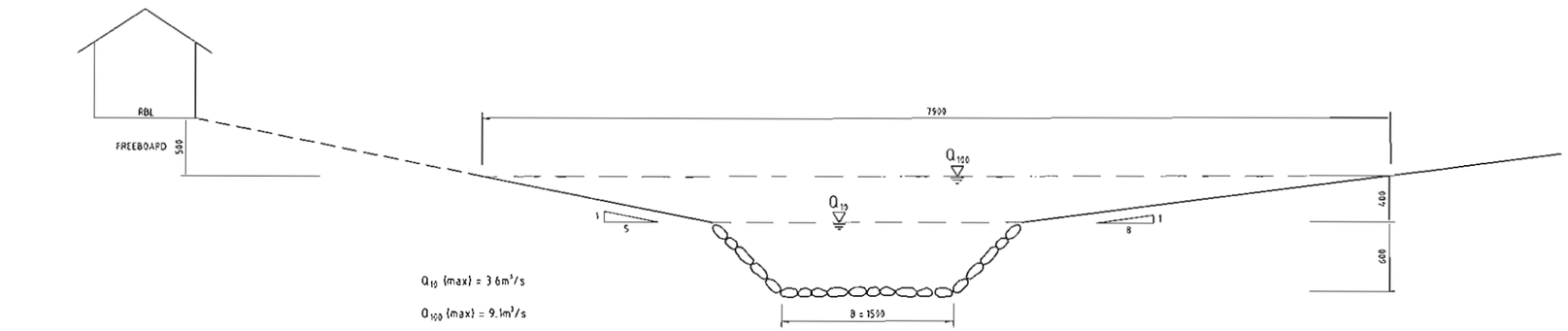
Drawing Title: TYPICAL CROSS SECTIONS
SHEET 1 OF 2

CW Project No. 209F

Scale 1:25 (AT A1) 1:50 (AT A3)

Drawing No. C102 Rev. 2

A1



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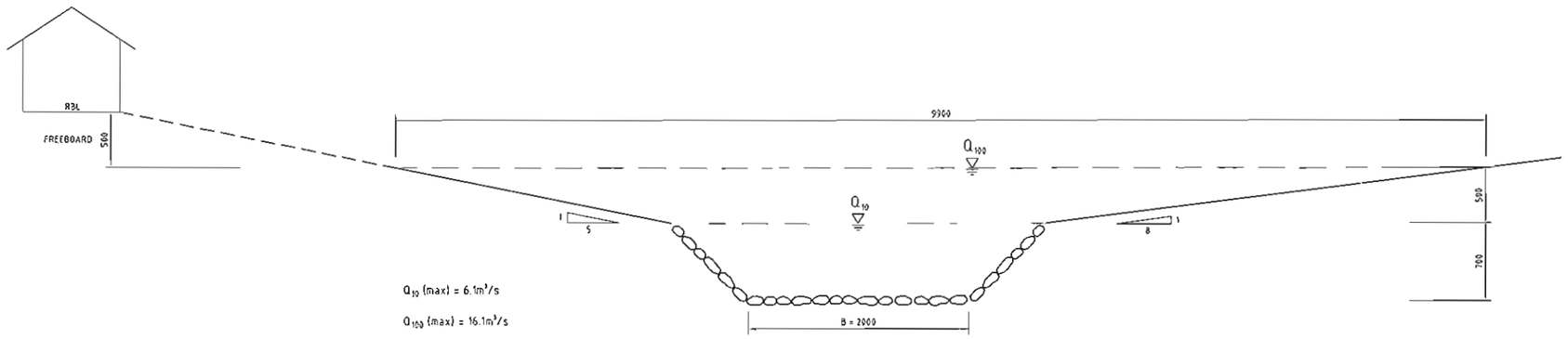

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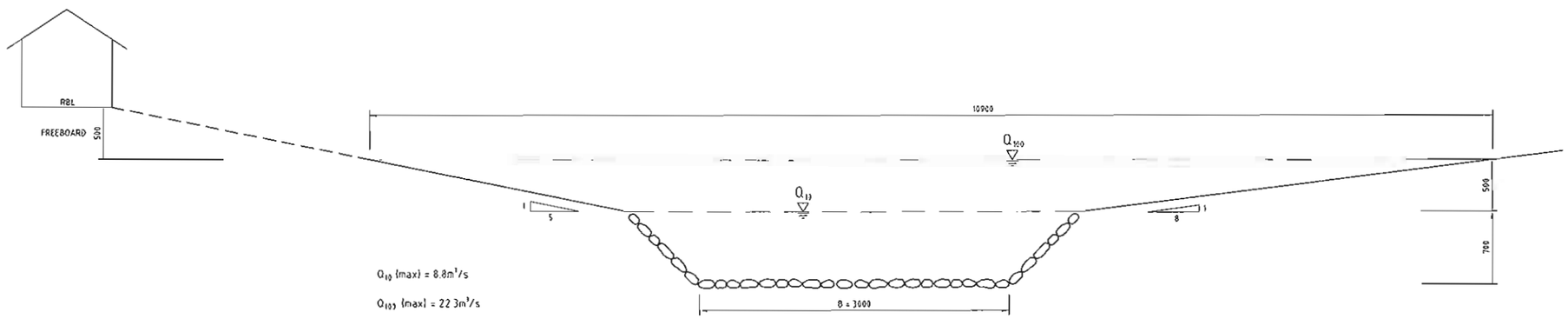

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INFORMATION

Rev	Date	Revision Details	By	Ver.	App.
1	26.11.07	ISSUE FOR INFORMATION	CMS	RVL	MMF

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Verified	RVL	Signed	Date 26.11.07
Approved	MMF	Signed	Date 26.11.07

Drawing Title: TYPICAL CROSS SECTIONS

SHEET 2 OF 2

CW Project No. 209F

Scale 1:25 (AT A1) 1:50 (AT A3)

Drawing No C103 Rev 1

Appendix B

Supporting Information

Appendix B

Supporting Information

The following supporting information is appended to this report:

- Photos 1 and Photo 2 (refer Drawing C101 for locations)
- HIRDS v2 rainfall data for Kingston area

KINGSTON PHOTOS

Refer C101 for locations

Photo 1

Photo 2

Catchment	**Kingston**										
Scource	**HIRDS**										
Station / Location	**5533000**	**2174000**									
ARI	**100 yr**										
Period	(min)	0	10	20	30	60	120	360	720	1440	2880
	(hr)					1	2	6	12	24	48
Rainfall Depth	(mm)	0.0	13.2	18.6	22.9	32.4	43.3	68.7	91.8	122.8	151.8
Climate change			16.0%	15.7%	15.4%	15.0%	14.8%	14.2%	13.8%	13.4%	11.0%
Design Rainfall	(mm)	0.0	15.3	21.5	26.4	37.3	49.7	78.5	104.5	139.3	168.5
Period	(min)	0	5	15	60	120	180	360	720	1440	2880
HEC	(mm)	0	**7.7**	**18.4**	37.3	49.7	**59.3**	78.5	104.5	139.3	168.5

Catchment		**Kingston**									
Scource		**HIRDS**									
Station / Location		**5533000 2174000**									
ARI		**10 yr**									
Period	(min)	0	10	20	30	60	120	360	720	1440	2880
	(hr)					1	2	6	12	24	48
Rainfall Depth	(mm)	0.0	5.5	8.2	10.4	15.7	21.9	37.1	51.8	72.3	87.6
Climate change			16.0%	15.6%	15.2%	14.8%	14.2%	13.6%	13.0%	12.4%	10.2%
Design Rainfall	(mm)	0.0	6.4	9.5	12.0	18.0	25.0	42.1	58.5	81.3	96.5
Period	(min)	0	5	15	60	120	180	360	720	1440	2880
HEC	(mm)	0	**3.2**	**7.9**	18.0	25.0	**30.7**	42.1	58.5	81.3	96.5

Appendix D

Water and Wastewater Flow Assessment

Appendix D

Water and Wastewater Flow Assessment

This appendix contains the water and wastewater flow demand assessment, depicting various flow scenarios and the Connell Wagner preferred solution.

It also contains copies of:

- the MWH memo to QLDC regarding water supply peaking factor clarifications, which clarifies elements of the Queenstown District Council's amendments to NZS 4404; and
- the draft Kingston Water Demand Management Plan

for reference.

KINGSTON, ULTIMATE SETTLEMENT

Revision 4, 25 September 2008

		QLDC COP (QLDC amend of 4404, MWH memo 27/11/07)								Proposed 12 June 2008 NZS:4404 loads and peaks with QLDC additions 1050l/lot/day								NZS 4404 CODE							
	Unit	PF / hrs	Kingston Township	Kingston Village	Commercial (Kingston Town, incl KA)	Total	Stage 1	Stage 2	Stage 3	PF / hrs	Kingston Township	Kingston Village	Commercial (Kingston Town, Incl KA)	Total	Stage 1	Stage 2	Stage 3	PF / hrs	Kingston Township	Kingston Village	Commercial (Kingston Town, incl KA)	Total	Stage 1	Stage 2	Stage 3
BASIS OF POPULATION																									
Number of lots	lots		379	744	0	1,123	532	875	1,123		379	744	0	1,123	532	875	1,123		379	744	0	1,123	532	875	1,123
Commercial beds available (incl KA)	ppl		0	0	404	404	202	404	404		0	0	404	404	202	404	404		0	0	404	404	202	404	404
Day visitors	ppl		57	0	404	461	245	461	461		57	0	404	461	245	461	461		57	0	404	461	245	461	461
WATER																									
Average residential population (3 ppl per lot)	ppl		1,137	2,232	0	3,369	1,597	2,625	3,369		1,137	2,232	0	3,369	1,597	2,625	3,369		1,137	2,232	0	3,369	1,597	2,625	3,369
Average commercial population (all beds filled)	ppl		0	0	404	404	202	404	404		0	0	404	404	202	404	404		0	0	404	404	202	404	404
Average day visitor population	ppl		57	0	404	461	245	461	461		57	0	404	461	245	461	461		57	0	404	461	245	461	461
Flow estimate basis		700l/p/day, 3ppl per lot, peak factors 3.3 & 6.6 (Note commercial and visitors not defined in code, but generally accepted as 350l/p/d, 40l/p/d respectively)								250l/p/day, 3ppl per lot + 100l/lot/day irrigation + 200l/lot/day leakage, peak factors 1.8 & 4.0 (Note for commercial "equivalent lots" (ppl / 3 No.) assumed $^2/_3$ x 1050l/lot/day flow estimate, visitors 40l/p/d)								250l/p/day, 3ppl per lot, peak factors 1.8 & 4.0 (visitors nominally 40l/p/d)							
Flow estimates																									
Average Daily Flow	m3/d		793	1,562	158	2,513	1,198	1,997	2,513		400	781	94	1,276	608	1,015	1,276		287	563	117	932	459	776	962
Average Daily Flow (24 hrs)	l/s		9.2	18.1	1.8	29.1	14	23	29		4.6	9.0	1.1	14.8	7	12	15		3.3	6.5	1.4	11.1	5	9	11
Peak Daily Flow	m3/d	3.3	2,634	5,190	520	8,310	3,954	6,591	8,310	1.8	720	1,406	170	2,296	1,094	1,828	2,296	1.8	516	1,001	211	1,721	827	1,378	1,731
Percentage peak flow	%		32%	62%	6%	100%	48%	79%	100%		31%	61%	7%	100%	48%	80%	100%		30%	58%	12%	100%	48%	80%	100%
Peak Daily Flow (over X hrs)	l/s	24	30.5	59.7	6.02					20	10.01	19.5	2.36	31.9	15	25	32	20	7.16	14.0	2.93				
Peak Daily Flow measured in m3/hr (water supply)	m3/hr		110	215	22						36	70	8	115	55	91	115		26	50	11				
Peak Hourly Flow (over 24 hrs)	m3/hr	6.6	219	430	43	692	330	549	692	4.0	67	130	16	213	101	169	213	4.0	48	93	20	160	77	128	160
Peak Hourly Flow (reticulation sizing)	l/s		61.0	119.4	12.0						18.5	36.2	4.4	59.1	28	47	59		13.3	25.8	5.4	44.5			
Storage Volume, reservoirs (m3)	m3		4 x PDF + 8 x ADF + 180m3 (FW), over 24 hours								Assist in the reduction in peak flows by keeping the reservoir the same volume.			2,500	1,250	2,500	2,500		Assist in the reduction in peak flows by keeping the reservoir the same volume.						
WASTEWATER																									
Average residential population (3.5 ppl per lot)	ppl		1,327	2,604	0	3,931	1,863	3,063	3,931		1,327	2,604	0	3,931	1,863	3,063	3,931		1,327	2,604	0	3,931	1,863	3,063	3,931
Average commercial population (all beds filled)	ppl		0	0	404	404	202	404	404		0	0	404	404	202	404	404		0	0	404	404	202	404	404
Average day visitor population	ppl		0	0	0	57	43	57	57		57	0	0	57	43	57	57		57	0	0	57	43	57	57
Flow estimate\ basis		300l/p/day residential, 3.5ppl per lot, 180l/p/d commercial, peak factors 2.5 & 2.0								300l/p/day residential, 3.5ppl per lot, 180l/p/d commercial, peak factors 2.5 & 2.0								250l/p/day, 3.5ppl per lot, peak factors 2.5 & 2.0							
Flow estimates																									
ADWF	m3/day		400	781	73	1,254	597	994	1,254		400	781	73	1,254	597	994	1,254		331	651	101	1,086	518	869	1086
Pumping and treatment level	m3/day					1,300	600	1000	1,300					1,300	600	1000	1,300					1,100	550	900	1100
Pumping and treatment level (20 hrs)	l/s		6	11	1	18	8	14	18		6	11	1	17	8	14	18		5	9	1	15	7	12	15
PDWF	l/s	2.5	12	23	2	36	17	29	36	2.5	12	23	2	36	17	29	36	2.5	10	19	3	31	15	25	31
Percentage peak flow	%		32%	62%	6%	100%	48%	79%	100%		32%	62%	6%	100%	48%	79%	100%		31%	63%	9%	100%	48%	80%	100%
PWWF	l/s	2.0	23	45	4	73	35	58	73	2.0	23	45	4	73	35	58	73	2.0	18	38	6	62	30	49	62

STAGING	Kingston Township	Kingston Village	Commercial (Kingston Town, incl KA)
Stage 1	3/4	1/3	1/2
Stage 2	1	2/3	1
Stage 3	1	1	1

MWH Ref: Z15707

20 November 2007

Queenstown Lakes District Council
Private Bag 50072
QUEENSTOWN

Attention: **John Porter**
Water Services Manager

Dear Sir

QLDC 07/02 Water Services Network Management
Notice To Engineer No. 042
Water Supply Peaking Factor Clarifications

The purpose of this notice is to clarify elements of the Queenstown District Councils amendments to NZS 4404:2004.

Background

QLDC produced a document of amendments to NZS 4404:2004 in September 2005. This document included Council's specific requirements for section 6.11.5 of NZS 4404:2002.

It has been noted that Councils amendments to NZS 4404:2004 and the requirements in Councils Asset Management Plans (AMP) have been applied inconsistently by various parties. In some cases the extent of the variation of application of these guidelines has been significant.

This document is intended to clarify the use of the AMP and amendments to NZS 4404:2004 and to obtain consistency in approach between those using these documents.

The preparation of this document follows a meeting attended by the following organisations:

- Connel Wagner (Martin Dasler & Max Skerratt)
- GHD (Graham Robinson)
- Tonkin and Taylor (Robert Frost)
- Rationale (Tom Lucas)
- Hadley Consultants (John McCartney)
- MWH (Derek Chinn)

Recommendations

Basic Factors

The basic factors are as follows:

1. Average Daily domestic flow rate = 700 litres / person / day
2. Occupancy per residence = 3 people
3. High density accommodation (hotels and accommodation units) Average Daily Flow rate = 350 litres / person / day; occupancy 2 people per bedroom.
4. Queenstown Peak Day Flow rate = 2.35 times Average Day Flow rate
5. All other places Peak Day Flow rate = 3.3 times Average Day Flow rate
6. Queenstown Peak Hour Flow rate = 4.0 times the Average Day Flow rate

7. All other places Peak Hour Flow rate = 6.6 times the Average Day Flow rate

Borefield and Intake Designs
It was agreed that the following figures would be used when designing new intakes or bores supplying systems including a reservoir designed in accordance with Councils AMP:

1. Bore/intake capacity = Peak Day Flow rate

Where the intake or bore pumps directly into a reticulation network without a reservoir, the bore or intake is to be designed to supply the governing design capacity of the reticulation network.

Reticulation Network Design
It was agreed that the following figures would be used when designing new water reticulation systems:
The reticulation system shall be designed to convey the greater of:

1. Peak Hour Flow rate
2. Fire fighting flows plus Peak Day Flow rate

The pressure requirements are:

1. At Peak Hour minimum 300 kPa at each service connection
2. Maximum of 900 kPa
3. All hydrants have residual pressure of 100 kPa while fire flow is being abstracted under the Peak Day Demand

Reservoir Design
It was agreed that reservoirs shall be designed with minimum available storage volume comprising of the sum of the following:

1. Fire fighting reserve (W5 - 540m^3, W4 - 180m^3, W3 - 45 m^3) plus;
2. Emergency Storage of 4 hours of the Peak Day Flow rate + 1 hour of indirect peak flow rate (flow to other reservoirs and flow to other than the reticulation network) plus;
3. Working Storage of 8 hours of Average Daily Flow rate to the network

Where standby generators, standby pumps and duplicate rising mains are provided, the AMP does not require emergency storage.

Yours sincerely
MWH New Zealand Limited

Derek Chinn
Engineer

Queenstown Lakes District Council

Kingston Water Demand Management Plan

Draft 1st Version

July 2008

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This disclaimer shall apply notwithstanding that the report may be made available to Queenstown Lakes District Council and other persons for an application for permission or approval to fulfil a legal requirement.

Quality Assurance Statement	
MWH New Zealand Limited Tower 1, Deans Park 7 Deans Avenue Addington P O Box 13-249 Christchurch 8141 New Zealand Phone : 64-3-366 7449 Fax : 64-3-341 5345	**Project Manager:** Will Doughty
	Prepared by: Christine McCormack
	Reviewed by: Murray Sorrell
	Approved for issue by: Will Doughty

Revision Schedule					
Rev No	Date	Description	Prepared By	Reviewed By	Approved By

Queenstown Lakes District Council

Kingston Water Demand Management Plan

Draft 1st Version

Contents

List of Tables

List of Figures

Appendices

1 Purpose of the Plan

The Queenstown Lakes District Council (the Council) commissioned MWH New Zealand Ltd to prepare a Water Demand Strategy to study where the water is being used inefficiently and to assess the range of water demand management options for each township within the District. The Strategy forms the framework for the development of Water Demand Management Plans for implementation of practical and targeted actions focusing on prioritised areas for reducing demands in each community.

At present there is no reticulated water supply scheme in Kingston, however a new water scheme is proposed for an adjacent development to the southwest. This new scheme is currently in the concept design stage and would become vested in Council.

The purpose of this Water Demand Management Plan is to:

- Provide an overview of the proposed Kingston water supply scheme, including proposed design criteria and predicted water demands;
- Summarise demand management measures identified in the strategy and outline a toolbox of specific measures suitable for Kingston;
- Provide an action plan for implementation of water demand management measures, identifying both short and long term actions; and
- Summarise monitoring and reporting requirements to demonstrate the effectiveness of the action plan.

The Water Demand Management Plan will be a living document that can evolve and be updated as experience grows, and additional information becomes available, and can therefore be adapted to changing circumstances and improving outcomes. References are made in this text to where additional information can be found in the either the Strategy document or the Water Asset Management Plan (AMP).

