# SHAW/HUNA ROAD REZONING

## ATTACHMENTS TO JOHN OLLIVER'S REPORT

1. **ATTACHMENT 1 – COMPARATIVE ASSESSMENT OF SHAW RD SITE**
2. **ATTACHMENT 2 – TRAFFIC REPORTS BY TRAFFIC GROUP**
3. **ATTACHMENT 3 – GEOTECHNICAL ASSESSMENT**
4. **ATTACHMENT 4 – STORMWATER FEASIBILITY STUDY**
5. **ATTACHMENT 5 – CONTAMINATION ASSESSMENT**

# Attachment 1

# Comparative Assessment of Shaw Rd Site

Assessment of Shaw Rd Site

| | 2010 Growth Strategy comments | My comments |
|---|---|---|
| Area | 21 ha | |
| Land ownership | General land, privately owned by 11 owners. | As set out in paragraph 3 there are five certificates of title, each in separate ownership. I.e. a total of 5 owners. |
| Cultural heritage | General land, no recorded sites, adjacent Maori land. | No further comment. |
| Natural heritage | No significant ecological values. | No further comment. |
| Landscape features | Need to avoid effects on dune landscapes. | While the land rises to the north it is not a 'dune landscape' that is of such value that it needs to be preserved. |
| Socially and physically contiguous | Isolated – Creating a new residential area. Not reinforcing existing town character. | The site is isolated from any existing urban development. This disadvantage would apply to any rezoning in Piripai/Paroa West of Keepa Rd area, except for land immediately adjacent to Coastlands. The land is close to the Hub which provides stopping facilities and close to industrial areas providing employment. |
| Versatile Soils/land contamination | 14ha versatile soil<br><br>No known waste disposal | The area of versatile soil is relatively small in the context of the District and any urban expansion to the west is likely to encounter the same issue.<br>A preliminary site investigation was prepared by Geosciences Ltd dated 13 December 2012 in accordance with the NES for Assessing and Managing Contaminants in Soil. The investigation covers only the Gray and Focus Trustee properties, being about 10ha of the site. The report concludes that the land is suitable for residential use and it is highly unlikely that there will be any risk to human health or the environment as a result of former horticultural activities on the site. |
| Social and Land Use Incompatibility | No incompatible land uses nearby. | The site is outside the 300m buffer area around the sewerage ponds, which provides guidance as to incompatibility. Traffic noise from the state highway can be mitigated by a standard 40m building setback. |
| Natural hazards/Land Stability/Flooding, | Low lying area with potential foundation and flooding | A geotechnical assessment has been undertaken by Coffey Geotechnics, dated 27 November 2012. The |

| | | |
|---|---|---|
| Inundation. | problems. Relatively stable contours. Some earthquake risk in high water table areas. Some areas of higher land may be suitable for development. May need to raise land to required building platform levels and stopbank treatment. | assessment excludes the two small rural residential blocks. It concludes that liquefaction settlement is expected to be relatively minor. Lateral spreading could occur near the Kope Canal but design can mitigate this. Bearing capacity of the shallow soils is generally insufficient for standard foundation construction and therefore either ground improvements or specific foundation design will be required. Overall it concludes that the site is suitable for residential zoning. The Regional Council have proposed a minimum floor level of 3.15m for the land. The landowner (Barney Gray) has advised his intention would be to undertake a cut and fill on the site to raise the lower-lying areas. This appears to be a feasible approach. |
| Services (Water, Wastewater, Stormwater). | Water - Extension and upgrade of existing reticulation system required. Additional storage required.<br><br>Sewer – New sewer reticulation and a pump station required. Upgrades to the existing treatment system required.<br><br>Stormwater – Reticulation pipes and pump station may be required. | The Manager Utilities has advised that the site can be serviced for water and wastewater. The extensions would be funded by Council and recovered via Development Contributions.<br><br>Preliminary stormwater feasibility study and discussions with Regional Council staff indicate that there will be acceptable technical solutions available, but that pump upgrading costs are unknown and may be significant. |
| Transport | Any growth west of the river will put further congestion on the bridge and routes into town. Increases the exposure to route security issues as there is only one bridge across the river to link town and the CBD to the westward growth. | A preliminary traffic scoping assessment has been undertaken by Traffic Design Group, dated October 2012. The report acknowledges that any development to the west of the river will place pressure on the Landing Rd bridge, but that this has been factored in to future planning for some time. An addendum to the report dated 25 January 2013 concludes that traffic can be accommodated on the existing intersections with some minor upgrading. |

Whakatane District Council

# Proposed Residential Subdivision, Shaw Road, Huna Road

## Scoping Assessment

October 2012

Traffic Design Group

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Whakatane District Council

# Proposed Residential Subdivision, Shaw Road, Huna Road

## Scoping Assessment
## Quality Assurance Statement

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# 1. Introduction

Traffic Design Group Ltd ("TDG") has been appointed by Whakatane District Council (Council) to prepare a Scoping Assessment of a proposed 200 lot residential subdivision on the outskirts of Whakatane, adjacent to State Highway 30 (SH30). This report includes an assessment of the traffic issues associated with the development, with particular focus on the effects of:

- Development traffic generation;
- Provision of connections to the wider road network, and
- Traffic effects on the surrounding road network, with specific consideration of the operating performance of the SH30 / Huna Road intersection and the SH30 Shaw Road intersections.

The purpose of the assessment is to identify the likely issues and potential mitigation that may be required in order to safely and efficiently manage the development on the road network.

# 2. Background Information

## 2.1 Whakatane District Council Urban Growth Strategy

The *Whakatane Integrated Urban Growth Strategy* (2010) assesses growth options for the Whakatane District to 2050. The purpose of this Strategy is to provide ways of planning for and managing growth in a proactive manner. The Strategy has identified potential future growth areas which are shown on a plan titled "Future Directions" and included below as Figure 1

The Strategy identified a preferred growth scenario targeting 25,000 people by 2050 with the location of growth subject to consultation as part of the District Plan Review.