2 Kingston Water Supply Scheme

2.1 Introduction to the Existing Situation

At present there is no reticulated water supply scheme in Kingston. The town's population comprises of permanent residents and holidaymakers. Existing properties have individual onsite water supplies sourced from a combination of either rainwater, private bores or tankered water. The main tourist attraction is the Kingston Flyer which is operated out of the Kingston Railway Station. The Kingston Railway Station has a surface water abstraction from a stream in Te Kere Haka Scenic Reserve, which is Department of Conservation (DoC) land.

A large development is proposed to the southwest of Kingston Township. This development will have a reticulated system for both water and wastewater, and there is opportunity for the existing township to be included in the reticulation and treatment projects of the new development. QLDC have expressed interest in having a reticulated water supply for Kingston Township through vesting the proposed scheme in Council. Currently the reticulated water supply is in the concept design phase. The "Kingston Water Supply and Wastewater New Scheme Concept Design Report" prepared by GHD for QLDC in May 2008 indicates a likely construction period of 2010/2011 for the new water supply and reticulation scheme.

2.2 Kingston Water Supply Issues

The key water supply issues to be addressed in Kingston are as follows:

- Residents in the existing township are likely to have a conservative approach to water consumption as they currently have to rely on and pay for their own individual supplies.
- The QLDC water demand design criteria specify an annual average demand of 700 L/capita/day plus a peak day factor of 3.3. These criteria are excessive compared to many New Zealand schemes, especially those with active water demand management measures.

Based on the issues outlined above, a proactive approach to water demand and lower design criteria are proposed for the new scheme as outlined in Section 3.

2.3 Proposed Kingston Water Supply Scheme

A schematic of the proposed Kingston scheme is included in Appendix A.

Table 1 provides a summary of the proposed Kingston water supply scheme. For more details, refer to the "Kingston Water Supply and Wastewater New Scheme Concept Design Report".

Table 1: Overview and Features of the Proposed Kingston Water Supply Scheme

Supply Overview	
Area serviced	Kingston Township, Kingston Village
Consumer types	Mix of residential, commercial (incl. accommodation and retail)
Existing number of lots with dwellings	208 (from AMP)
Estimated resident population	144 (from AMP
Proposed ultimate # of lots	1,250
Proposed lot size in the new development	400 m^2 for high density lots 600 m^2 for medium density lots (no low density lots)
Supply Features	
Proposed general supply type	Pressurised system Full fire-fighting capacity and storage.
Proposed supply source	Deep lake intakes in Lake Wakatipu
Proposed treatment	To meet the requirements of DWSNZ 2005 and a minimum grading of C for source, treatment and the reticulation system
Proposed connection types	Unrestricted (i.e. on-demand)
Proposed metering	Individual metering proposed for all properties

3 Kingston Water Demands

3.1 Proposed Kingston Water Demand Criteria

A workshop to discuss the proposed Kingston water demand design criteria was held on 12th June 2008 with attendance by staff from GHD (QLDC's engineer), Connell Wagner (the developer's representative), QLDC/Lakes Engineering and MWH.

The current QLDC design criteria are shown in the column "QLDC Amendments to NZS 4404" in Table 2 below. These design criteria are based on the document of amendments to NZS 4404:2004 Land Development and. Subdivision Engineering produced by QLDC in September 2005. QLDC's Water Network Manager, MWH, clarified the design criteria in a letter dated 20th November 2007 to all QLDC's consultants. The purpose of this clarification was to prevent the design criteria being applied inconsistently as it had been in the past. The Peak Day and Peak Hour Factors represent the factors that need to be multiplied by the Average Day Flow rate to get either the Peak Day or Peak Hour flowrates.

During the June 12th workshop discussions, it was agreed that the QLDC design criteria are excessive for a new scheme such as Kingston, particularly as existing Kingston residents will be used to living with their limited onsite supplies. To illustrate this point, it is useful to consider the QLDC criteria for a peak day, i.e. 3.3 times the average day of 2,100 L/residential lot/day. This equates to a very large consumption figure of 6,930 Litres per residential lot per day (or 2,310 L/capita/day). These high demands are only likely to be realised from rural-residential properties with large irrigation schemes which are not present in the existing Township. In addition, as outlined in Table 1 above, the proposed lot sizes in the new development are urban in nature and relatively small. The largest lots would be approximately 600 m^2, therefore would not be expected to have high irrigation demands.

At the workshop, proposed design criteria for the new Kingston scheme were finalised, as shown in the table below. The proposed design criteria for Kingston have a water demand management focus. The proposed Peak Day and Peak Hour factors were taken directly from New Zealand Standard 4404.

Table 2: Comparison of Water Demand Design Criteria

	QLDC Amendments to NZS 4404	Proposed Kingston Design Criteria
Residential Average Day Demand (ADD)	Residents at 2,100 L/residential lot/day (based on 700 L/capita/day)	Residents at 1,050 L/residential lot/day (equivalent to 350 L/capita/day)
Occupancy per residence	3 people	3 people
High Density Accommodation (Visitor) ADD	Visitors at 350 L/capita/day & 2 people per visitor unit (i.e. 700 L/visitor unit/day)	Visitors at 700 L/visitor unit/day
Peak Day Factor	3.3	1.8
Peak Day Demand (residential)	6,930 L/lot/day (2,310 L/capita/day)	1,890 L/lot/day (630 L/capita/day)
Peak Hour Factor	6	4

NOTE: The proposed residential allowance of 1,050 L/residential lot/day is based on residential domestic use of 250 L/capita/day plus annual average allowances of 100 L/lot/day for irrigation & 200 L/lot/day for leakage. The

proposed visitor allowance of 700 L/visitor unit/day is based on two thirds of the proposed residential allowance of 1,050 L/residential lot/day and matches the QLDC design criteria.

It is expected that the proposed design criteria should not be a hardship to Kingston residents as they will either be from existing houses with a conservative approach to consumption or in new houses which will typically have water efficient appliances and fixtures, small scale garden irrigation and planting restrictions (see Section 4.3 for more details). Universal metering is also proposed and it is expected that this will encourage wise use of water and allow early identification of high users and property leakage. The system leakage allowance of 200 L/lot/day was specified by QLDC and represents almost 20% of the residential demand allowance. This is high and a conservative allowance for leakage, especially for a new scheme properly designed and constructed, but will allow more contingency in the future. New Zealand Councils that proactively monitor leakage in existing schemes typically set their leakage targets between upper limits of 120 to 240 L/lot/day.

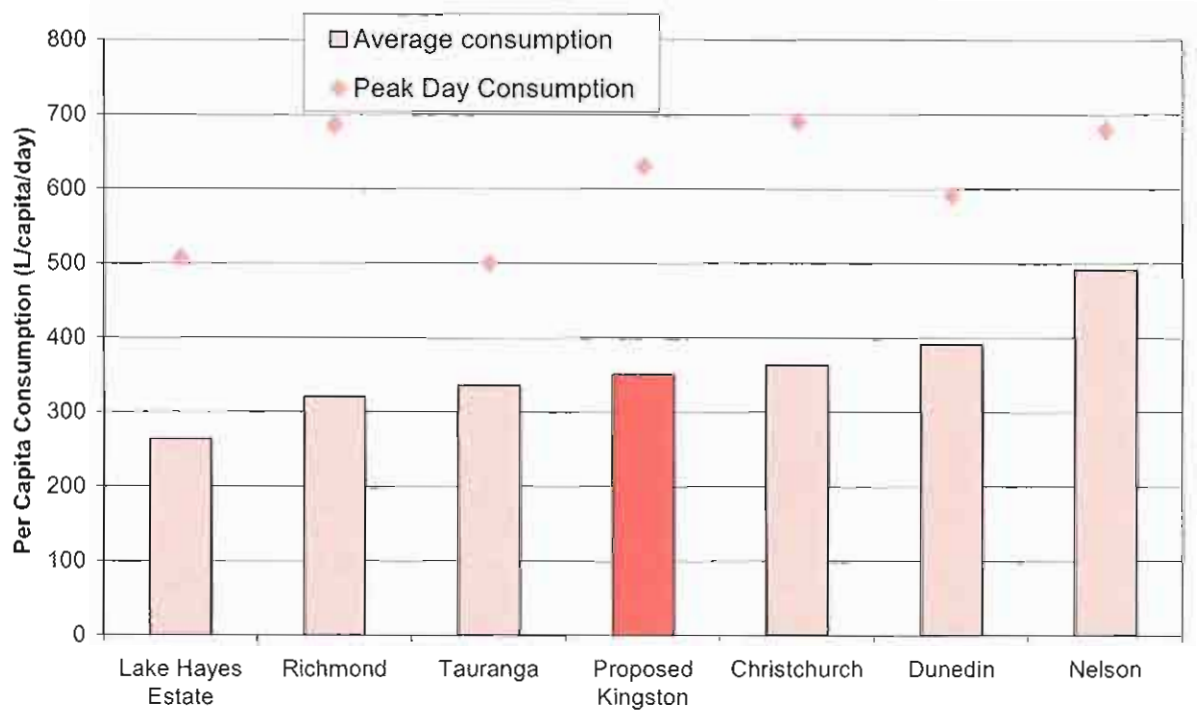
It should also be noted that the New Zealand Standard 4404: 2004 Land Development and. Subdivision Engineering (Section 6.11.5), specifies a typical daily water demand of 250L/capita/day, which is less than the proposed total allocation of 350 L/capita/day. The NZS 4404 figure represents 'total system demand' which includes leaks, delivery losses, and other non-indoor demands.

More evidence is provided in the following sub-section which compares the proposed design criteria to a representative sample of New Zealand consumption figures.

3.2 Comparison of Recent New Zealand Consumption Data

It is useful to compare the proposed Kingston design criteria to a representative sample of recent New Zealand consumption figures to ensure that the proposed design criteria will provide sufficient water for residents. This comparison is shown in Figure 1 below. The representative sample excludes Auckland cities which have high water rates and a typically wetter climate. Only Richmond, Nelson and Tauranga have universal metering (as proposed for Kingston). The most comparable area in the Queenstown District was considered to be Lake Hayes Estate as it is a large new subdivision (currently about 400 lots) with smaller lot sizes and negligible system leakage. The peak day shown for Lake Hayes represents the peak week (due to lack of daily data), therefore the actual peak day is likely to be slightly higher. Other Queenstown District schemes were excluded from this comparison.

Figure 1: Comparison of Proposed Kingston Design Criteria to Representative Sample of NZ Cities



The above graphical comparison shows that the proposed allocation for Kingston appears reasonable, given that universal metering is also proposed and both the average and peak day figures are in line with similar cities.

It is also relevant to consider the Final Report of the Water End Use and Efficiency Project (WEEP) published by the Building Research Association of New Zealand (BRANZ) in 2007. This report presented the results of a detailed survey of the water use of typical 2 to 3 bedroom houses on the Kapiti Coast, undertaken in two survey periods (summer & winter) in 2006/2007.

Some of the principal findings of the report are:

1. The average annual residential water use was 186 L/capita/day (includes property leakage)
2. Summer seasonal average use was 200 L/capita/day and winter seasonal average use was approximately 170 L/capita/day
3. Demand increased in summer and decreases in winter, however:
    1. Summer increases were due to outdoor use, principally irrigation (average 44 L/capita/day in summer and 29 year round)
    2. There was little seasonal change in indoor water use (Summer: 153 L/capita/day, Winter: 147 L/capita/day)
4. The average residential indoor water use was approximately 150 litres per person per day

The findings of this report show that the summer seasonal average use of 200 L/capita/day and a peaking factor of 1.5 to 2 can be combined to estimate a maximum day demand of 300 to 400 L/capita/day. This survey was

limited to the Kapiti Coast so is not representative of all New Zealand geographic areas but indoor water use could be considered typical. These findings indicate that the proposed Kingston indoor allocation of 250 L/capita/day (or 350 L/capita/day including irrigation and leakage) and the proposed maximum day demand of 630 L/capita/day should be sufficient.

The findings also suggest that the proposed indoor allocation of 250 L/capita/day should not be a hardship to Kingston residences as this is almost twice the reported average residential indoor water use of 150 litres per person per day. It is worth noting that the NZS4404 typical daily water demand figure of 250L/capita/day is more conservative than the BRANZ research findings as it is a total figure including system leakage.

3.3 Conclusions on Kingston Water Demand Criteria

The proposed allocation of 1,050 L/lot/day (equivalent to 350 L/capita/day and includes 250 L/capita/day for domestic use plus 100 L/lot/day for irrigation and 200 L/lot/day for leakage) should not be a hardship to Kingston residents as they will either be from existing houses with a conservative approach to consumption or in new houses which will typically have water efficient appliances and fixtures, small scale garden irrigation and planting restrictions. In addition, those existing residents with rainwater or bore supplies may wish to retain these for an alternative irrigation source.

The proposed allocation is considered to be achievable but is a clear departure from existing reticulated supplies available in the larger QLDC communities. Kingston residents and visitors will need to view potable water as a finite resource and adopt a lifestyle that allows for this which will include using water efficient fixtures, appliances and planting/landscaping practices.

The proposed Kingston visitor allowance of 700 L/visitor unit/day matches the QLDC design criteria.

4 Water Demand and Supply-Side Management Measures

4.1 Demand Measures Identified in the Strategy

Refer to Section 5 of the Strategy for detailed information on different demand management tools.

4.2 Screening Process for Selection of Demand Management Measures

In order for the Demand Management Plan to be most effective, the primary demand sources should be identified in order to prioritise the demand management measures that will provide the greatest opportunity for reducing water demands and return on investment.

Due to the current lack of a reticulated scheme, there is no information available on the primary demand sources for existing Kingston properties, however the peak population and peak indoor demands are expected to occur during the summer months.

4.3 Shortlisted Future Demand Management Measures for Kingston

The assumptions discussed earlier have been used to prioritise a short list of demand management measures for Kingston as summarised in Table 3 below (numbered in general priority order), along with comments on their relevancy.

Refer to Section 5 of the Water Demand Management Strategy for detailed information on the shortlisted demand management measures. A staged approach to implementation of the demand management measures is recommended, with both a short term (6 months to 1 year) and a long term (1 to 5 years) action plan as outlined in Section 5 of this document.

The Water Supply Bylaw (currently being updated) and the District Plan provide the legislative framework under which the recommended demand management measures will be implemented.

Table 3: Summary of Relevant Water Demand Management Tools

Tool		Comments on Relevancy of Tool
1	Universal Metering	This tool will be essential to allow for monitoring of water demands at individual properties, to identify high users and for early identification of property leakage. NZWWA supports universal metering and user pricing and believes that all users should pay for their consumption in a reasonable and transparent manner. Future consideration should be given to volumetric pricing. This would require a carefully designed communication and education programme and would need to allow for recovery of meter installation and opex costs through appropriate pricing regimes.
2	Water Efficient Fixtures	This is relevant as it will reduce water use from fixtures such as taps. To ensure high uptake of this tool, covenants could be placed on the property title to mandate installation. Water efficient fixtures, low flow showerheads and taps should be the top priority. A mandatory New Zealand Water Efficiency Labelling Scheme (WELS) covering both fixtures and appliances is due to be fully implemented by July 2009, however water efficiency information is currently available online (http://www.mfe.govt.nz/issues/water/wels-scheme.html).

Tool		Comments on Relevancy of Tool
3	Water Efficient Appliances	This is relevant as it will reduce water use from appliances such as toilets. To ensure high uptake of this tool, covenants could be placed on the property title to mandate their installation. Dual flush toilets and washing machines should be the highest priority due to their high proportion of typical residential consumption (compared to other appliances such as dishwashers).
4	Water Efficient Landscaping	Planting restrictions should be proposed for the new development to ensure that only plants that suit the climatic conditions and landscape are used. The mechanism to enforce this could be through placing covenants on the property titles which could also mandate installation of drip irrigation for landscaping. Public education may be required to encourage use of other water efficient landscaping practices such as mulching.
5	Public Education	This tool will be essential to increase public awareness of the benefits of reducing water consumption and to encourage a culture of valuing and conserving water. A formal Council-wide public education programme on water conservation is proposed. Before connecting existing properties to the proposed scheme, the Council may wish to undertake water audits through the education programme to identify the best water efficient fixtures and appliances and water conservation practices.
6	Rainwater Harvesting	Existing lots that currently utilize rainwater as their water source may wish to retain their rainwater harvesting system to provide an alternative source of water for irrigation. Similarly, existing lots that have private bores may wish to retain them to provide an alternative source of water for irrigation. In both cases, the onsite supplies could potentially also be used for other non-potable water activities such as toilet flushing if appropriate plumbing was installed.
7	Water Loss Reduction Programme	The proposed scheme must have the ability to provide at least accurate 15 minute flow data from the water take and treated water production. This data should be monitored regularly (at least weekly if not daily) to identify changes in demand or the occurrence of leakage.
8	Greywater Re-use	Greywater re-use can provide a source of water for irrigation (and other non-potable water activities such as toilet flushing). Provides higher volumes than rainwater harvesting during dry periods as volumes are based on occupation. A separate storage tank would be required plus additional plumbing arrangements which would lead to a higher cost per individual property. Also has a higher potential for health risks than rainwater harvesting. Not recommended for implementation in Kingston at this time.

4.4 Cost-Benefit Analysis of Shortlisted Demand Management Measures

A cost-benefit analysis has not yet been undertaken for this draft edition due to a lack of data. A comprehensive cost benefit analysis of the shortlisted demand management options is recommended as an improvement to this Demand Management Plan in a future stage of the scheme design. This analysis should include whole-life costing to ensure that the full lifetime costs of assets are accounted for.

5 Implementation of the Demand Management Plan

The demand management measures recommended for implementation in the proposed Kingston scheme are outlined in Table 4.

Table 4: Implementation Plan for Shortlisted Demand Management Measures

	Action	Responsibility	Estimated Funding/Resourcing	Timeframe
1	Universal Metering	Developer/Council	Property Owner	With construction of scheme
2	Water Efficient Fixtures	Developer/Council	Property Owner	tba
3	Water Efficient Appliances	Developer/Council	Property Owner	tba
4	Water Efficient Landscaping	Developer/Council	Property Owner	tba
5	Public Education	Council	tba	tba
6	Rainwater Harvesting	Property Owner	Property Owner	tba
7	Water Loss Programme	Council	tba	tba

6 Monitoring and Reporting Requirements

6.1 Programme Monitoring and Tracking

The key objectives behind monitoring and analysing water use in Kingston are:

- To develop a database of water use in Kingston
- To understand and quantify the effectiveness of water demand measures as they are implemented
- To collate data that can be used for public education and consumer awareness
- To provide feedback to Council on changes to the implementation programme or methodology

6.2 Roles and Responsibilities

The key roles and responsibilities for ongoing monitoring, analysis and reporting of water use and the impact of demand management in the Kingston Supply are summarised in Table 5.

Table 5: Key Roles and Responsibilities for Monitoring, Analysis and Reporting in the Kingston Supply

Organisation	Key Responsibilities
ORC	Enforce resource consent conditions.
QLDC	Forward demand records and report to ORC in accordance with the resource consent conditions.
O&M Contractor	Record electronic bulk meter readings & analyse demands. Provide demand data and analysis to QLDC to meet resource consent conditions.

6.3 Water Demand Management Plan Review

It is recommended that this Water Demand Management Plan is reviewed after implementation of the recommended demand management measures and on an ongoing annual basis. In addition, there must be sufficient infrastructure and instrumentation in the proposed water supply to enable effective demand management (e.g. sufficient flow meters).

In order to develop and implement an effective Water Demand Management Plan, the following improvements are recommended:

1. Undertake a comprehensive cost benefit analysis of the shortlisted demand management options. This analysis should include whole-life costing to ensure that the full lifetime costs of assets are accounted for.