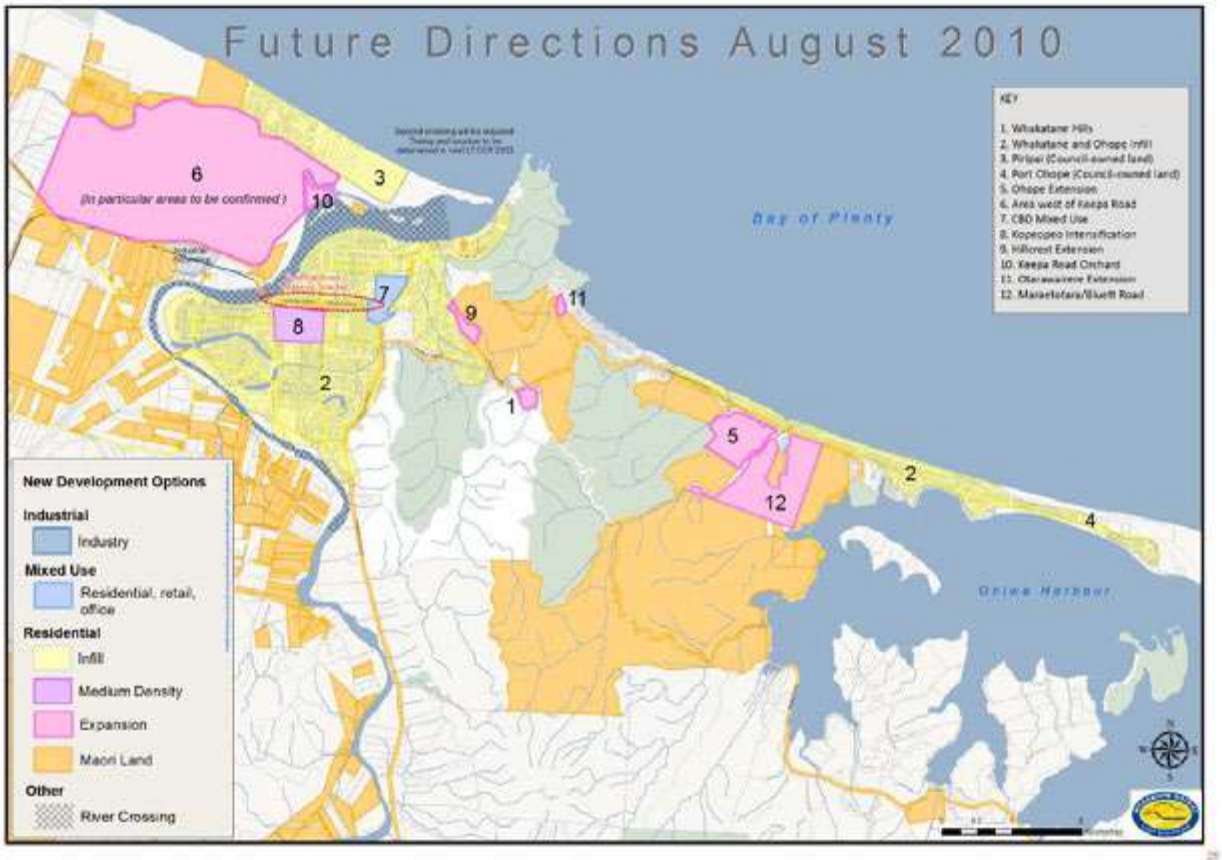


**Figure 1: Future Directions Growth Areas**

Area 6 (Area west of Keepa Road) as identified as potential future residential expansion encompasses a large block of land which includes the site that is the subject of this current assessment.

The Strategy recognised that all options would have an effect on the Landing Road Bridge as stated in Section 7:

> *"A new State Highway river crossing will be required regardless of options. The Whakatane River bridge will require additional capacity by 2016*".

An assessment of the effects of each growth area option was included in the Strategy and the following transport issues where identified for the area west of Keepa Road:

- *Any growth west of the river will put further congestion on the bridge and routes into town;*
- *Increases the exposure to route security issues as there is only one bridge across the river to link town and the Town Centre to the westward growth; and*
- *Increased densities will increase the threshold to support public transport*.

## 2.2 Whakatane Township Network Investigation

A report prepared for WDC in 2007 "*Whakatane Township Network Investigation Report*" (Burnett and Olliver Ltd and Gabites Porter Consultants included modelling of the Whakatane transport was based on an earlier Residential Review (being the medium growth scenario). It is understood that this modelling was reasonably consistent with Council's preferred growth scenario of a population of 25,000.

The following extracts from the *Whakatane Integrated Urban Growth Strategy* summarise the key aspects of this study:

> *"Part of the 2007 study included the SH2 Alternative Routes Scoping Study which was jointly undertaken between the Whakatane District Council and New Zealand Transport Agency. This considered the State Highway network in relation to route efficiency and security. The outcome of this study was a proposal to consider replacing the Pekatahi Bridge closer to the urban area of Whakatane. It concluded that the most affordable option for a second bridge was a location on the southern outskirts of the Whakatane urban area, crossing the river from Poroporo to Taneatua Road at a point where the river is at its most narrow.*
>
> *….However, a shift in the focus of the NZTA since the 2007 report means that the form, function and route security of the State Highway network through the district is being reviewed. This work will consider how the network contributes to the region and to the rest of the New Zealand economy, in particular, the connectivity between the ports of Tauranga and Gisborne*.

*Previous transportation modelling done to assess the implications of urban growth show the following:*

- *Residential growth in Coastlands has a minimal effect on traffic volumes and congestion compared to The Hub retail development.*
- *Traffic modelling shows that additional capacity on the existing bridge is required by 2016. Faster population growth would bring the need for this forward which means planning for this should be underway.*
- *Similarly, the bridge/Landing Road roundabout will need upgrading, probably sooner than 2026 if population growth is faster than medium growth projections*".

## 2.3 District Plan Review

Subsequent to the preparation of the *Integrated Urban Growth Strategy*, it is understood that Council have updated growth and land development forecasts and Council have advised that they have adopted a medium growth rate. The growth rate projections for the Whakatane Ward for the period 2011 to 2026 are now provided as Household Equivalent Units (HEU). The growth rate now being projected is 540 HEUs split over each five year period.

# 3. Existing Transportation Infrastructure

## 3.1 Location in the Road Network

The proposed development site occupies two adjacent blocks of land bounded by SH30 to the south, Huna Road to the west, Shaw Road to the east / southeast, and rural pastoral land to the north. The site is located approximately 2km west of the SH30 road bridge over the Whakatane River. Figure 2 shows the location of the site within the local road hierarchy, while Figure 3 shows the site in the context of its immediate surroundings.

### 3.1.1 SH30

SH30 is classified in the District Plan as a Primary (Regional) Arterial Road, providing the sole road crossing of the Whakatane River in the vicinity of the Whakatane township, and linking the town to the wider road network to the west, making it the link to the larger population centres of Rotorua and Tauranga and further afield to Hamilton and Auckland.

Adjacent to the site SH30 is a two lane rural road on flat terrain with a 100km/h speed restriction. The total sealed width is typically around 10.0m, and marked with two 3.5m wide traffic lanes and 1.5m wide shoulders. Photograph 1 below shows the form of SH30 adjacent to the site.

**Photograph 1: Facing east toward Whakatane on SH30 with the site on the left**

### 3.1.2 Huna Road

Huna Road is a rural road classified in the District Plan as a local road, with the primary purpose of providing access to adjacent properties. Adjacent to the site Huna Road is in level to gently rolling terrain and has a 100km/h speed restriction. The total sealed width is typically around 7.2m, and marked with a centreline only.

**Photograph 2: Facing south on Huna Road toward SH30 with the site on the left**

At the south-western corner of the site, is the intersection of Huna Road and SH30 as shown in Photograph 2 above.

### 3.1.3 Shaw Road

Shaw Road is also classified as a local road. Shaw Road intersects SH30 at right angles along the southern boundary of the site then immediately undergoes a sharp bend to the right, to run in an east west direction almost parallel to SH30, with the Kope Canal located between Shaw Road and SH30, as shown on Figure 2 (the section of Shaw Road parallel to the canal is also known as Kope Canal Road). The road continues in this direction for a length of 230m before undergoing another sharp bend, this time to the left. Beyond this sharp bend Shaw Road runs along the eastern boundary of the site parallel to Huna Road for a length of 790m before the road ends, effectively making it a rural cul-de-sac for property access only.

**Photograph 3: Facing east on Kope Canal section of Shaw Road with the site on the left**

**Photograph 4 Facing south on Shaw Road with the site on the right**

The section of Shaw Road parallel to the canal shown in Photograph 3 is typically 6.0 to 6.2m wide and marked with a centreline only. The road is elevated above the surrounding land by approximately 2.0m effectively running along the top of a stop-bank for the canal, with sloping embankments adjacent to both sides of the road.

The section of Shaw Road adjacent to the eastern boundary of the site is at the grade of the surrounding land as is shown in Photograph 4 above. This section of Shaw Road has a seal width typically between 6.5 and 6.8m wide.