The Water Demand Management Plan will be a living document that will evolve and be updated through the review process. It is anticipated that future versions of the Demand Management Plan will become more specific after a database of Kingston water usage is created.

6.4 Policy for Non-Compliance with Water Conservation Directions

Under the proposed Water Supply Bylaw, any customer who is in breach of the Bylaw is liable to summary conviction by the Council and a maximum fine of $20,000, under section 242(2) of the Local Government Act. Offences that are considered breaches of the Bylaw include:

- Interference with the water supply
- Failure to comply with water use restrictions or prohibitions introduced by Council
- Bypassing or tampering with Council water meters and restrictors
- Failure to install water conservation fittings when required to do so by Council in accordance with Council's water demand management policy (i.e. as detailed in this DMP)

This section will be developed further with input by the QLDC after finalisation of the revised Water Supply Bylaw.

MWH

Appendix A Kingston Water Supply Schematic

Appendix E

Water Scheme Plans and Investigation

Appendix E

Water Scheme Plans and Investigation

This appendix contains the following water plans.

- C300 Water Scheme Plan Rev 3
- C301 Reservoir Details Rev 2

It also contains the scala results for investigation undertaken at the proposed reservoir site.

PROPOSED LAKE EXTRACTION PUMPSTATION LOCATION

2 x STAGED RESERVOIRS 1250m³ AND 1250m³. TOTAL STORAGE VOLUME= 2500m³. RL= 400m

RAW WATER

(TREATED) (RAW)

WATER TREATMENT PLANT

PURIFIED WATER

KENT STREET

SH6

KINGSTON TOWNSHIP WATER SUPPLY DEMARCATION

NOTE: THIS PLAN IS SUBJECT TO SUBDIVISION. WATER REQUIREMENTS FOR THE DEVELOPMENT ARE THOSE ANNOTATED ON THE PLAN.

TEST BOREHOLE LOCATION (EXISTING)

KINGSTON VILLAGE WATER SUPPLY

LEGEND

- RAW WATER
- PURIFIED WATER (BULK SUPPLY AND RING MAIN)
- TEST BORE LOCATION (EXISTING)
- PROPOSED LAKE EXTRACTION PUMPSTATION LOCATION

INFORMATION

Rev.	Date	Revision Details	By	Ver.	App.
3	13/10/08	ISSUE FOR INFORMATION	CMS	MMF	MMF
2	17/03/08	RE-ISSUE FOR INFORMATION	CMS	MMF	MMF
1	22/02/08	ISSUE FOR INFORMATION	CMS	MMF	MMF

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Project: KINGSTON VILLAGE

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Designed	CMS	Signed	Date 22/02/08
Verified	MMF	Signed MMF	Date 22/02/08
Approved	MMF	Signed MMF	Date 22/02/08

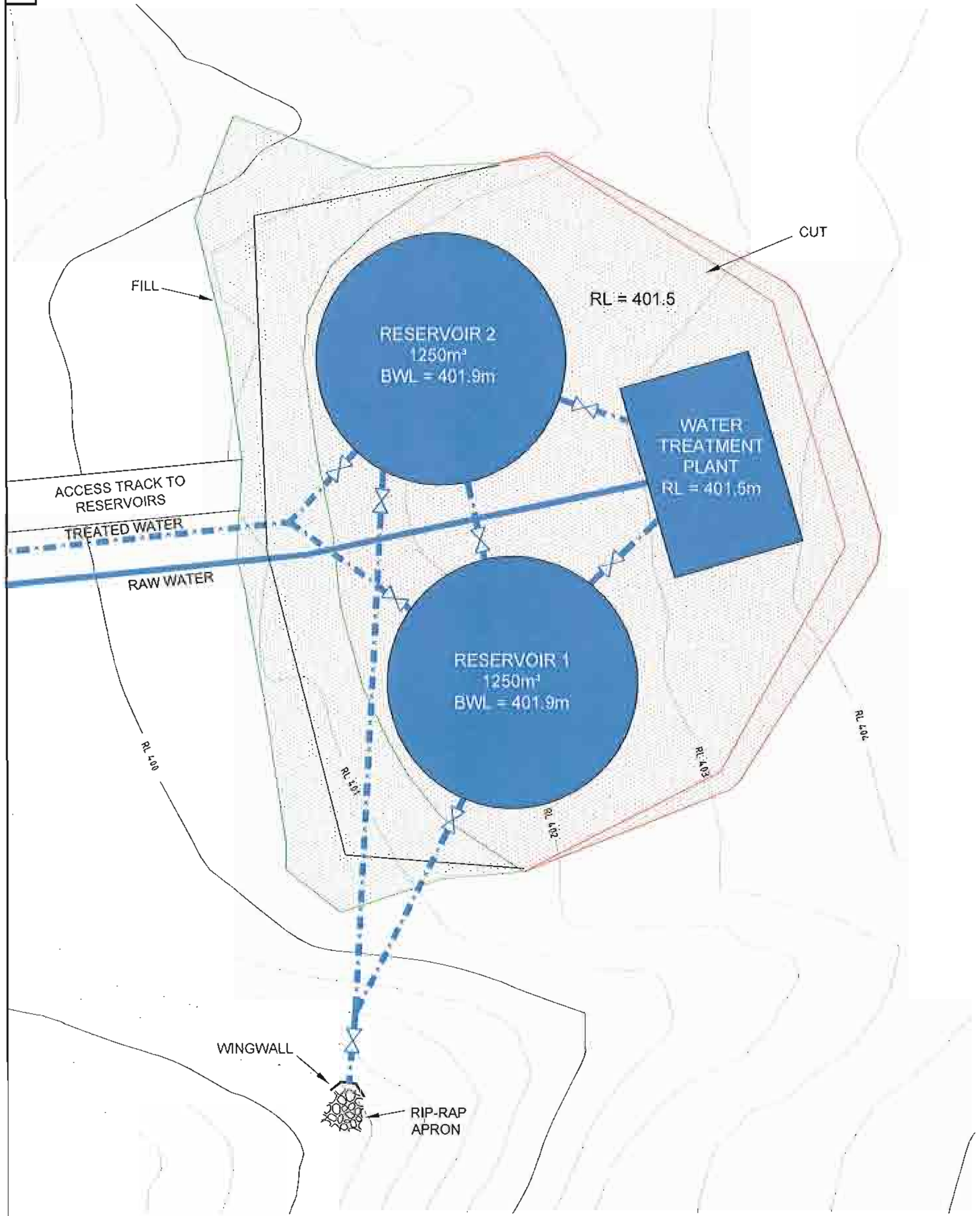
Drawing Title: WATER SCHEME PLAN

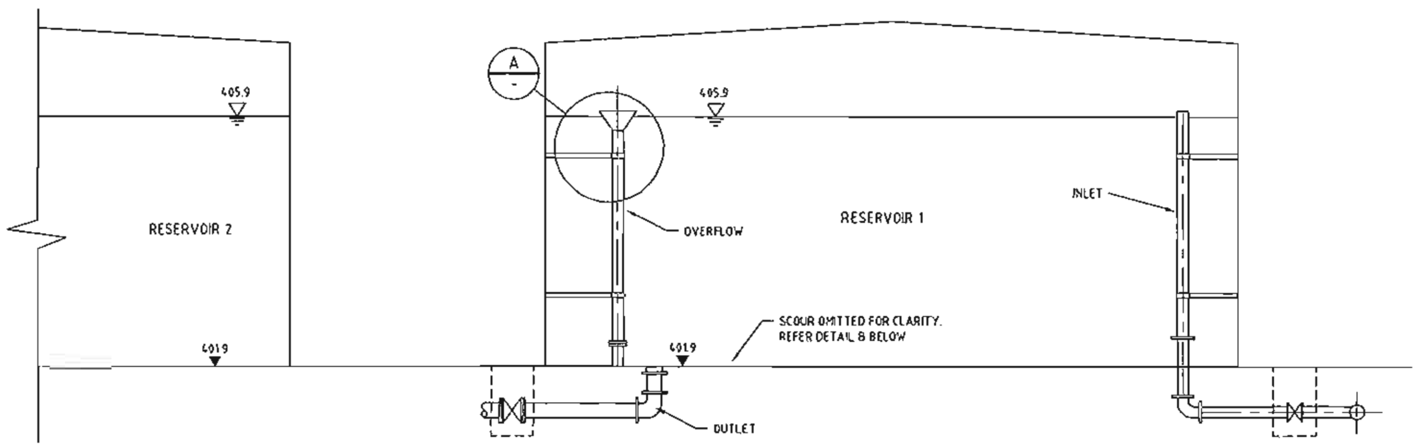
CW Project No.: 209F

Scale: 1:4000 (AT A1) 1:8000 (AT A3)

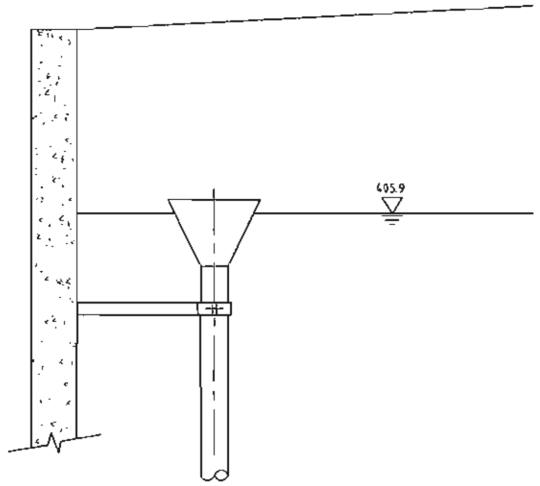
Drawing No.: C300 Rev.: 3

A1





TYPICAL DETAIL OF INLET, OUTLET AND OVERFLOW
TO PROPOSED RESERVOIRS
NOT TO SCALE



DETAIL A
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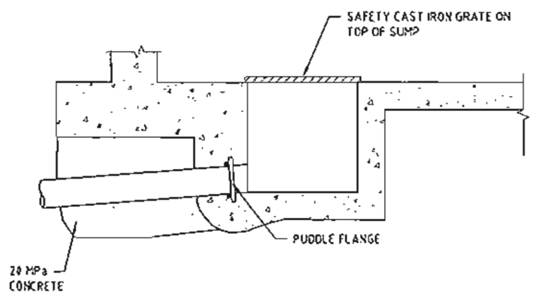
OVERFLOW



DETAIL B SCOUR
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INFORMATION

Rev.	Date	Revision Details	By	Ver.	App.
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1	22/02/08	ISSUE FOR INFORMATION	CMS	MMF	MMF

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Drawn	IH	IH	22/02/08
Designed	CMS	CB	22/02/08
Verified	MMF	MMF	22/02/08
Approved	MMF	MMF	22/02/08

Drawing Title: RESERVOIR DETAILS

CW Project No. 209F

Scale 1:200 (AT A1) 1:400 (AT A3)

Drawing No. C301 Rev 2

Scala Log Sheet
Kingston Proposed Water Reservoir Location

Depth Meas. (mm)	Cum Depth (mm)	Blows (no.)	Blows per 50mm
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100	200	7	3.50
100	300	12	6.00
100	400	18	9.00
100	500	17	8.50
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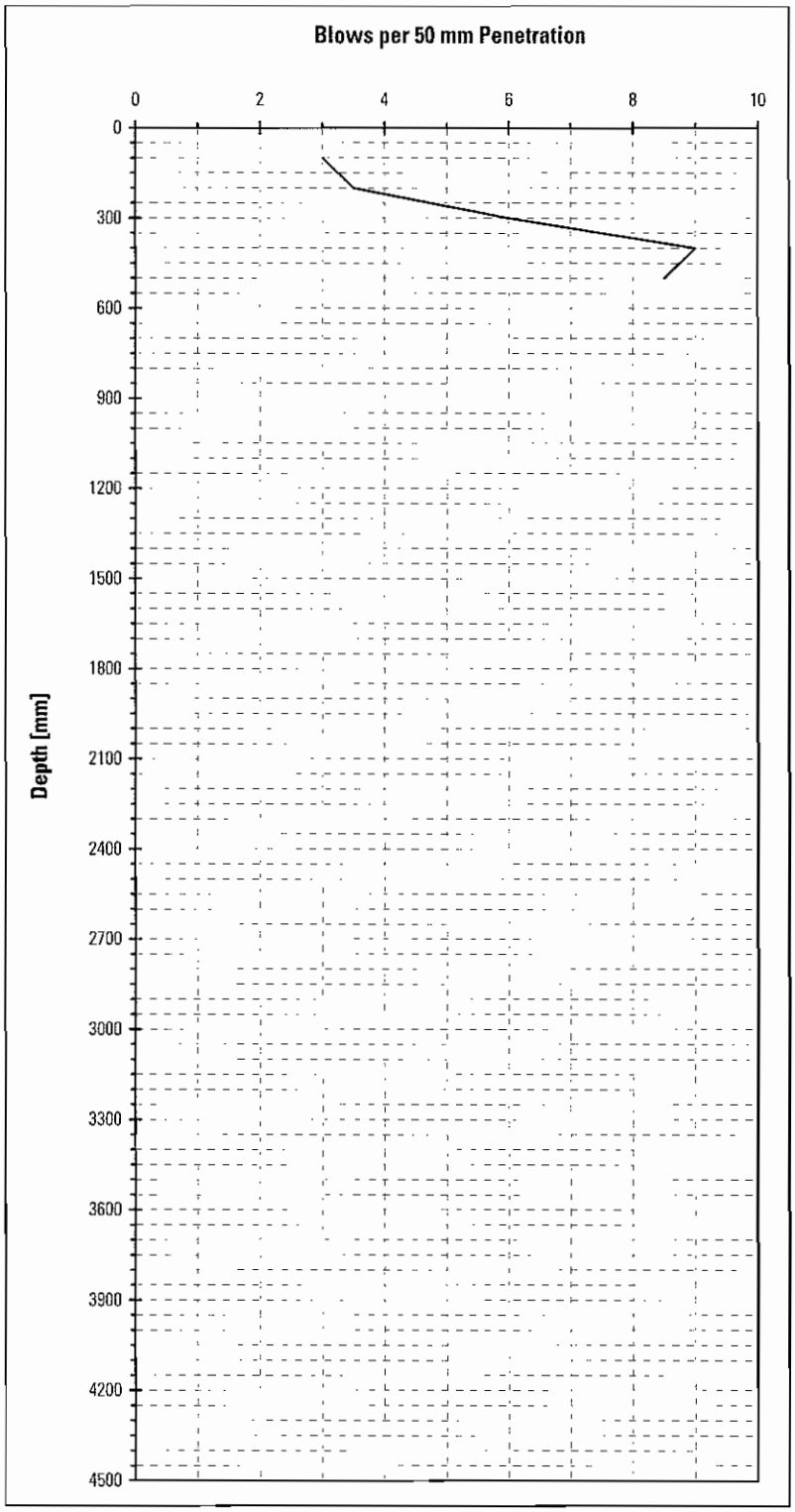

PROBE DESCRIPTION:
9 kg Hammer falling half a metre to a steel anvil, driving a 16 mm diameter rod fitted with a 20 mm diameter cone.

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Client: Kingston Village Ltd		SCALA PROBE	
Location: Plan Change Site		No: SP 1a	A4
Tester: James Lynch	Date: 14 Feb 2008	Job Number: 209F-003	Rev: 1

Scala Log Sheet
Kingston Proposed Water Reservoir Location

Depth Meas. (mm)	Cum Depth (mm)	Blows (no.)	Blows per 50mm
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100	200	11	5.50
100	300	12	6.00
100	400	11	5.50
100	500	26	13.00
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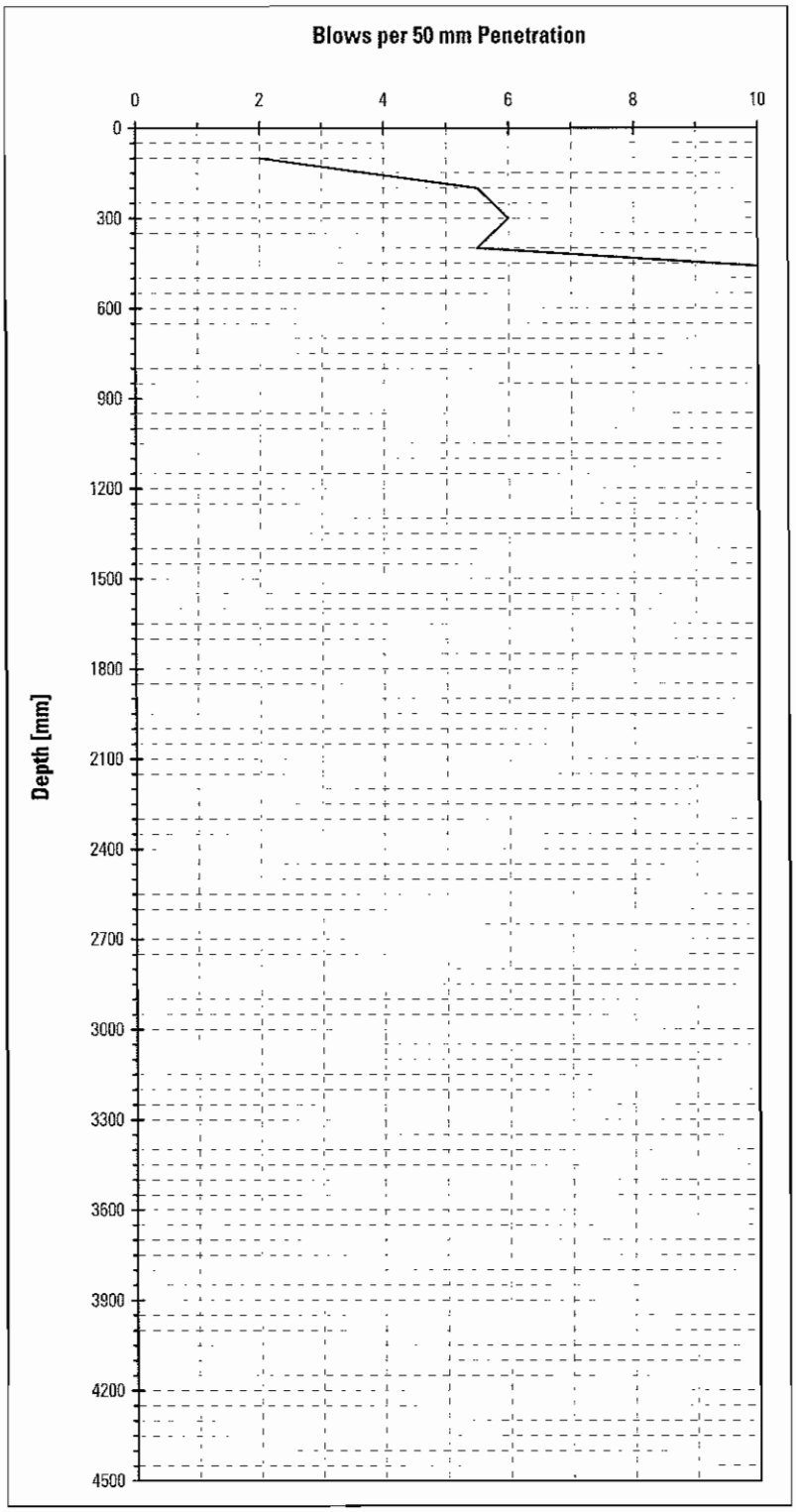

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Plan Change Site		SP 1b	
Tester	Date	Job Number	Rev
James Lynch	25 Jul 2007	209F-003	1