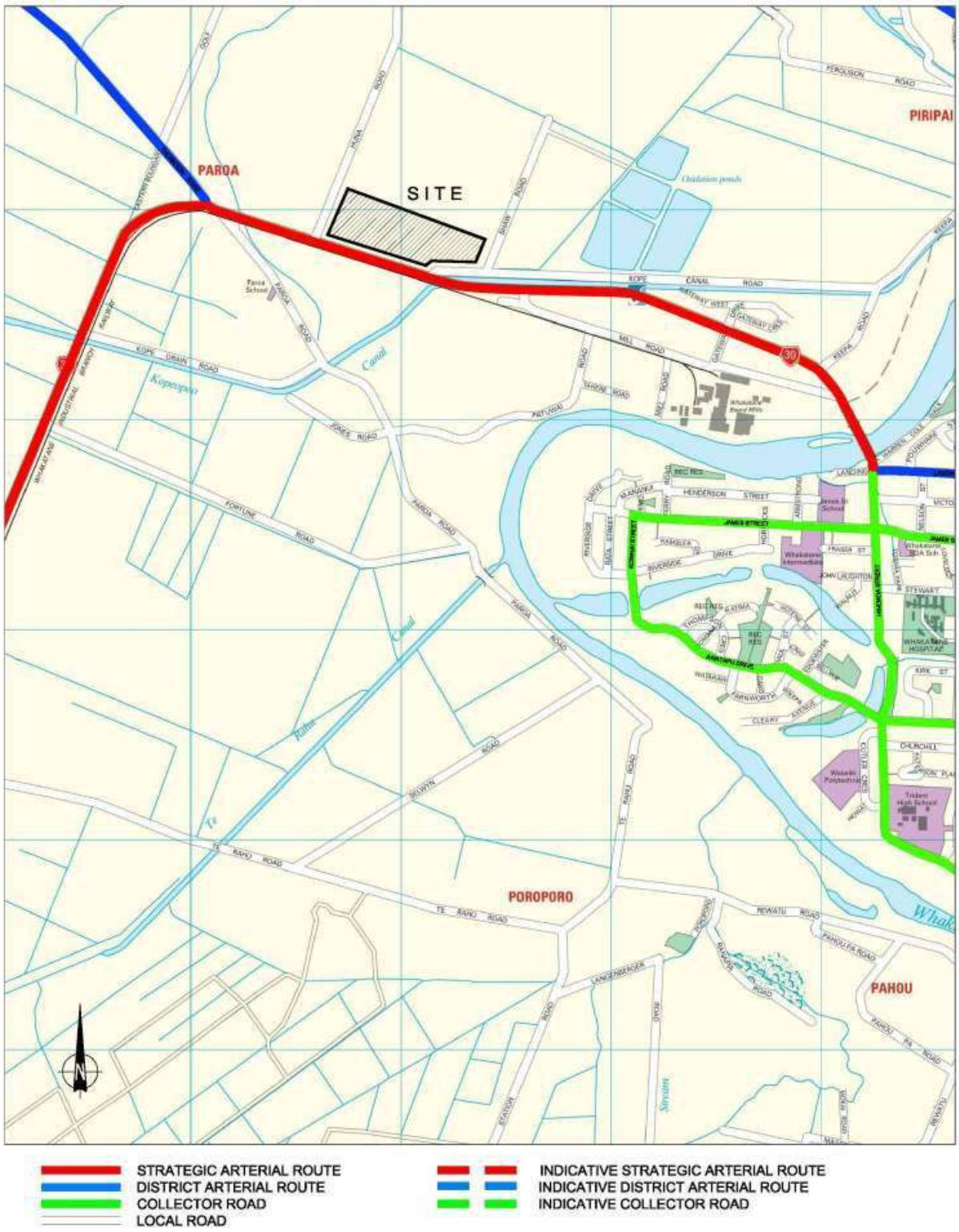


**Figure 2: Site Location in the Road Network**


**Figure 3: Aerial Photograph of Site and Surrounds**

# 4. Travel Patterns

## 4.1 SH30

Traffic volumes on SH30 are relatively high for a two lane rural facility. NZTA's latest reported ADT for SH30 recorded approximately 1.5km east of the site, 315 to the west of Keepa Road is 14,489vpd. This ADT was recorded in 2011, and is approximately 900vpd lower than the peak recorded at this site of 15,401 in 2008.

The latest available week of count data from this site was recorded in June 2012. The peak hour data from the June 2012 count is summarised in the table below:

| DIRECTION | 7- DAY AVERAGE DAILY TRAFFIC (vpd) | WEEKDAY AVERAGE DAILY TRAFFIC (vpd) | WEEKDAY AM PEAK HOUR (vph) | WEEKDAY PM PEAK HOUR (vph) |
|---|---|---|---|---|
| Westbound | 6,814 | 7,330 | 572 | 764 |
| Eastbound | 6,802 | 7,302 | 811 | 789 |
| **Two-Way** | **13,616** | **14,631** | **1,383** | **1,553** |

**Table 1: SH30 Count Site Traffic Flow Data from June 2012**

The data in Table 1 indicates that the ADT on SH30 has continued the reducing trend shown between 2008 and 2011 into 2012.

It is noted that a count was undertaken on SH30 during Easter weekend in 2012, and that on the Thursday the PM peak hour traffic volume peaked at 2,140vph. While this illustrates that traffic volumes do at times reach volumes significantly higher than the typical weekly peaks, these volumes have not been specifically considered in the effects assessment.

Peak hour turning movement surveys were undertaken at the intersection of Shaw Road / SH30 on Wednesday the 12th of September and Huna Road SH30 on Thursday the 13th of September.

On reviewing the survey data there is a discrepancy between the recorded traffic volumes on SH30 between the two survey days. The data recorded during the Huna Road survey is consistent with our expectation of what the peak hour flows would be, being approximately 10% lower than those recorded at the NZTA count site, closer to Whakatane and the Hub retail centre.

On that basis the traffic volumes recorded on SH30 during the Huna Road turning movement survey have been adopted as the SH30 peak hour volumes in this assessment. These volumes are shown in Table 2 below.

| DIRECTION | WEEKDAY AM PEAK HOUR (vph) | WEEKDAY PM PEAK HOUR (vph) |
|---|---|---|
| Westbound | 551 | 774 |
| Eastbound | 710 | 636 |
| **Two-Way** | **1,261** | **1,410** |

**Table 2: SH30 Peak Hour Traffic Flows in Vicinity of the Site**

## 4.2 Future Traffic Forecasts for SH30

The most recent traffic forecasts for SH30 to the west of the Landing Road bridge have been adopted from the draft report currently under preparation for WDC and NZTA – the "Whakatane Access and Security Scoping Study". This study includes traffic growth forecasts for the "medium growth scenario" and these forecasts which are based on the Whakatane Regional Transport Model are tabled below in Table 3.

| YEAR | HOUSEHOLDS | ADT (vpd) |
|---|---|---|
| 2006 | 6,546 | 16,142 |
| 2016 | 7,590 | 18,850 |
| 2026 | 8,839 | 21,266 |
| 2036 | 9,965 | 22,679 |

**Table 3: ADT Flow Data from WRTM: SH30 East of Keepa Road**

Based on the growth forecasts in Table 3, the equivalent annual compounding growth rate is 1.14%.

### 4.2.1 Local Roads

Traffic Counters were installed on Huna Road and Shaw Road for the week ending 21 September 2012 .and the following table summarises the daily and peak hour volumes record

| ROAD | ADT (vpd) | WEEKDAY PEAK HOUR (vph) |
|---|---|---|
| Huna Road | 269 | 39 |
| Shaw Roadd | 154 | 24 |

**Table 4: Local Road ADT and Peak Hour Traffic Volumes**

# 5. Road Safety

A search has been undertaken of the NZTA Crash Analysis System to identify all recorded crashes in the vicinity of the site. The search covered SH30, from a point 250m west of Huna Road to a point 250m east of Shaw Road, plus Huna Road and Shaw Road along the site frontages. The search covered the years 2007 to 2011 inclusive, and also includes data available for 2012. Three crashes were recorded which matched these parameters, all of which were on SH30.

One crash occurred approximately 150m west of Huna Road when a westbound car driver lost control in heavy rain and collided with an oncoming vehicle, resulting in minor injuries.

One non-injury crash occurred approximately 400m east of Huna Road when an eastbound car driver was distracted by a cigarette and lost control, leaving the carriageway.

One non-injury crash occurred approximately 80m east of Shaw Road when a westbound van driver lost control and collided with the guardrail.

No trend is evident in terms of crash type or location which would suggest a safety issue with the existing road layout.

# 6. Proposed Development

It is proposed to develop the full site as a 200 lot residential subdivision. Figure 3 shows that this is expected to occur in two stages, with the first block to be developed being the eastern block.

However the full development will be critical in terms of assessing traffic effects and will also dictate the location and number of access points onto the exiting road network. On this basis the subdivision has been considered as a whole, with an assessment of the full 200 lots. The potential staged approach to the development has also been considered based on a first stage of approximately 100 lots located on the eastern side of the site.

Currently the site is occupied by three residential dwellings, a berry fruit orchard, an olive grove and general pastoral farmland. The berry fruit orchard has a 60 space car park and is expected to generate reasonable volumes of traffic at times of peak operation. However peak traffic activity is unlikely to occur during the AM and PM commuter peak periods when a residential subdivision generates its peak traffic. For the purposes of this assessment the traffic generated by these existing activities on the site has not been specifically assessed which is therefore results in a conservative approach to the effects analysis.

# 7. Traffic Generation

The expected traffic generation rates for the proposed activities have been identified from the Trips Database Bureau report, November 2011.

The report identifies an 85th percentile daily traffic generation of residential dwellings at 10.4 trips per day.

Ten per cent of the daily figure has been adopted as the peak hour figure, giving 1.0 traffic movements per lot per hour. It is noted that areas on the edge of town tend to have lower rates due to combined trips and on that basis the selected trip rates are may be marginally conservative, i.e. high.

| Time Period | Data Source | Trips per Dwelling | Total Trips |
|---|---|---|---|
| Daily | Trips Database Bureau 2011 | 10.4 vpd | 2,080 vpd |
| Peak Hour | Trips Database Bureau 2011 | 1.0 vph | 200 vph |

**Table 5: Trip Generation**

Table 5 shows that the site is expected to generate up to 2,080 vehicle movements per day and up to 200 vehicle movements per hour in the morning and evening peak periods.

Given the nearest commercial centre is located approximately 1.5km from the site it is likely that pedestrian movements to and from the site will be low. However, the site is within easy cycling distance of the CBD and consideration to the provision of cyclists may be necessary in the detailed traffic assessment phase for the site.

## 7.1 Trip Distribution

It is expected that the vast majority of site generated traffic will travel to / from Whakatane to the east. For the purposes of this assessment it has been assumed that 90% of traffic will travel to / from the east and that 10% will travel to / from the west. Consistent with this expectation it follows that more traffic will access SH30 via Shaw Road to the east of the site, than by Huna Road to the west of the site. For the purposes of this assessment it has been assumed that 75% of traffic will access SH30 via Shaw Road and that 25% of traffic will access SH30 via Huna Road.

During the AM and PM peak hours the split between site entry and site exit movements has been based on the ITE[1] proportions of 75% outbound and 25% inbound in the morning, and 37% outbound and 63% inbound in the PM peak hour.