Scala Log Sheet
Kingston Proposed Water Reservoir Location

Depth Meas. (mm)	Cum Depth (mm)	Blows (no.)	Blows per 50mm
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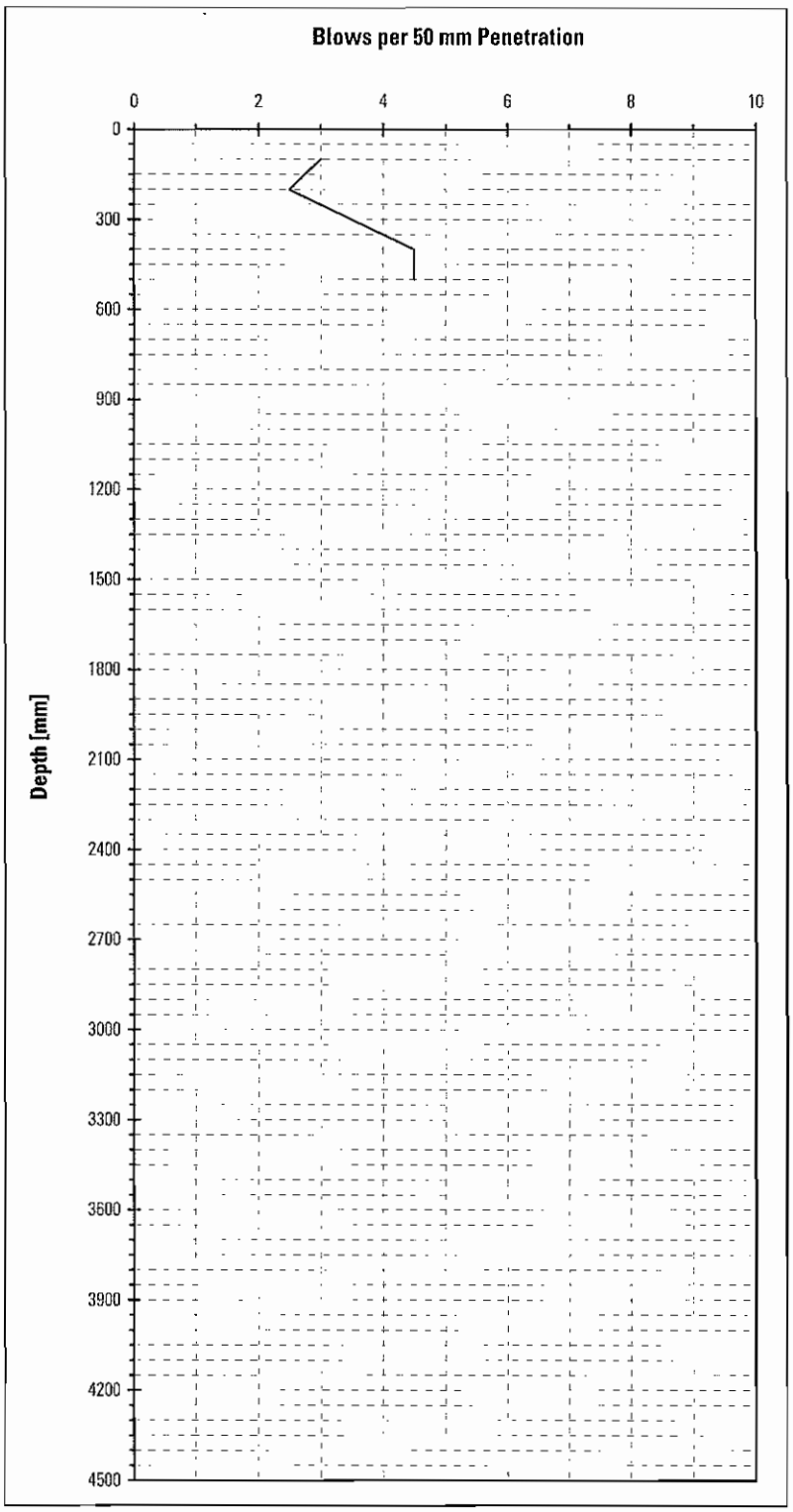

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Location	No		
Plan Change Site	SP 1c		A4
Tester	Date	Job Number	Rev
James Lynch	14 Feb 2008	209F-003	1

Scala Log Sheet
Kingston Proposed Water Reservoir Location

Depth Meas. (mm)	Cum Depth (mm)	Blows (no.)	Blows per 50mm
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100	200	7	3.50
100	300	11	5.50
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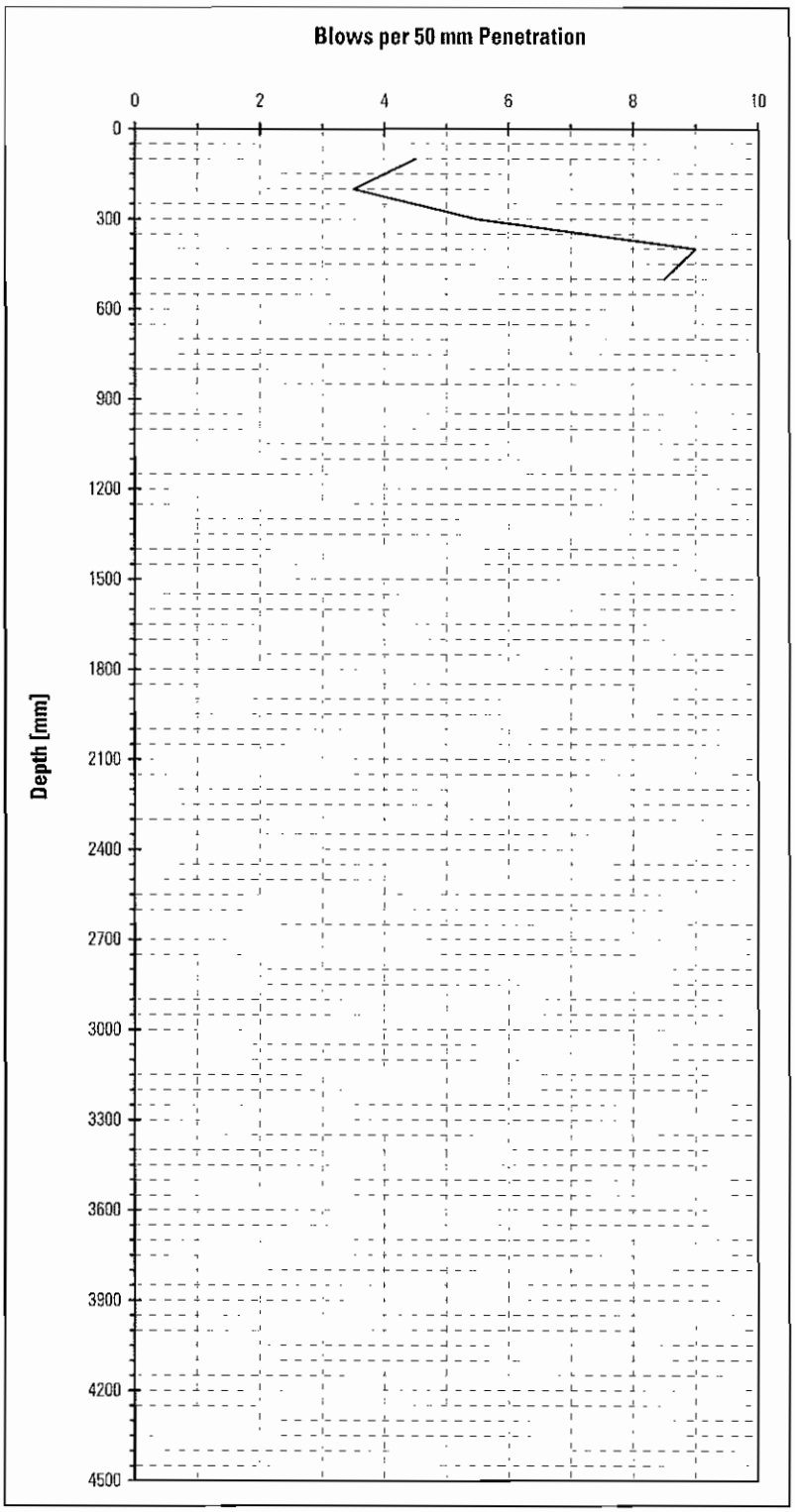

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Tester: James Lynch	Date: 14 Feb 2008	Job Number: 209F-003	Rev: 1

Scala Log Sheet
Kingston Proposed Water Reservoir Location

Depth Meas. (mm)	Cum Depth (mm)	Blows (no.)	Blows per 50mm
100	100	6	3.00
100	200	6	3.00
100	300	10	5.00
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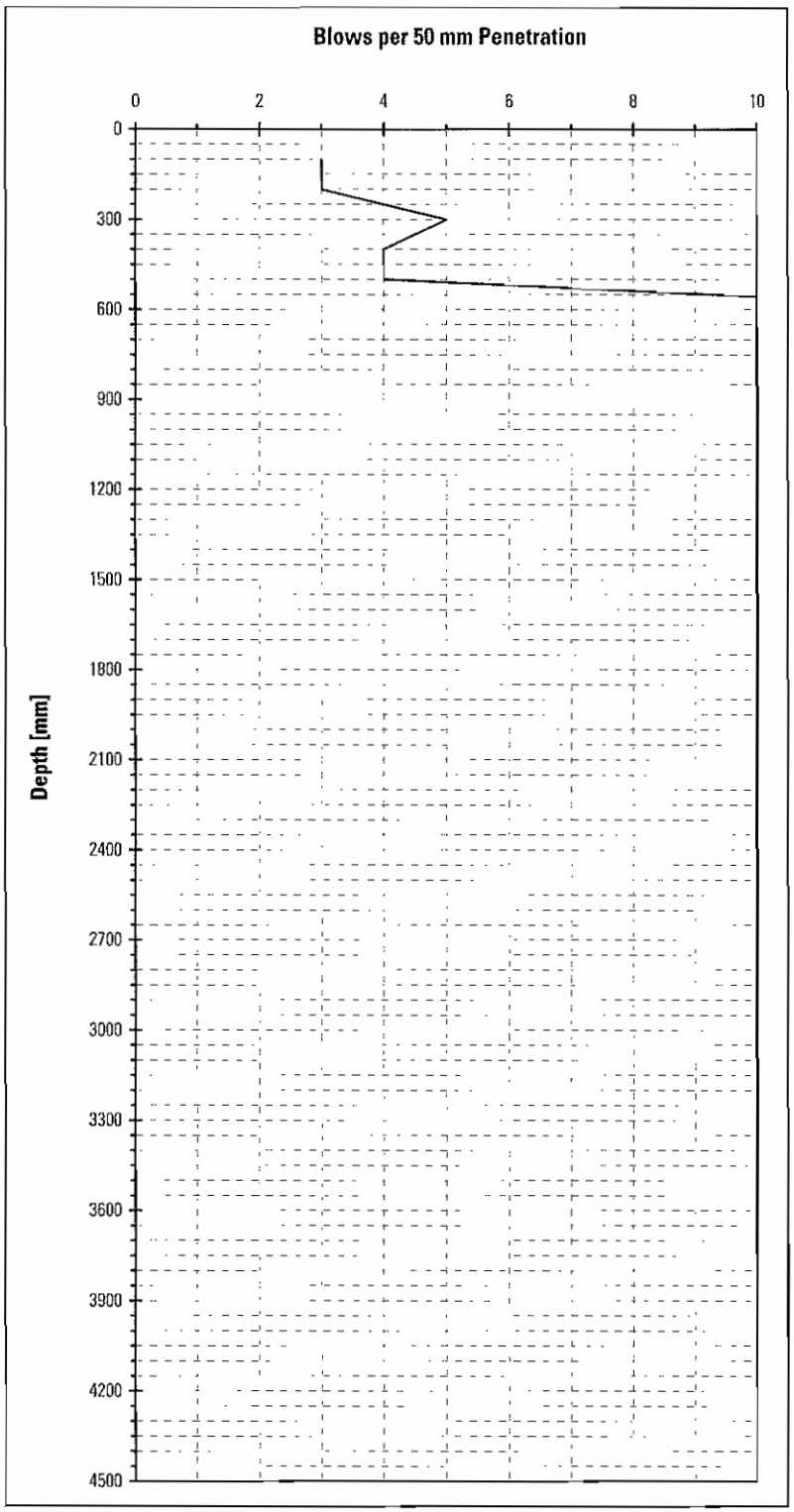

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Tester: James Lynch	Date: 14 Feb 2008	Job Number: 209F-003 Rev: 1

Scala Log Sheet
Kingston Proposed Water Reservoir Location

Depth Meas. (mm)	Cum Depth (mm)	Blows (no.)	Blows per 50mm
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100	200	7	3.50
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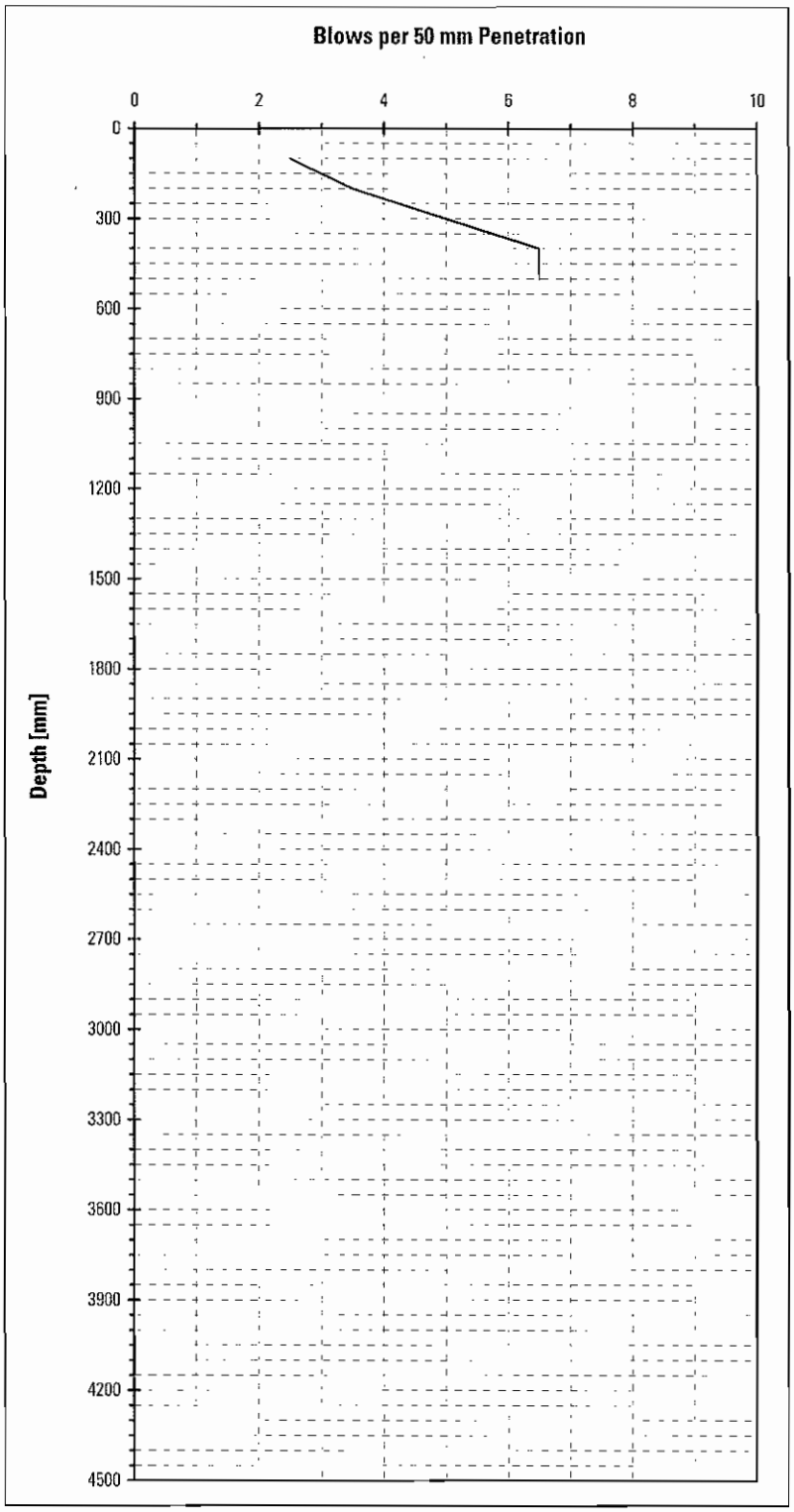
100	500	13	6.50
		End Test	



PROBE DESCRIPTION:
9 kg Hammer falling half a metre to a steel anvil, driving a 16 mm diameter rod fitted with a 20 mm diameter cone.

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Tester: James Lynch	Date: 14 Feb 2008	Job Number: 209F-003	Rev: 1

Scala Log Sheet
Kingston Proposed Water Reservoir Location

Depth Meas. (mm)	Cum Depth (mm)	Blows (no.)	Blows per 50mm
100	100	3	1.50
100	200	5	2.50
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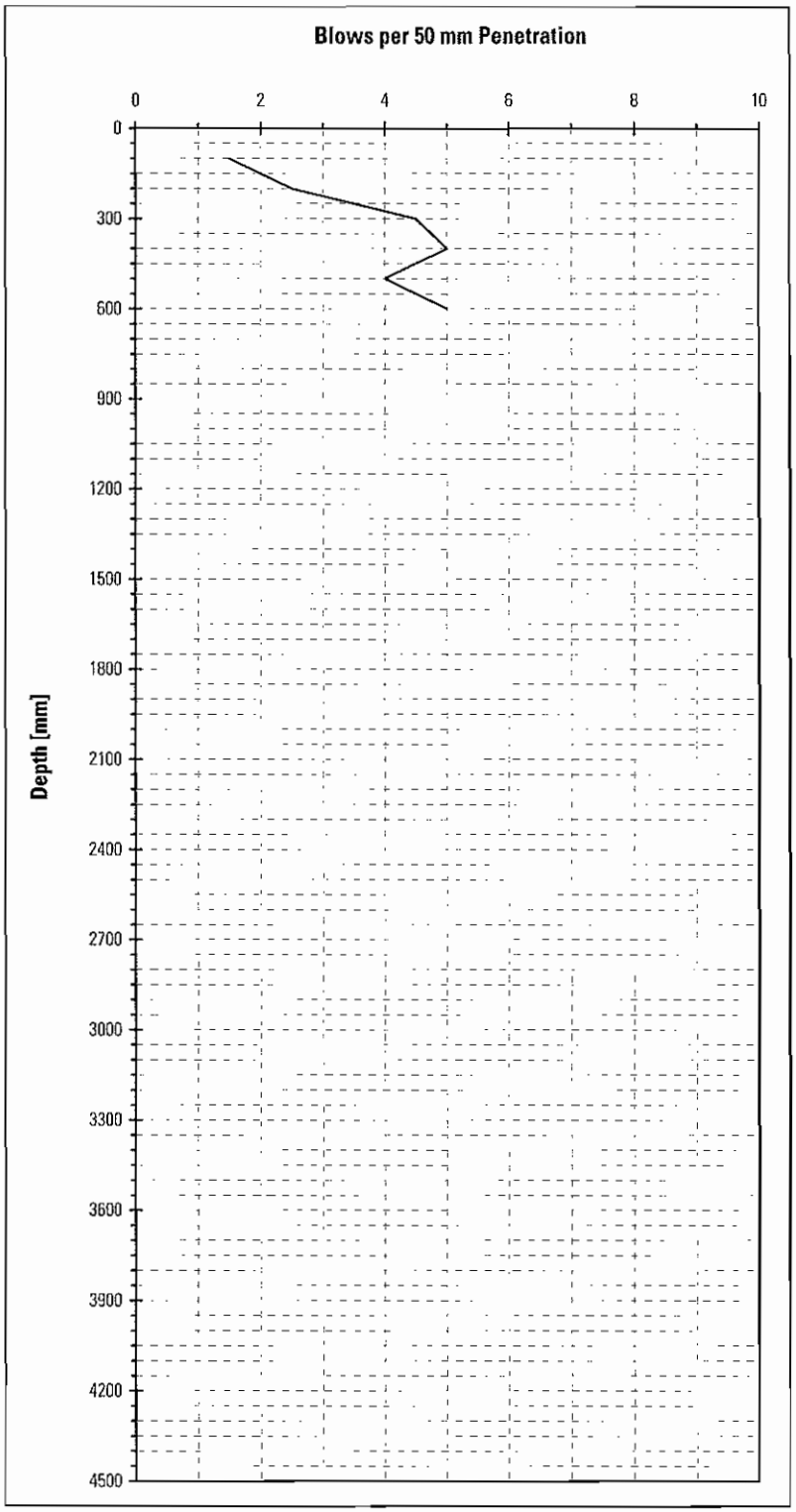

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Tester: James Lynch	Date: 14 Feb 2008	Job Number: 209F-003	Rev: 1

Scala Log Sheet
Kingston Proposed Water Reservoir Location

Depth Meas. (mm)	Cum Depth (mm)	Blows (no.)	Blows per 50mm
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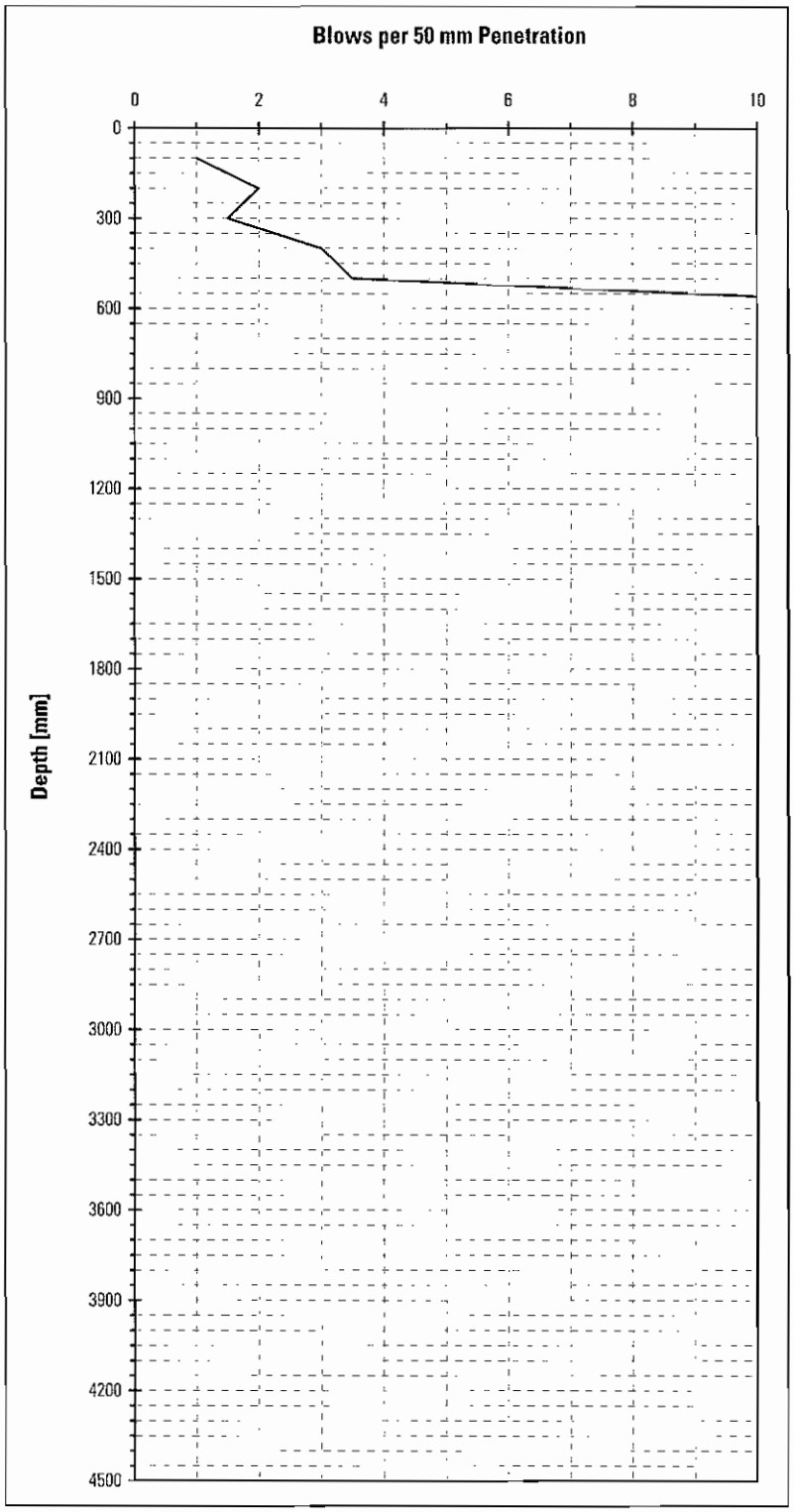

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Tester: James Lynch	Date: 14 Feb 2008	Job Number: 209F-003	Rev: 1

Scala Log Sheet
Kingston Proposed Water Reservoir Location

Depth Meas. (mm)	Cum Depth (mm)	Blows (no.)	Blows per 50mm
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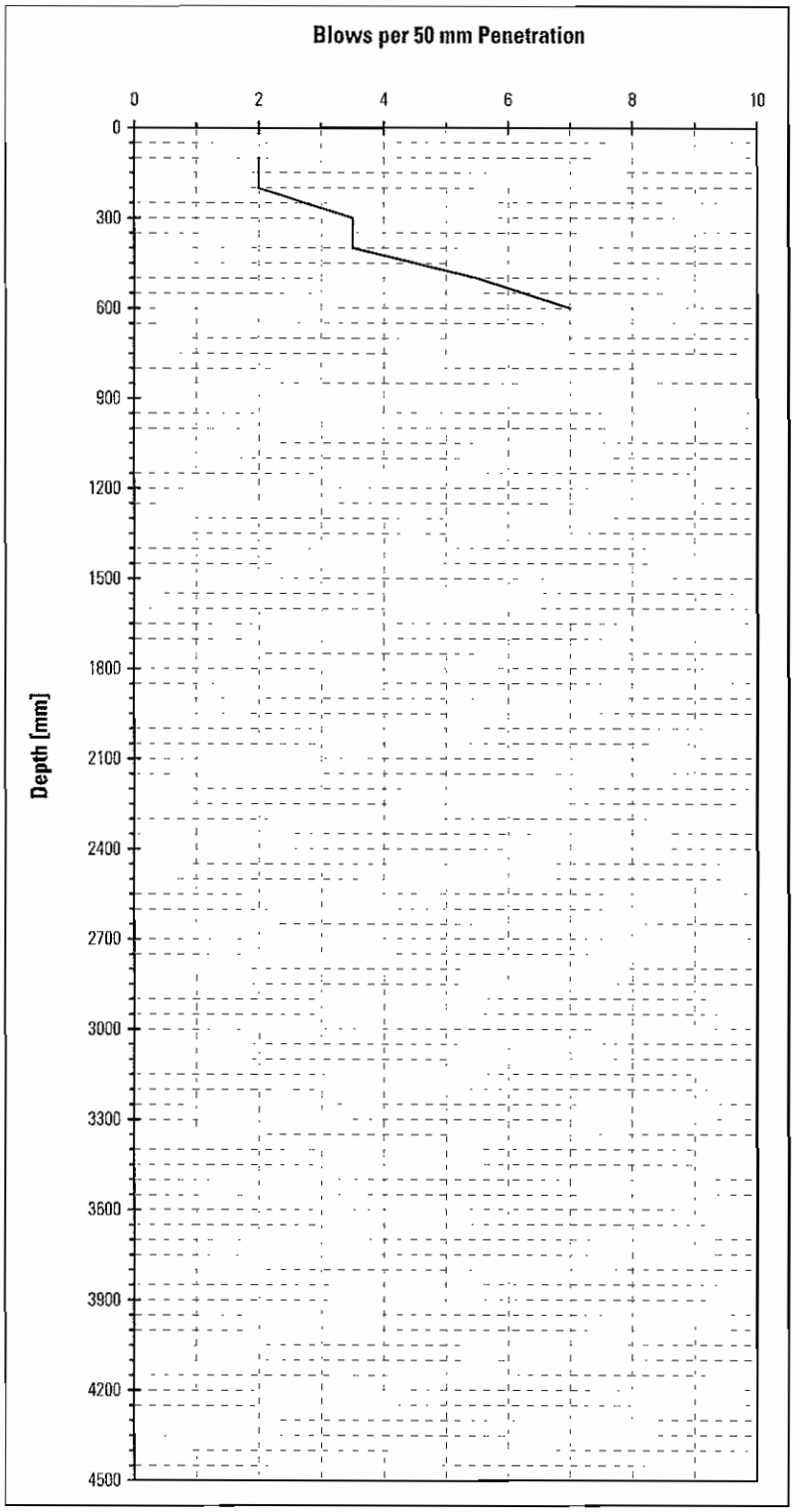
100	600	14	7.00
		End Test	



PROBE DESCRIPTION:
9 kg Hammer falling half a metre to a steel anvil, driving a 16 mm diameter rod fitted with a 20 mm diameter cone.

Connell Wagner

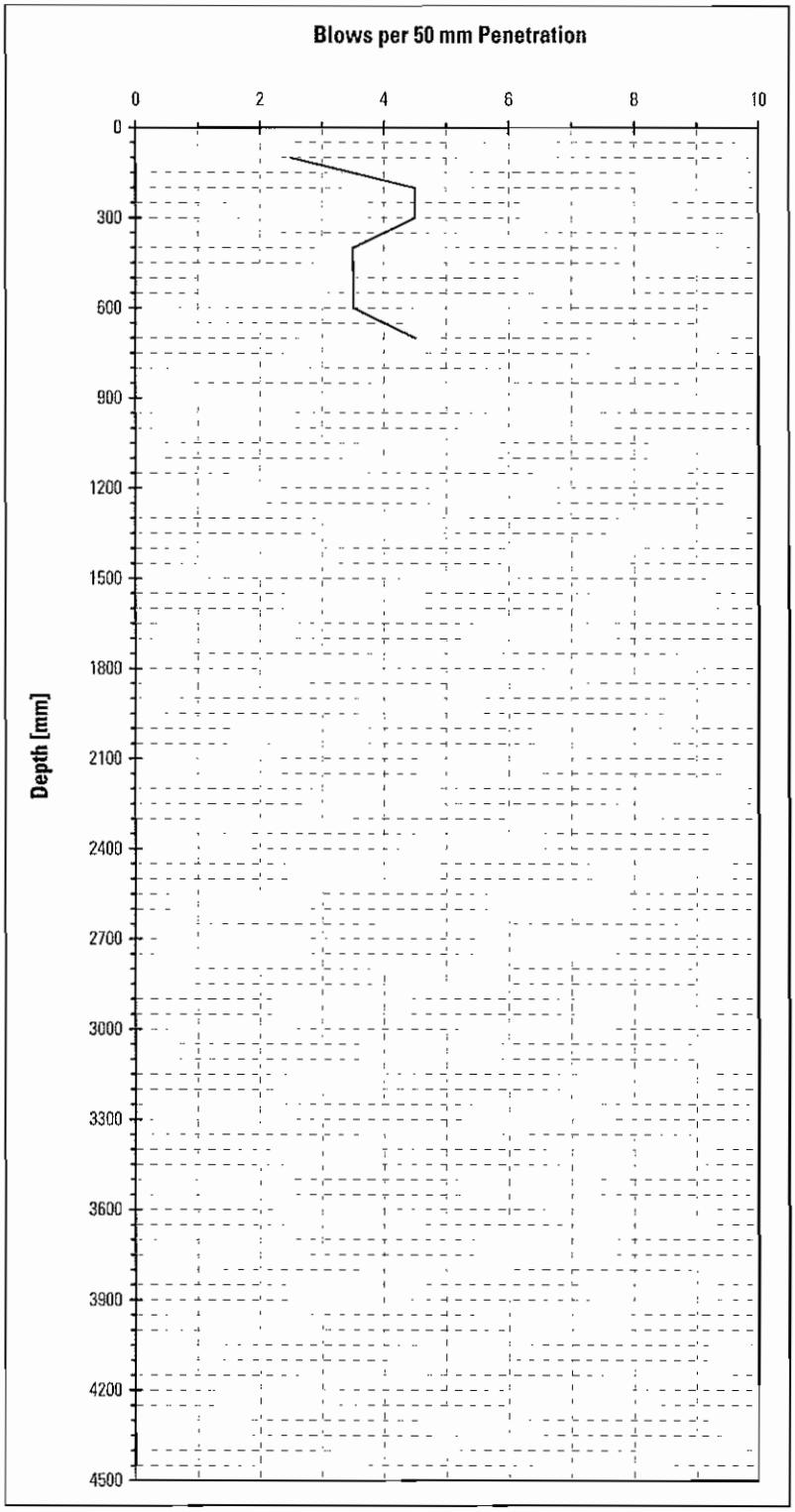
Connell Wagner Limited
1-7 Earl Street (PO Box 1531)
Queenstown
New Zealand

Telephone: +64 3 441 0346
Facsimile: +64 3 442 6180
Email: cwwe@wellconwag.co.nz

Client		SCALA PROBE	
Kingston Village Ltd			
Location	No		
Plan Change Site	SP 2d		A4
Tester	Date	Job Number	Rev
James Lynch	14 Feb 2008	209F-003	1

Scala Log Sheet
Kingston Proposed Water Reservoir Location

Depth Meas. (mm)	Cum Depth (mm)	Blows (no.)	Blows per 50mm
100	100	5	2.50
100	200	9	4.50
100	300	9	4.50
100	400	7	3.50
100	500	7	3.50
100	600	7	3.50
100	700	9	4.50
		End Test	



PROBE DESCRIPTION:
9 kg Hammer falling half a metre to a steel anvil, driving a 16 mm diameter rod fitted with a 20 mm diameter cone.

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Client: Kingston Village Ltd		SCALA PROBE	
Location: Plan Change Site		No: SP 2e	A4
Tester: James Lynch	Date: 14 Feb 2008	Job Number: 209F-003	Rev: 1

Appendix F

Wastewater Scheme Plan

Appendix F

Wastewater Scheme Plan

This appendix contains the following wastewater plan.

- C200 Wastewater Scheme Plan Rev 3

Rev.	Date	Revision Details	By	Ver.	App.
3	13/10/09	ISSUE FOR INFORMATION	CMS	MMF	MMF
2	29/05/08	ISSUE FOR INFORMATION	CMS	MMF	MMF
1	22/02/08	ISSUE FOR INFORMATION	CMS	MMF	MMF

Connell Wagner

Connell Wagner Limited
Old Bank Chambers
102 Customhouse Quay (PO Box 1591)
Wellington New Zealand

Telephone: +64 4 472 9589
Facsimile: +64 4 472 9922
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Client: KINGSTON VILLAGE LTD

Project: KINGSTON VILLAGE

Role	Name	Signed	Date
Drawn	IH	IH	22/02/08
Designed	CMS	CB	22/02/08
Verified	MMF	MMF	22/02/08
Approved	MMF	MMF	22/02/08

Drawing Title: WASTEWATER SCHEME PLAN

CW Project No. 209F

Scale 1:4000 (AT A1) 1:8000 (AT A3)

Drawing No. C200 Rev. 3

Appendix G

Water Treatment Products

Appendix G

Water Treatment Products

This appendix contains details on the following water treatment products.

- UV Treatment
- Industrial Softeners
- Multimedia Filters
- pH Neutralising Filters

TROJAN UV SWIFT™ SC

Trojan Technologies

Trojan UVSwift™SC

Technology you can trust from the industry leader

Trojan Technologies Inc. is an ISO 9001 registered company and for more than 25 years has set the standard for proven UV technology and ongoing innovation. With unmatched scientific and technical expertise, a dedicated network of specialists, representatives and technicians, Trojan is trusted more than any other firm as the best choice for municipal UV solutions – worldwide.

The Trojan UVSwift™SC (Small Community) is one of the reasons why. Ideally suited for applications up to 1.5 MGD (240 m³/h), this compact, robust UV system offers small communities an efficient, economical solution for drinking water disinfection of surface water or groundwater sources. Like all Trojan drinking water products, the UVSwift™SC is bioassay validated, having undergone rigorous DVGW certification to ensure verified dose delivery, maximum public safety and peace of mind. It's engineered and built to provide reliable performance, simplified maintenance, and reduced operating costs with innovative features like a hydraulically optimized, "L-Shaped" reactor, high intensity amalgam lamps and optional automatic or manual sleeve wiping.

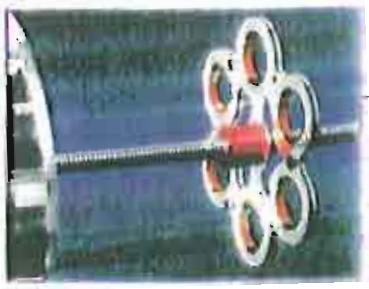
Trojan UVSwift™SC *with optional automatic sleeve wiping system.*

TROJAN UVSWIFT™ SC

Designed for maximum performance

Amalgam Lamps

Utilizes 2 to 12 low-pressure, high-output amalgam lamps. Each is located within its own protective quartz sleeve and supported by a removable, sleeve holder assembly. Designed for easy lamp replacement.



Sleeve Wiping System

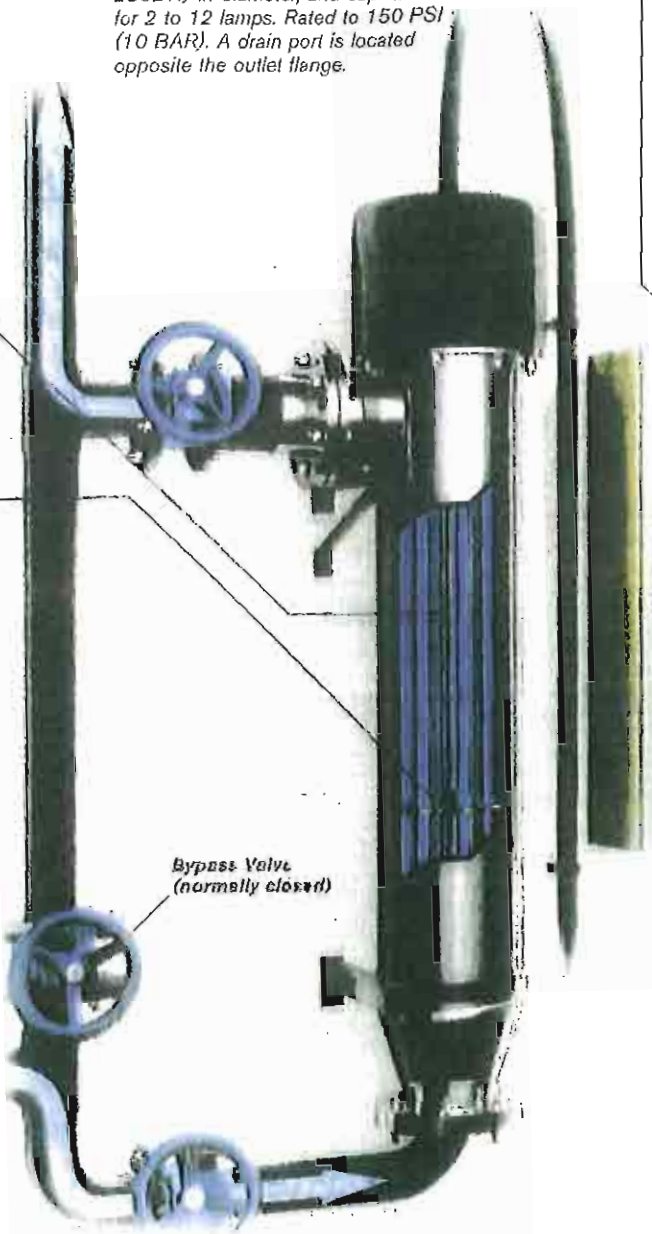
Optional manual or automatic systems available; both operate online, without interrupting disinfection. EPDM wipers are mounted in stainless steel yoke around the quartz sleeve of each lamp. The manual system is driven by hand using an external handle. The automatic system allows cleaning at preset intervals using a motor driven wiper assembly.