[1] Institute of Transportation Engineers Trip Generation 8th Edition

# 8. Site Access / Egress

The site has direct road frontage onto Huna Road, Shaw Road (Kope Canal section), Shaw Road, and SH30.

The Shaw Road and Huna Road intersections with SH30 are separated by 585m. It would be undesirable to construct an additional road intersection onto SH30 between them, unless one or both of the existing intersections was closed.

While options that include the construction of a new SH30 intersection and closure of both or one of the existing intersections could be developed, it unlikely that such an option would be cost effective and or offer significant benefits over the provision of access directly onto Huna Road and Shaw Road.

On this basis the primary focus of this assessment has been on identifying an access arrangement for the site that considers the provision of an access onto Huna Road, a further access onto Shaw Road, and an optional third access onto Shaw Road (Kope Canal section).

## 8.1 Huna Road

Huna Road is a rural road with a straight horizontal alignment and 100km/h speed limit. Operating speeds in the vicinity of the site frontage are expected to be 90-100km/h.

Three is a vertical crest curve in Huna Road immediately beyond the sites northern boundary that restricts sight lines to the north from the sites frontage onto Huna Road.

**Photograph 5: Facing north on Huna Road toward with the site on the right**

Photograph 5 above was taken from a location approximately 20m south of the access to the berry orchard on the right, and shows the crest curve in the distance that limits sight lines. The available sight line from the location of the existing berry orchard was measured to be 215m during the site visit. Lesser sight distances are expected from locations further to the north although the rate at which the sight distance reduces was not specifically measured.

*Austroads Part 4A: Unsignalised and Signalised Intersections* provides guidance on sight distance requirements at intersections.

The desirable Safe Intersection Sight Distance (SISD) for a 90km/h and 100km/h operating speeds respectively are 214m and 248m based on a two second reaction time and a 3 second observation period before braking.

The desirable sight distance standard is often not achieved on the rural road network in New Zealand, and lesser standards have been developed acknowledging this, known as Extended Design Domain (EDD) values. These are values outside the Normal Design Domain that through research and or operating experience have been found to provide a suitable solution in constrained situations.

The very low existing traffic volumes on Huna Road, and the simple form of the proposed new intersection as a Tee junction make it an appropriate location for consideration of EDD principles.

Adopting a 100km/h operating speed and the Austroads EDD parameters including a reaction time of 2.0 sec, and observation time of 2 sec, gives a SISD requirement of 197m.

On this basis it is assessed that 200m is an acceptable minimum sight distance to provide at a new access intersection onto Huna Road from the subdivision.

The locations where this minimum sight distance can be achieved will need to be quantified accurately during further design, but it is anticipated that this minimum sight distance will prevent the new intersection from being located more than 20-30m north of the existing berry orchard access.

Locating the new intersection in close proximity to SH30 is also undesirable, as it can result in interaction of vehicles slowing or accelerating for the respective intersections and queuing interaction. Vehicle volumes on Huna Road are very low and queuing by vehicles on Huna Road waiting to turn into the subdivision road will not be expected to exceed one or two vehicles.

It is recommended that the intersection is restricted to being no closer to SH30 than 100m. At this location sight distance to north would nominally meet the Austroads desirable standard for a 100km/h operating speed, while still maintaining an acceptable separation from SH30, with no risk of queuing interaction between the new intersection and SH30.

There is an existing access on the western side of Huna located 110m from SH30. This access services one dwelling, and three further rural lots developed with pastoral land or crops. While it would be desirable to locate the new intersection away from this access it is not assessed as a necessary requirement, as the access is expected to generate very low traffic volumes.

## 8.2 Shaw Road

Shaw Road is also a rural road with a 100km/h speed limit. The section of Shaw Road parallel to Huna Road is 790m long and has an assessed operating speed of 80km/h. Vehicles negotiating the tight curve that separates Shaw Road from the Kopu Canal section have an assessed operating speed of 30km/h- 40km/h. The desirable Austroads SISD for a 40km/h operating speed is 73m. On this basis any new intersection constructed onto Shaw Road should not be located within 73m of the curve.

There is an existing site access onto Shaw Road at a distance of 115m from the apex of the tight curve. This access is shown in Photograph 6 below on the right hand side. The access serves one small lot with a single residential dwelling, and it is anticipated that this lot and its access will remain.