UV Sensor

Highly accurate, DVGW approved, photodiode sensor monitors UV output within the reactor. Mounted within the sensor port on the side wall of the reactor for easy access.

UV Reactor

Electropolished Type 316L stainless steel. Can be installed vertically or horizontally. Reactor configurations are available with inlet/outlet sizes ranging from 3" to 10" (80DN to 250DN) in diameter, and capacities for 2 to 12 lamps. Rated to 150 PSI (10 BAR). A drain port is located opposite the outlet flange.



Control Panel (CP)

Epoxy-painted, carbon steel cabinet is designed for indoor, wall-mount installation. Houses a microprocessor based controller with I/O connection points, and electronic power supplies. Distributes power to the UV reactor as well as the UV sensor and optional automatic wiping system. UV intensity, lamp elapsed time and lamp status are continuously monitored and displayed on the operator interface, located on the control panel door.

Remote & Automatic Operation

Standard input/output signals include a Remote Power Control to permit on/off control of the system from a remote location.
Upon alarm condition the controller can send signals to operate pumps and valves.
The UV Intensity Analog Output option allows remote monitoring of the UV intensity. The system also features a high temperature alarm.

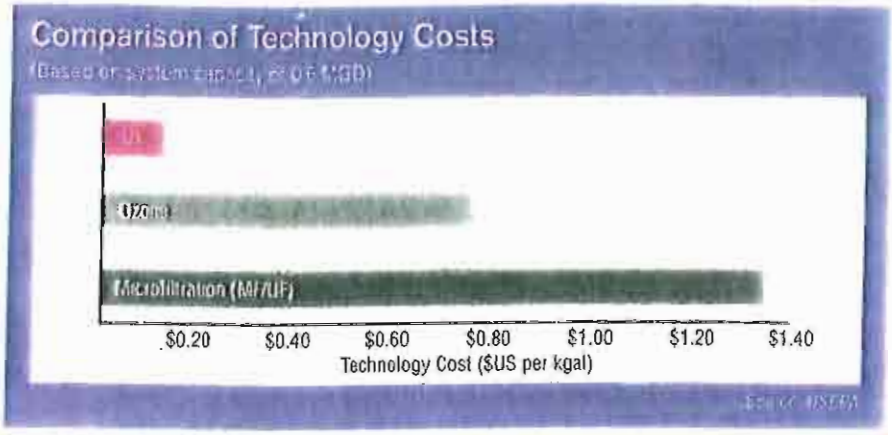
Extremely Cost-Effective to Install and Maintain

Proven UV protection at a fraction of the cost of other technologies

Benefits:

- Affordable, DWGW validated, disinfection is available to small communities
- Capital costs of the UVSwift™SC range from 2¢ to 7¢ US per US gallon of installed capacity
- O&M costs range from 0.5¢ to 3¢ US per thousand US gallons treated

- A cost comparison by the USEPA (1996) demonstrated that, for a dose of 40 mJ/cm², UV was cost-effective compared to ozonation and chlorination over a flow range of 0.024 to 3.8 MGD (4 to 784m³/hr)



The capital costs of UV disinfection are a fraction of those of competing technologies, making the Trojan UVSwift™SC an economical way for small communities to safeguard their drinking water.

- Unlike chemically-based disinfection options, the Trojan UVSwift™SC doesn't have the added expenses associated with accident insurance or specially ventilated storage facilities as protection against gas leaks or liquid spills

Energy Efficient Amalgam Lamps

Need for fewer lamps reduces capital and O&M costs

Benefits:

- Fewer lamps are needed to deliver the required dose, compared to traditional low-pressure lamps
- With fewer lamps, the system can be located in compact spaces, reducing installation costs
- Trojan amalgam lamps draw less energy than competitive high-output systems

- Trojan's amalgam lamps produce significantly higher UV output than conventional low pressure lamps. Thus fewer lamps are required to achieve the same level of disinfection
- Conventional low-pressure lamp systems may require two to three times more lamps than the Trojan UVSwift™SC
- The hydraulically efficient Trojan UVSwift™SC reactor can accommodate from 2 to 12 lamps
- The lamps are sealed inside heavy-duty quartz sleeves by Trojan's multi-seal system, maintaining a watertight barrier around the internal wiring while individually isolating each lamp
- Lamp changeovers are fast and simple – replacing a lamp takes about five minutes, and requires no tools
- Lamps are pre-heated for reliable startup, and prolonged life

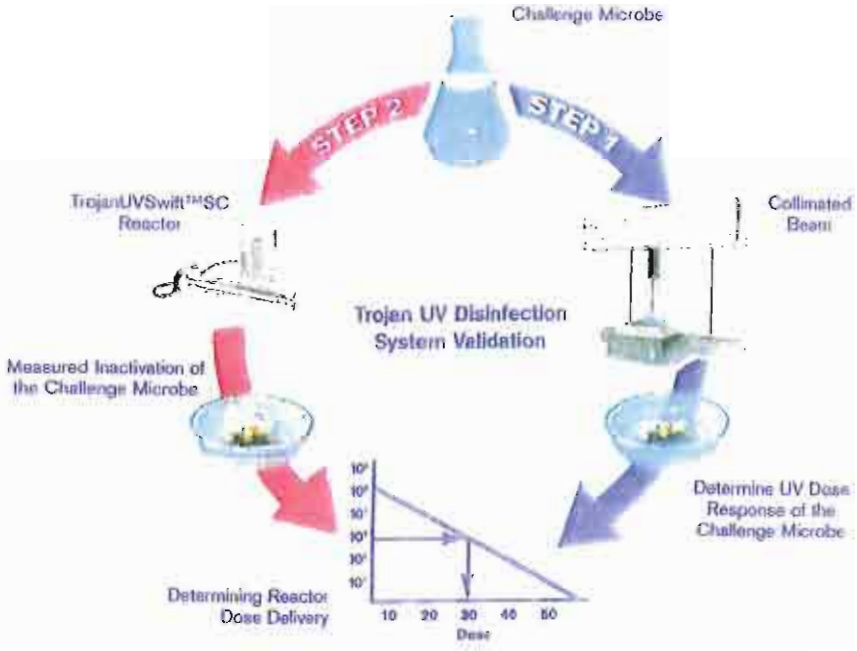
Trojan's amalgam lamps generate stable UV output and are very efficient in converting electrical energy to UVC.

DVGW - Bioassay Validated Performance

Real world testing ensures that regulatory targets are met at all operating conditions

Benefits:

- The stringent standards of Deutsche Vereinigung des Gas und Wasserfaches e.v. - German Association of Gas and Water (DVGW) are recognized internationally
- Performance data is generated from real-world testing at a dose of 40 mJ/cm² over a range of flows using the DVGW W294 standard
- Ensures maximum public safety by providing physical verification that system will perform as expected
- All Trojan UVSwift™SC units are certified for source water of various qualities, having been DVGW bioassay tested under a range of UV transmittances
- Provides accurate assessment of equipment sizing needs



Trojan UVSwift™SC systems installed under real-life conditions have been challenged using live organisms.

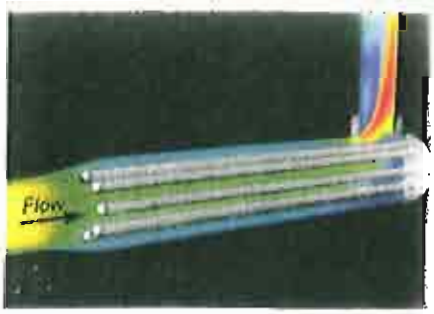
- Bioassay validation offers peace of mind through verified dose delivery, not theoretical calculations that can lead to inaccurate assessments of system sizing and delivered dose
- Many competitive systems have been validated for only one transmittance level, typically 95-99% UVT. As a result, such systems are not certified for use at other UVT levels
- The performance of the Trojan UVSwift™SC system was evaluated at a number of flow levels under the worst-case orientation – with a 90° elbow at the inlet
- The U.S. Environmental Protection Agency (USEPA) has endorsed bioassays as the standard for assessment and comparison of UV technologies

Optimized Hydraulics Maximize Performance

Reactor design maximizes efficiency and reduces headloss

Benefits

- Computational Fluid Dynamic (CFD) modeling has resulted in an innovative "L-Shaped" reactor which is twice as efficient as a conventional "U-Shaped" reactor
- Significantly increases reactor efficiency which permits the system to use fewer lamps - thus reducing O&M costs
- Permits vertical or horizontal reactor installation to minimize space requirements and installation costs
- Reactor design minimizes headloss eliminating the need for additional pumps and the associated capital and operational costs



Trojan's innovative "L-Shaped" reactor reduces hydraulic interruptions thus increasing the efficiency of the system.

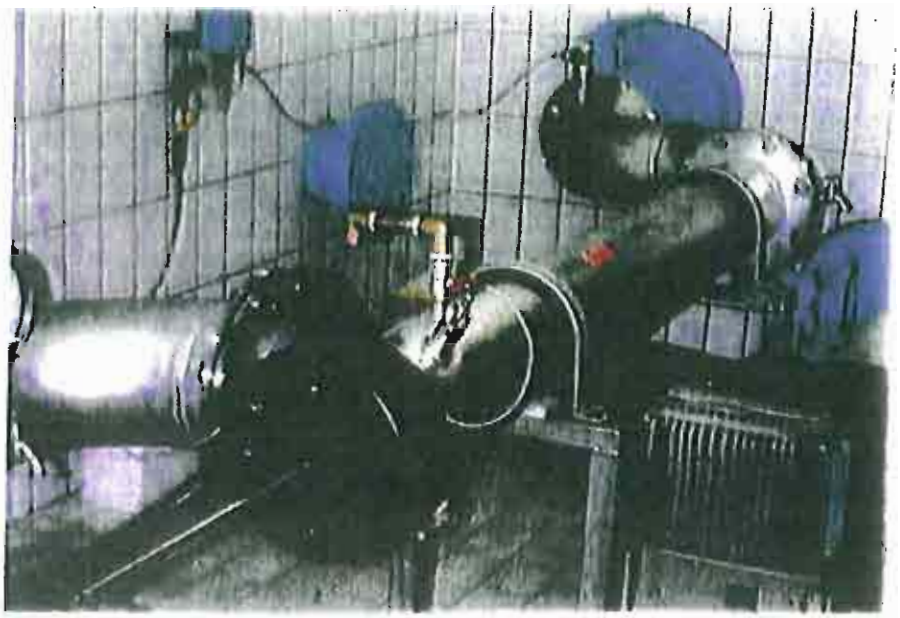
Compact Footprint Provides Installation Flexibility

Cost-saving design can be installed vertically or horizontally

Benefits:

- **Small footprint enables easy installation and retrofitting into small spaces and pipe galleries**
- **Stainless steel reactor can be installed vertically or horizontally for maximum flexibility, with no reduction in disinfection performance**
- **Small reactor size reduces the requirements and costs for buildings or expansion of existing facilities**
- **Compact, wall-mounted control panel can be located up to 16' (5m) from the reactor**
- **Reactor can be installed in a vertical orientation virtually eliminating footprint requirements**

- Small size of reactor and control panel permits running multiple units in parallel
- Clearance for maintenance and lamp changeovers is required at only one end of the reactor, thus allowing orientation of the input port against walls or at floor level
- The Type 12 (IP54) Control Panel houses the controller, and electronic ballasts in a small, wall-mountable cabinet



With its small footprint and installation flexibility, the system can be designed with units running in parallel to meet capacity or redundancy requirements. Shown with optional manual wiping system.

The compact Trojan UVSwift™SC system can easily be accommodated in very restrictive spaces, and can be mounted vertically or horizontally. Shown with optional automatic wiping system.

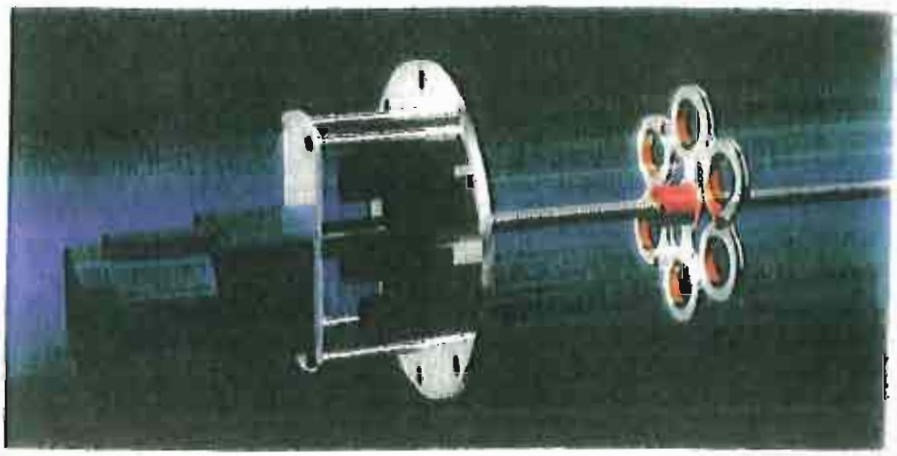
Sleeve Wiping System Ensures Maximum UV Dose Delivery

Optional manual or automatic wiping simplifies routine maintenance

Benefits:

- Allow amalgam lamps to deliver full UV dose for maximum disinfection, public safety and peace of mind
- Easy maintenance by preventing fouling from accumulating on quartz lamp sleeves
- Reduce the frequency, inconvenience and cost of manual chemical cleaning
- Operate online while the lamps are disinfecting – there's no need to shut down the reactor or bypass the lamp modules for routine wiping
- Automatic wiping system can be programmed to wipe lamp sleeves at preset intervals

- Wiping systems can be retrofitted to Trojan UVSwift™SC systems after installation

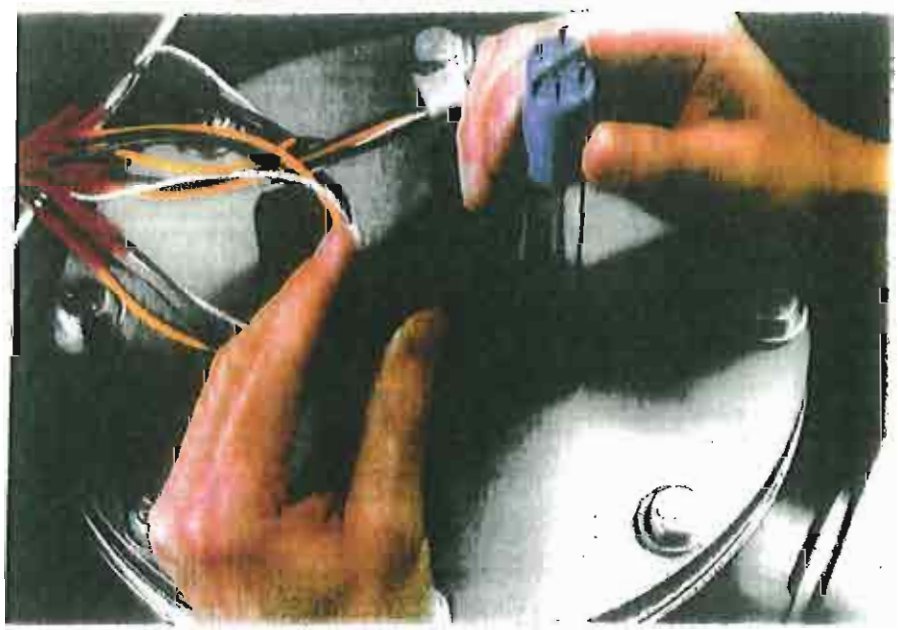


The optional wiping systems reduce maintenance costs. Operators have a choice of the manual system that is operated by hand, or the automatic, motorized system (shown above) which can be programmed to wipe at preset intervals with no operator involvement.

Designed for Easy Maintenance

Operator-friendly design simplifies routine maintenance

- Fewer lamps are required, thus reducing associated maintenance
- Single-ended lamp design simplifies lamp changeovers and eliminates the need for tools
- Annual lamp replacement is fast and does not require the reactor to be drained. Each lamp takes only five minutes to change.
- Sensor mounted on outside of reactor for easy access
- Control Panel can be mounted up to 15' (5m) from the reactor
- Optional wiping systems reduce the frequency of manual chemical cleaning



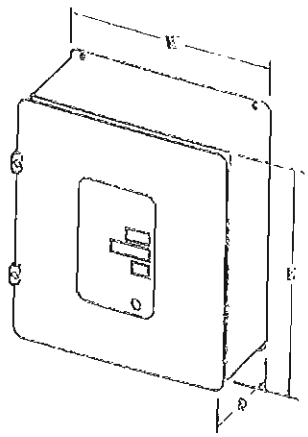
The Trojan UVSwift™SC design simplifies maintenance procedures. For example, lamp changeovers require no tools and take only about five minutes per lamp.

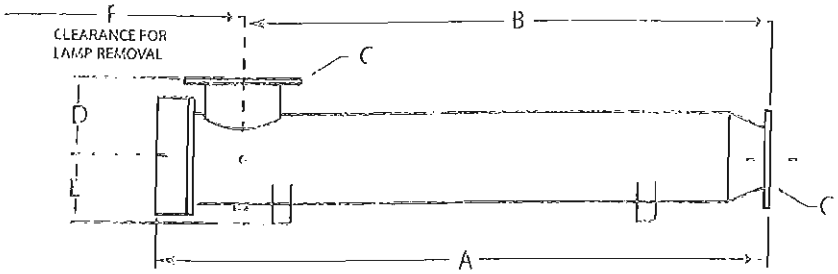
TROJAN UVSWIFT™ SC

System Specifications

Model #		A03	B03	B04	B05	B08	C06	B12
System Characteristics								
Maximum Disinfection Flow Rate (95%UVT 40 mJ/cm²): GPM (m³/hr)		49 (11)	118 (27)	162 (37)	256 (58)	278 (63)	493 (112)	821 (187)
Number of Lamps		2	3	4	5	8	6	12
Electrical Requirements								
50 Hz, Single Phase 120V, 2 wire + ground		✓						
50/60 Hz, Single Phase 208-240V, 2 wire + ground		✓	✓	✓	✓	✓	✓	✓
50/60 Hz, 277V/480V, 3 Phase, 4 wire + ground			✓	✓	✓	✓	✓	✓
Nominal Power (watts)		260	503	650	950	1250	1650	1900
Nominal Power (VA) Single Phase		275	522	682	992	1312	1627	1997
Ballast Type		Electronic Constant Output	Electronic Constant Power					
Control Panel								
Dimensions	inches	16x14x6	24x16x8	24x16x8	24x20x8	24x24x8	24x20x8	36x36x12
	(cm)	(41x36x16)	(61x42x20)	(61x42x20)	(61x51x20)	(61x61x20)	(61x51x20)	(91x91x30)
Rating		Type 12 (IP54)						
Wiper Chamber (Engineered Materials/Options)								
Materials of Construction, Stainless Steel		316L (1.4404 / Europe)						
Max Operating Pressure PSI (BAR)		150 psi (10)						
Max Fluid Temp °F (°C)		120 (50)						
Sleeve Cleaning Mechanism, Options		Manual Only	Manual or Automatic	Manual or Automatic	Manual or Automatic	Manual or Automatic	Manual or Automatic	Automatic Only
Dimensions - Inches (cm)								
A: without auto wiper		35 (89)	60 (127)	60 (127)	60 (127)	50 (127)	72 (183)	60 (152)
A: with auto wiper		N/A	60 (152)	60 (152)	60 (152)	60 (152)	80 (203)	60 (152)
B:		29 (74)	43 (109)	43 (109)	42 (107)	42 (107)	62 (158)	375 (95)
C:		3 (80DN)	4 (100DN)	4 (100DN)	6 (150DN)	6 (150DN)	6 (150DN)	10 (250DN)
D:		6 (15)	8 (20)	8 (20)	8 (20)	8 (20)	8 (20)	14 (36)
E		6 (15)	8 (20)	8 (20)	8 (20)	8 (20)	8 (20)	12 (30)
F:		40 (102)	48 (122)	48 (122)	48 (122)	48 (122)	74 (188)	48 (122)

* A: 59.48 (15) on Bronze Automatic Wiper
A: 60 (203) on Chrome Automatic Wiper





Find out how your drinking water system can benefit from Trojan UVSwift™SC - call us today

Trojan Technologies Inc.

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London, Ontario
Canada N5V 4T7
Telephone: (519) 457-3400
Fax: (519) 457-3030

www.trojanuv.com

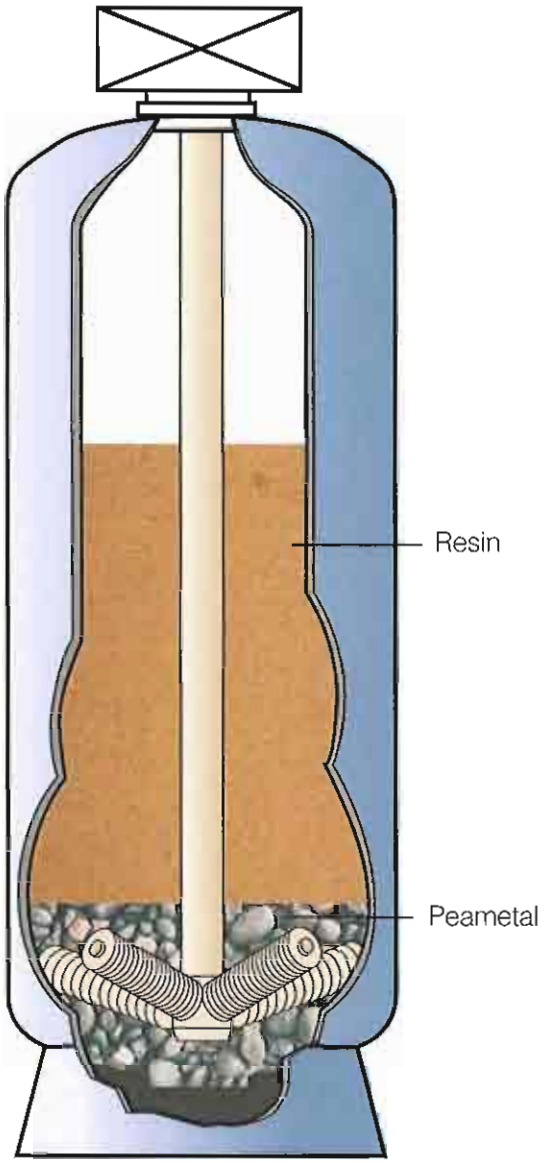
Trojan Technologies is a publicly traded company on the Toronto Stock Exchange under the symbol TUV.

Industrial Water Softeners

...AquaFlo automatic water softeners are available in a versatile range of seven standard models, capable of outputs up to 30 m^3/hr of softened water for almost any requirement. Larger designs by request.

Water softeners can be utilised in a large variety of commercial and industrial applications to prevent the formation of hard water scale deposits, (calcium and magnesium), remove some undesirable dissolved minerals and remove low levels of iron and manganese. These contaminants can result in unnecessary plant down time, excessive maintenance costs and increased operating costs (fuel, labour, chemicals, etc.)

By using an AquaFlo Water Softener these problems can be overcome. To do this the water is passed though a bed of ion exchange resin and as the water passes through, the calcium and magnesium ions are absorbed by the ion exchange resin and in return sodium ions are released. The result is softened water. Other minerals can be removed in a similar manner. This process continues until all the sodium ions are exchanged for calcium or magnesium, at which point the resin will then become exhausted. In order to obtain a further supply of softened water, the resin must be regenerated and this is achieved automatically by passing a solution of brine through the resin to remove the hardness minerals.



Features

AquaFlo systems are robust, corrosion resistant designs with a five cycle automatic control valve and glass fibre reinforced pressure vessel. The brine tank is polyethylene.

Applications

- *Boilers and Heating Systems*
- *Circulating Cooling Systems*
- *Laundries*
- *Food and Beverages*
- *Cosmetics and Pharmaceuticals*
- *Industrial Processes*
- *Resorts and Caravan Parks*
- *Hotels, Motels and Home Units*
- *Hospitals, Laboratories and Schools*
- *Renal dialysis units*

Control Valve Options

Magnum / 962

180 / 440i

Performa

GENERAL SPECIFICATIONS

Fully automatic softeners with Autotrol valves giving timer or demand regeneration with turbine meter and microprocessor control to minimise running costs. System includes valve, controller, fibreglass resin tank and brine tank. For example SF330A is autobackwashing based on time, while the SF330C or D is autobackwashing based on volume of water used. The SF330D-96TW is a highflow/twin alternating tank system which offers either duty, standby or highflow with both tanks working together.

Pressure Vessels

Wound resin bonded fibreglass filament for strength and polyethylene inner shell for chemical inertness.

Working Pressure 200 - 1040 kPa. (30 - 150 psi)
Operating Temperature 5 - 40°C maximum

Valves*

Autotrol Performa, 180, Magnum or 172.

Controllers

- OPTION A - a standard 440i, 740 or 942 day timer for either daily or alternate day backwash. These are less economical as they regenerate regardless of water usage.
- OPTION C - Auto-regeneration by water usage with 460i or 760 controller.
- OPTION D - Auto-regeneration by water usage with 962 controller which provides additional programming and control of reserve capacity, plus flow monitoring.

Instructions

Installation and Operating Instructions are included with all Water Softeners.

Resin

High quality Rohm & Haas IR120NA resin is used in all AquaFlo Water Softeners.

Pipe Work

Standard pipe work, where fitted, connecting filter vessel to valving is rigid PVC. Galvanised pipe is available as an option.

Installation

Any regulatory requirements pertaining to plumbing and drainage connections are the responsibility of the client. Plant should be installed under cover protected from direct sunlight, rain and frost.

Additional Options

- *Twin Alternating or Highflo configuration available for continuous, uninterrupted soft water.*
- *Skid Mounted Plant*
- *Water meter initiated regeneration with batch register and automatic reset.*
- *Automatic shut-off of hard water bypass during regeneration.*
- *Flow monitors or flow totalisers*
- *Micro switches for regen alert/pump control*

Client to Provide:

- Single phase 230 volt power.
- Drain for backwash.
- All connecting pipe work and valving from control unit.
- Minimum 200 kPa (30 psi) operating pressure
- Water flow or pump operation at 2.00 am to allow for Automatic regeneration.

** Filtec reserves the right to offer an alternative valve(s) depending on customer requirements.*

SYSTEM SPECIFICATIONS

Model	Flow Rate (m^3/hr)			Valve Controller	Inlet/ Outlet (mm)	Drain (mm)	Approx. Softening Capacity kg $CaCO_3$	Resin Volume (Litres)	Diffrential Pressure (kPa)		Softener Size dia x h (mm)	Brine Tank base (mm)	Salt/ Regen (kg)
	Norm	Peak	B/Wash						Norm	Max			
SF300	3	4	1.0	255	25	15	4	75	105	170	300 x 1400	410[2]	13
SF330	4	5.5	1.2	268	25	20	6	100	105	170	330 x 1850	410[2]	15
SF400	5.5	6	1.6	268	25/40	25	7.5	125	105	170	400 x 1850	Ø900	19
SF500	10	14	2.2	180	50	40	12	200	105	170	500 x 1850	Ø900	30
SF600	14	17	2.9	Magnum	50	40	18	325	105	170	600 x 1850	Ø900	48
SF750	17	27	4.7	172	50	50	26	425	105	170	750 x 2m	Ø1000	72
SF900	23	30	6.8	172	50	50	36	575	105	170	900 x 2m	Ø1000	90

A minimum 200 kPa (30 psi) operating pressure is required. *Larger units are available.*

For more information contact...

Filtration Technology Ltd
PO Box 58-853, Greenmount, Auckland.
Auckland: Tel: 0-9-274 4223, Fax: 0-9-274 4224
Wellington: Tel: 0-4-232 2402, Fax: 0-4-232 2403
Dunedin: Tel: 0-3-488 6200, Fax: 0-3-488 6300
Website: www.aquaflo.co.nz

Local Agent:

AquaFlo

Multimedia Sand Filters

...for effective removal of sediment in your water supply system for house & farm

Sediment (or grit and turbidity) in your water supply may cause problems when it settles out in tanks, or plugs pipes and fittings. Household and plumbing fixtures can be stained, while washing machines, dishwashers or hot water cylinders may be permanently damaged.

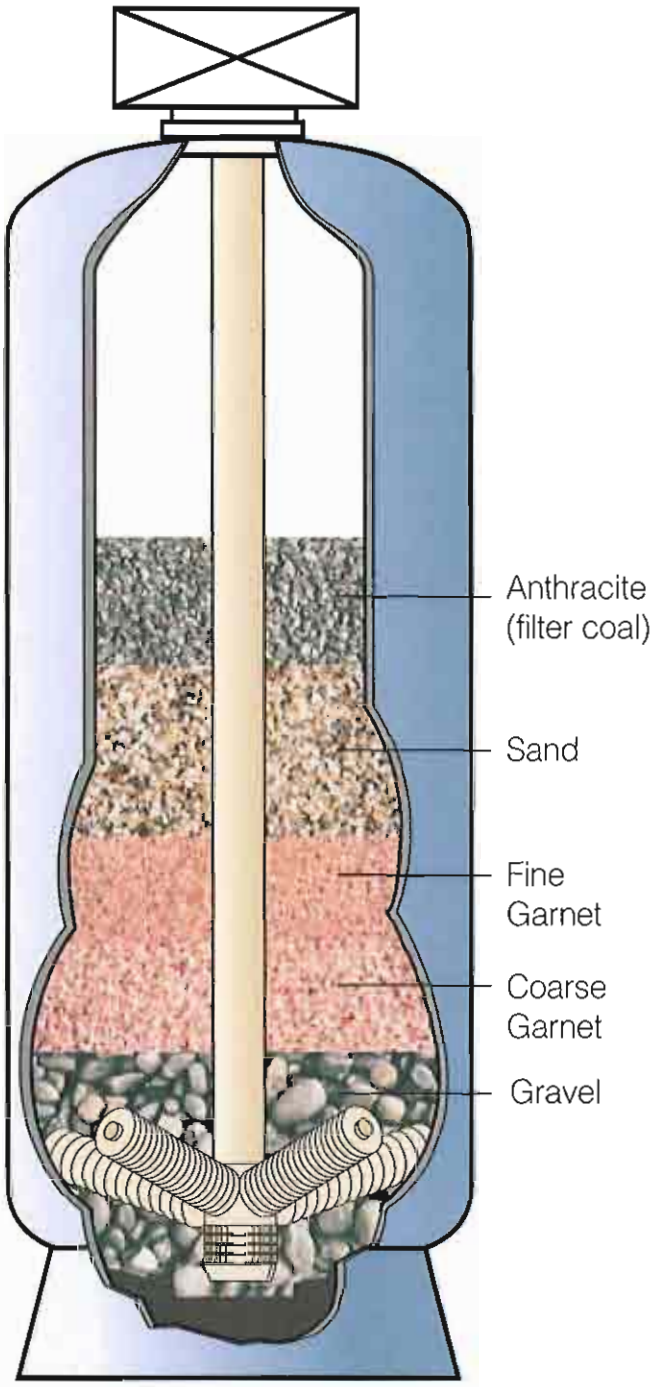
An AquaFlo Multimedia Sand Filter is the best solution for removing visible solids (and thus turbidity), so you have clear water which won't damage household appliances.

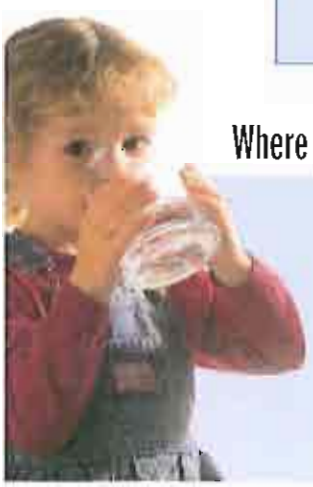
The filter consists of progressively finer layers of anthracite, sand, garnet and a gravel support layer. As the supply water travels through the filter bed, the suspended solids are removed (from 10 - 15 micron in size). The filter bed is maintained in optimum condition by regular backwashing (either automatically (recommended), or manually), which simply reverses the flow through the filter flushing out accumulated dirt and reconditioning the filter material. The media lasts for many years so maintenance and running costs are very low.

"AquaFlo Multimedia Sand Filters are recommended for surface water supplied containing unacceptable levels of suspended solids and discoloration."

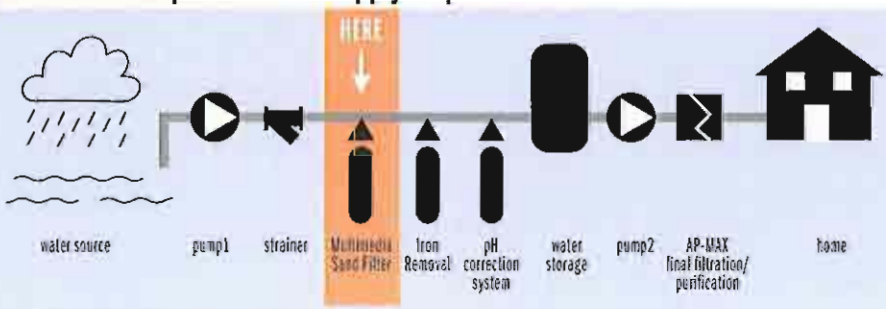
- *High flowrate GRP reinforced polyethylene pressure vessels*
- *Automatic or manual backwash*
- *Filtration down to 10 - 15 micron*
- *Flows up to 23 m³/hour per unit* with 8 models*

** Multi-unit options available for larger flow requirements*





Where you are in the AquaFlo water supply sequence...



GENERAL SPECIFICATIONS

Pressure Vessels
Wound resin bonded fibreglass filament for strength and chemical inertness.
Working Pressure 200 - 700 kPa.
Operating Temperature 5 - 40°C maximum

Valves*

- Autotrol Series Performa 263 &273 Noryl Control valve for small to mid size plant
- Autotrol Series Magnum (Mag) multiport Noryl valve for larger capacity plants.
- Manual head is available for all models. Replace "A" in model number with "M".

Controllers

Electrically operated 6 or 7 day time clock driving control valve as standard.
Optional volume based control available.

Instructions

Installation and Operating Instructions are included with all softeners.

Media

High quality graded sands, garnets, gravels, carbon and other media as appropriate for the application.

** Filtec reserves the right to offer an alternative valve(s) depending on customer requirements.*

Pipe Work

Standing pipe work, where fitted, connecting filter vessel to valving is rigid PVC. Galvanised pipe is available as an option.

Installation

Any regulatory requirements pertaining to plumbing and drainage connections are the responsibility of the client. Plant should be installed under cover protected from direct sunlight and rain.

Client to Provide

- Single phase 230 volt power.
- Drain for regeneration.
- All connecting pipe work and valving from control unit.
- Minimum 200 kPa (30 psi) operating pressure (Note Manual 20 kPa.)

Cautions:

- *A Pressure Reductio Valve should be installed in areas of high water pressure (over 300 kPa)*
- *If abrasive grit or large particles are present in the water supply, an inlet strainer is recommended to prevent damage to the backwash valve*

SYSTEM SPECIFICATIONS

	Model	Flow Rate (m^3/hr)		Inlet/ Outlet (mm)	Valve	Tank Diameter (mm)	Tank Height (mm)	Tank Volume (Litre)
		Max	B/Wash					
Household/Farm	MM250A	1.85	1.85	25	Performa 263/440	250	1525	62
	MM300A	2.65	2.65	25	Performa 263/440	300	1470	84
	MM350A	3.50	3.50	25	Performa 263/440	350	1800	150
Medium	MM400A	4.70	4.70	25	Performa 180/440	400	1800	185
	MM500A	8.10	8.10	50	Magnum	540	1830	322
Commercial/ Industrial	MM600A	10.7	10.7	50	Magnum	620	2080	450
	MM750A	16.7	16.7	50	Magnum	760	2080	700
	MM900A	23.0	23.0	50	Magnum	914	2080	980

A minimum 200 kPa (30 psi) operating pressure is required. *Larger units are available.*

For more information contact...

Filtration Technology Ltd
PO Box 58-853, Greenmount, Auckland.

Auckland: Tel: 0-9-274 4223, Fax: 0-9-274 4224
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Dunedin: Tel: 0-3-488 6200, Fax: 0-3-488 6300
Website: www.aquaflo.co.nz

AquaFlo

pH Neutralising Filter

...for raising water pH levels from acidic to neutral and above neutral preventing corrosion.

New Zealand bore and roof water supplies are naturally acidic, some as low as 4.5 pH. Most are around 6.0 - 6.8 pH. At these pH levels, the water is a natural solvent and will try to correct itself by absorbing minerals/metals from pipes, taps and tanks.

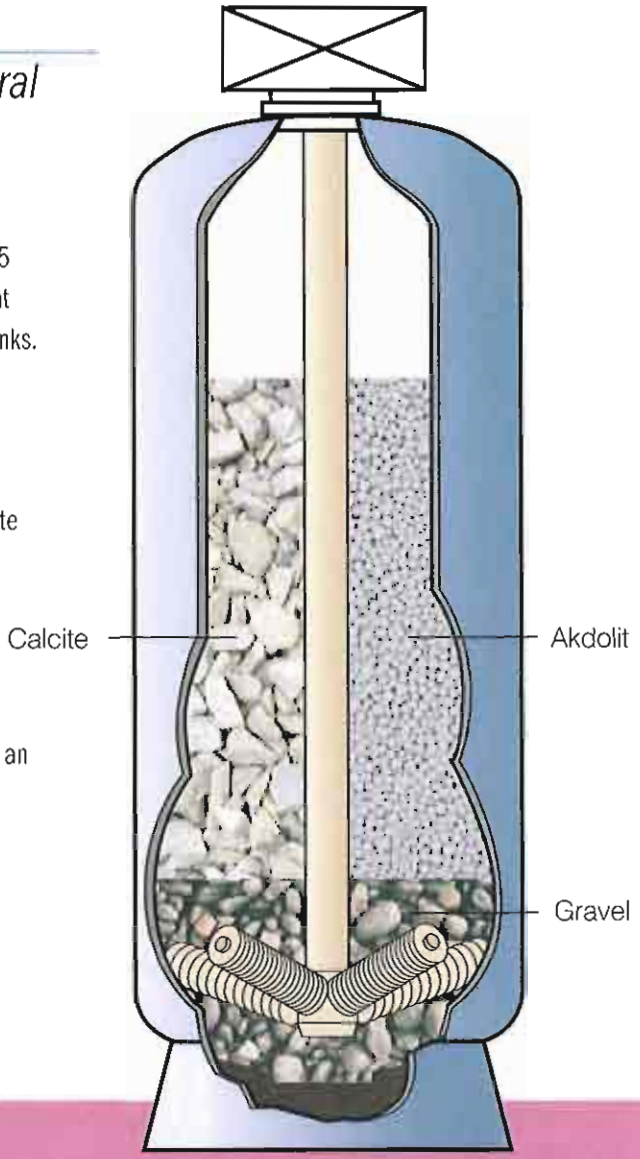
A lot of the minerals/metals contained in the water are toxic to humans in larger doses (i.e. lead, zinc, copper and others).

AquaFlo pH Neutralisers use natural minerals, such as calcium (lime) and Dolomite (heat treated to produce Akdolit) to raise the pH level of the water into the desired range. The low pH water takes up these minerals until it reaches a stable state. This prevents the water from corroding pipe work, PLUS makes your drinking water safer to drink without high lead, zinc or copper levels.

If iron and manganese concentrations are high, use a Calcite Filter, otherwise use an Akdolit Filter. If hardness is above 80gm/m^3 $CaCo_3$, consult with supplier.

The control of the filter system ranges from Forward Flow or Up Flow, Manual Backwashing to Fully Automatic (a time-clock allows adjustable backwashes from 1 to 6 days.

The system connects to the main waterline to your home and treats all incoming water. Units can be fitted with blending valves to control pH levels.



Control Valve Options

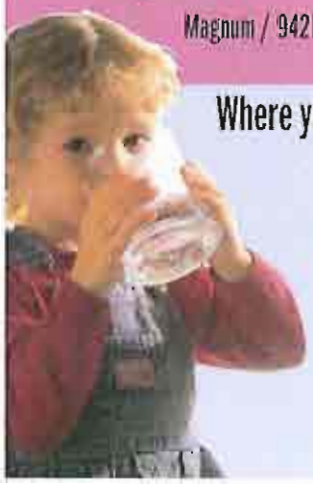

Magnum / 942F


180 / 440i

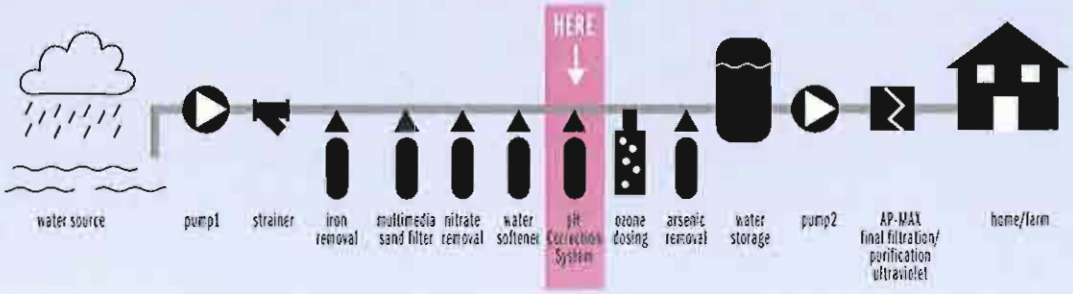

Performa / 740


Performa / 440i


Manual Multifunction



Where you are in the AquaFlo water supply sequence...



GENERAL SPECIFICATIONS

Pressure Vessels
Wound resin bonded fibreglass filament for strength and polyethylene inner shell for chemical inertness.
Working Pressure 200 - 1040 kPa. (30 - 150 psi)
Operating Temperature 5 - 40°C maximum

Valves*
- Multifunction manual
- Autotrol Series Performa 263 & 273 Noryl Control valve for small to medium size plant
- Autotrol 180 Brass valve for medium to large

Controllers
Electrically operated 6 or 7 day time clock driving control valve as standard. Optional volume based control available.

Instructions
Installation and Operating Instructions are included with all Multimedia Filters.

Media
High quality greensand, gravels and optional other media as appropriate for the application.

Pipe Work
Standing pipe work, where fitted, connecting filter vessel to valving is rigid PVC. Galvanised pipe is available as an option.

Installation
Any regulatory requirements pertaining to plumbing and drainage connections are the responsibility of the client. Plant should be installed under cover protected from direct sunlight, rain and frost.

Client to Provide
- Single phase 230 volt power (only if autocontroller used).
- Drain for backwash.
- All connecting pipe work and valving from control unit.
- Minimum 200 kPa (30 psi) operating pressure if backwashing
- Water flow or pump operation at 2.00 am to allow for Auto-backwashing

** Filtec reserves the right to offer an alternative valve(s) depending on customer requirements.*

Cautions:

- *A Pressure Reduction Valve should be installed in areas of high water pressure over 800 kPa (120 psi)*
- *If abrasive grit or large particles are present in the water supply, an inlet strainer is recommended to prevent damage to the backwash valve*

SYSTEM SPECIFICATIONS

Model	Flow Rate (L/min)		Inlet/ Outlet (mm)	Valve	Tank Diameter (mm)	Tank Height (mm)	Media (kg)	Tank Volume (Litre)
	Max	Backwash						
AK180	28	28	20	M=Manual Multi A=Performa timeclock	180	935	20	20
AK200	45	45	20	M=Manual in/out A=Performa timeclock	200	935	20	25
AK225	50	50	20	M=Manual in/out A=Performa timeclock	225	935	20	31
AK250	60	60	20	M=Manual in/out A=Performa 740/440i	250	1420	20	62
AK300	75	75	25	M=Manual in/out A=180/440i	300	1360	20	84
CL250	20	25	20/25	M=Manual Multi A=Performa timeclock	250	1420	40	62
CL300	35	35	25/25	M=Manual in/out A=Performa timeclock	300	1360	60	84
CL330	48	48	25/25	M=Manual in/out A=Performa timeclock	330	1800	75	154
CL400	55	50	25/25	M=Manual in/out A=Performa 740/440i	400	1830	100	185
CL500	100	80	32/32	M=Manual in/out A=180/440i	500	1830	170	322

A minimum 200 kPa (30 psi) operating pressure is required for automatic units. Larger units are available. Backwash thread 15mm

For more information contact...

Filtration Technology Ltd
PO Box 58-853, Greenmount, Auckland.
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Dunedin: Tel: 0-3-488 6200, Fax: 0-3-488 6300
Website: www.aquaflo.co.nz

Local Agent:

Appendix H

Wastewater Treatment Options

Appendix H

This Appendix provides additional information on the treatment processes described in Section 7.6 of the report.

1. Sequencing Batch Reactor

An SBR is a variation of the conventional activated sludge process where the wastewater is treated by micro-organisms suspended in the wastewater. Wastewater is treated in "batches" rather than by means of a continuous inflow and outflow. The treatment cycle (with anoxic phases if required) is completed in a single tank which eliminates the need for a separate clarifier. The treatment cycle usually consists of aerobic, anoxic, settling and decant stages.

For an SBR the following has been allowed for:

- Grit Removal
- Screening
- Alum Dosing for Phosphorus Removal
- SBR Tanks
- Filtration System
- UV Disinfection
- Waste Activated Sludge (WAS) Tanks
- Ancillary Items: pumps/pipework/valves/blowers etc.

SBR systems are suitable for small communities; however they require a high level of control and operator expertise to run successfully. As with all high rate processes they will produce a quantity of waste sludge that must be treated and disposed of separately from the treated effluent.

Waste sludge would be stored and thickened in a sludge holding tank and removed for further processing and disposal at an approved landfill site.

The Process Flow Diagram of the SBR system is shown in below.

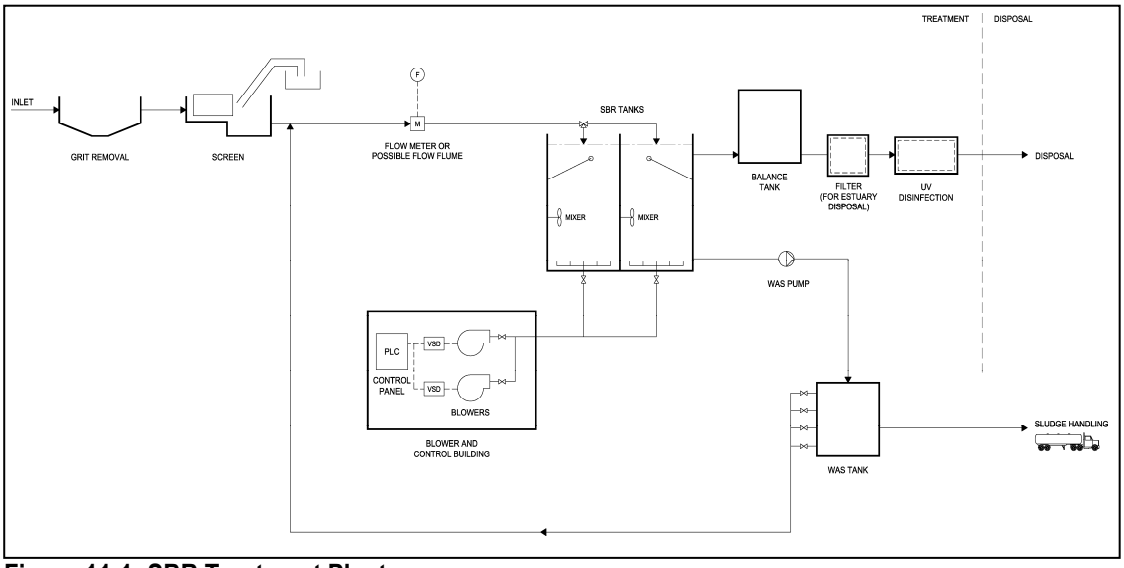

Figure 11-1: SBR Treatment Plant

An SBR treatment system is seen as a flexible unit than can be altered to produce various effluent quality requirements simply by programmed changes to the PLC.

2. Membrane Bio-Reactor (MBR)

Membrane Bio-Reactor (MBR) plants are a combination of standard activated sludge wastewater treatment plants and membrane filtration. The membrane filter replaces the clarifier and disinfection stages of a conventional treatment plant. The membrane provides an absolute barrier and restricts all particles greater than 0.2mm. As the membrane removes almost all suspended solids they also remove pathogens and significant biological loads.

For MBR the following has been allowed for:

- Grit Removal
- Fine Screening
- Alum Dosing for Phosphorus Removal
- Reactor Tanks
- Permeate Tank
- Membranes
- Hypochlorite Cleaning System
- WAS Tanks
- Ancillary Items: pumps/pipework/valves/blowers etc.

After entering through the inlet works the sewage flows into the treatment tank containing the submerged membranes and air diffusers. The effluent is aerated to provide the necessary oxygen for the aerobic breakdown of the sewage by micro-organisms and also for continuous cleaning of the membrane modules. The effluent then filters through the membranes by gravity flow or vacuum and the treated effluent is stored in the permeate tank prior to discharge.

Sludge is removed directly from the MBR tanks and stored in Waste Activated Sludge (WAS) storage tanks for settling and periodic removal. The blowers, with built-in noise attenuation, would be housed in a small building along with the control panel.

The membrane filtration combines the solids separation (removing the treated liquid effluent from the sludge) and disinfection (removal of pathogens) in one process. Chemical cleaning of the membranes (hypochlorite backwash) is required once every six months. Waste sludge is stored and thickened in a sludge holding tank and then removed for further processing, as discussed above.

MBR plants can achieve effective nitrogen removal and can be upgraded to enhance phosphorus removal. Modular expansion of MBR plants is easily achieved by the addition of membrane units or additional membrane sheets within a membrane unit. Membrane sheets have a typical life expectancy of 10 years or more. It should be noted that the chemical dosing system for Phosphorus removal has not been presented in MBR Process Flow Diagram in Figure 11-2 below.

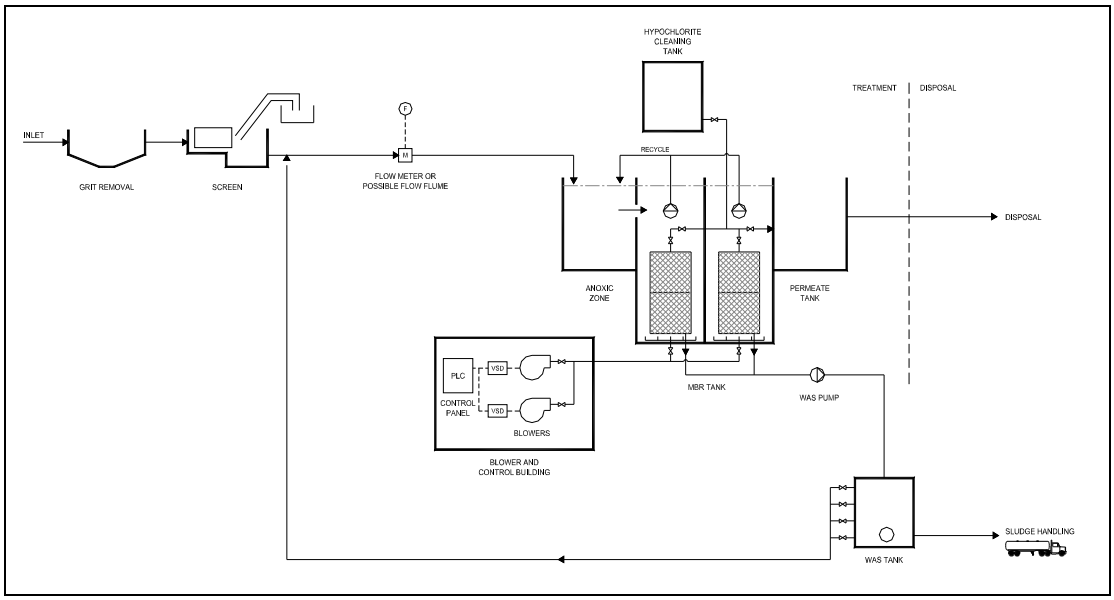

Figure 11-2: MBR Treatment Plant

3. Submerged Aerated Filter (SAF)

SAF plants are a fixed film process that utilise a solid media (typically made of plastic with a large specific surface area) in the process tanks which provides a surface upon which the biomass grows. The biomass treats the wastewater and removes the BOD and solids (as well as reducing ammonia if sized appropriately). The media is submerged in the tanks and air is mixed into the wastewater at the base of the tank using a diffuser/blower arrangement. The media used and the fact that it is submerged allows for substantially higher applied organic loading than more conventional fixed film processes such as trickling filters.

For the SAF process the following has been allowed for:

- Grit Removal
- Screening
- SAF Tanks
- Fixed Media and Accessories
- Clarifier
- Filtration System
- UV Disinfection
- WAS Tanks
- Ancillary Items: pumps/pipework/valves/blowers etc.

The biomass on the media continually sloughs off in the wastewater and is removed by a solids/liquid separation device, typically a clarifier, as waste sludge.

Waste sludge would be stored and thickened in a sludge holding tank and then removed for further processing, as discussed above. The Process Flow Diagram of the SAF system is shown in Figure 11-3 below.

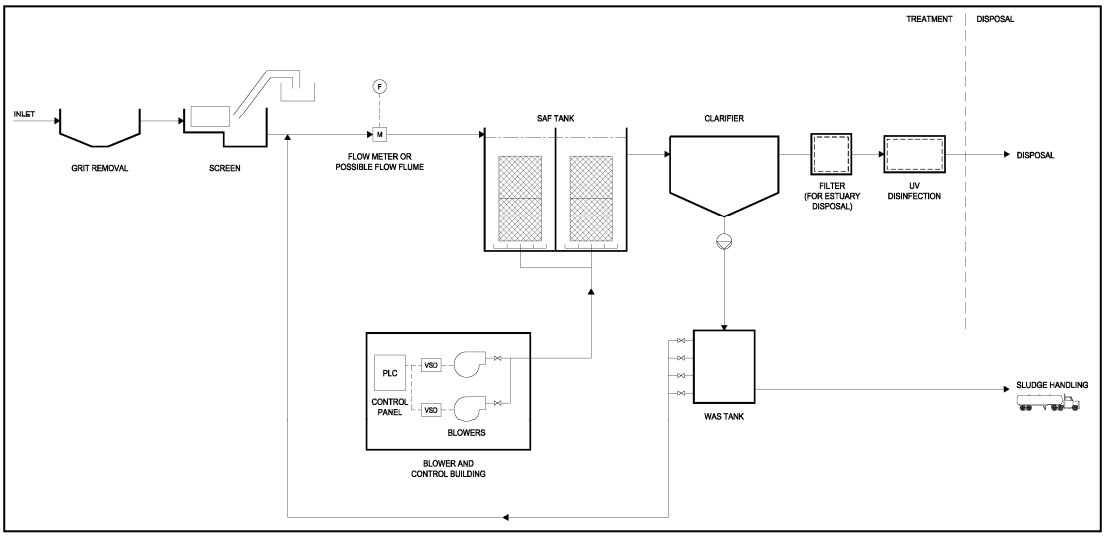

Figure 11-3: SAF Treatment Plant

There are several suppliers of package SAF systems on the market, alternatively it is also possible to undertake site specific designs. Either system can be designed with the flexibility for modular expansion as necessary.

4. Packed Bed Reactor (PBR)

Settleable solids and oil and grease etc are removed within the primary tank. The effluent passes to a recirculation chamber and is either sprayed into the PBR (which is a simple chamber containing a textile type media) or discharged to the disposal area. Flow is passed to the PBR in preference to the disposal field at a ratio of approximately 3:1 – 5:1. Recycled flows pass back into the recirculation chamber/primary tank.

For a PBR the following has been allowed for:

- Grit Removal
- Screening
- Primary Tank
- Recirculation Tank
- PBR Reactor Tank
- Filtration System
- UV Disinfection
- WAS Tanks
- Ancillary Items: pumps/pipework/valves/blowers etc.

PBR media is now (in New Zealand) mainly either plastic foam cubes or a hung textile fabric. This is more in keeping with high rate trickling filter design/operation. The Process Flow Diagram of the PBR system is shown in Figure 11-4 below.

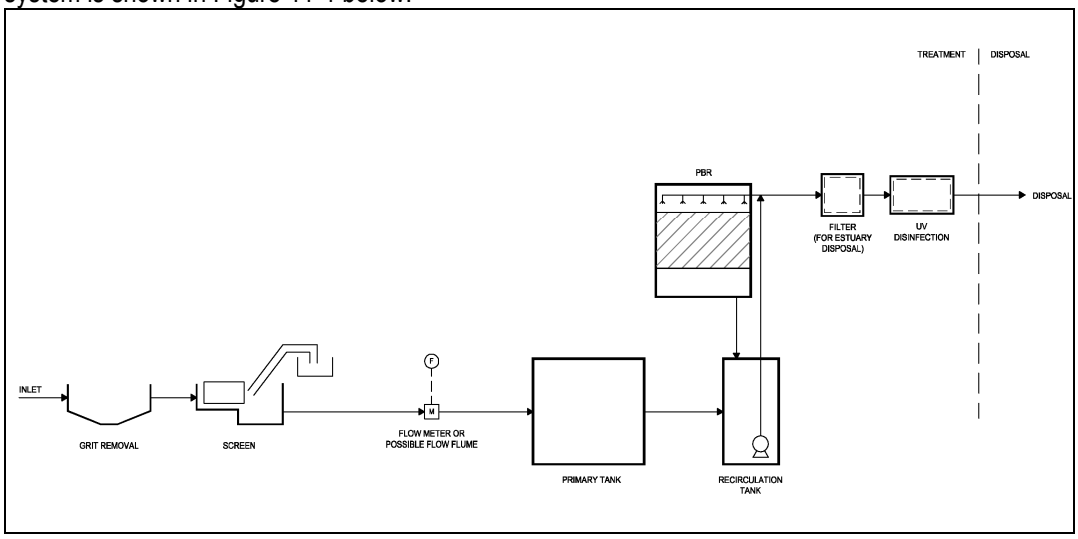

Figure 11-4: PBR Treatment Plant

There are several suppliers of package PBR systems on the market, alternatively it is also possible to undertake site specific designs. Either system can be designed with the flexibility for modular expansion as necessary.