**Photograph 6: Facing south on Shaw Road with the site on the right**

**Photograph 7 Facing north on Shaw Road with the site on the left**

On this basis there is some scope to locate a new intersection between this access and a point 73 north of the curve. However the most desirable location for a new road intersection onto Shaw Road would be to the north of this access driveway to separate the intersection from two dwellings located adjacent to the eastern side of Shaw Road at distances 40m and 80m from the curve.

There is approximately 85m of site frontage to the north of the existing access driveway, and it is recommended that the new road intersection is constructed in this length.

## 8.3 Shaw Road (Kope Canal Section)

The section of Shaw Road running parallel to the canal is 220m long with a very tight horizontal curve at each end.

Based on an operating speed of 40km/h at both curves the desirable Austroads SISD requirement is 73m. It is therefore recommended that a new intersection onto the Kope Canal section of Shaw Road be restricted to the middle section at least 73m clear of the commencement of the curve in either direction. This leaves a centrally located length of approximately 74m over which a new road intersection can be located.

## 8.4 New Intersection Design Standards

For the purposes of assessing appropriate intersection design standards it has been assumed that three new intersections will be constructed. With the majority of traffic expected to be to / from Whakatane the distribution between the three intersections has been assumed to favour what will likely be the most convenient / shortest route for this movement. On this basis the following table summarises the indicative expected percentage and volume of subdivision traffic that will use each intersection.

The design of the subdivision layout will be able to be modified to influence these percentages, but the percentages shown are expected to be indicative of an internal layout that does not attempt to specifically influence the distributions.

| INTERSECTION | PERCENTAGE OF TRAFFIC | DAILY VOLUME OF TRAFFIC | PEAK HOUR VOLUME OF TRAFFIC |
|---|---|---|---|
| Huna Road | 25% | 520 | 50 |
| Shaw Road | 25% | 520 | 50 |
| Shaw Road (Kope Canal) | 50% | 1,040 | 100 |

**Table 6: New Intersection Turning Volumes**

### 8.4.1 Huna Road

The turning movements at this intersection will be almost exclusively right turn movements from Huna Road and left turn movements onto Huna Road.

A Basic Right Turn (BAR) widening treatment as detailed at Figure 7.5 of *Austroads Part 4A Unsignalsied and Signalised Intersections* is recommend for the Huna Road intersection. This will facilitate the passing of vehicles that are slowing to pull into the subdivision, by following vehicles on Huna Road.

### 8.4.2 Shaw Road

The turning movements at both intersections on Shaw Road will be almost exclusively left turn movements from Shaw Road and right turn movements onto Huna Road.

A Basic Left Turn (BAL) widening treatment as detailed at Figure 8.2 of *Austroads Part 4A Unsignalsied and Signalised Intersections* is recommend for both Shaw Road intersections, with the widening on the Shaw Road (major road) approach to the intersection.

# 9. Assessment of Effects

## 9.1 Local Road Carriageway Widths

Daily traffic volumes on Huna Road between SH30 and the new subdivision access road will increase by 520 vpd, and on Shaw Road they will increase by 1,040 vpd up to the first subdivision access road intersection, and by 5,20 vpd up to the second intersection.

The resultant ADTs on the local roads will be:

- Huna Road 789vpd
- Shaw Road 1,194vpd

*Table 3.3 Rural Roads*, of Council's Engineering Code of Practice specifies minimum carriageway widths for rural roads. For local roads 6.0m is the specified seal width requirement.

On this basis both existing carriageways will continue to comply with Council's requirements for a rural local road.

However it is assessed that the section of Shaw Road that would carry an ADT of approximately 1,194vpd between SH30 and the first access intersection would warrant having its seal widened from the current 6.0-6.2m to a seal width of 7.0m (7.2m carriageway). This is the seal width requirement for a rural collector road, and the projected volumes are considered to be more consistent with this level of road status and carriageway width requirement.

Huna Road is already constructed to a 7.2m seal width and this is assessed as an appropriate width to accommodate the additional traffic.

Further, if the subdivision intends to provide an urban style frontage onto any of the existing roads with vehicle crossings providing individual lot access, then it will be necessary to apply the urban road design standards from the Engineering Code of Practice to the existing roads for the purpose of upgrading.

## 9.2 SH30 Intersections

The following four figures show the modelled flows adopted at the two intersections for the AM and PM peak hours. The flows represent a design year of 2022, with the base flows increased at a compounding rate of 1.14% over the surveyed 2012 flows. The full 200 lot subdivision flows have been added to the base flows in accordance with the traffic distribution described earlier.

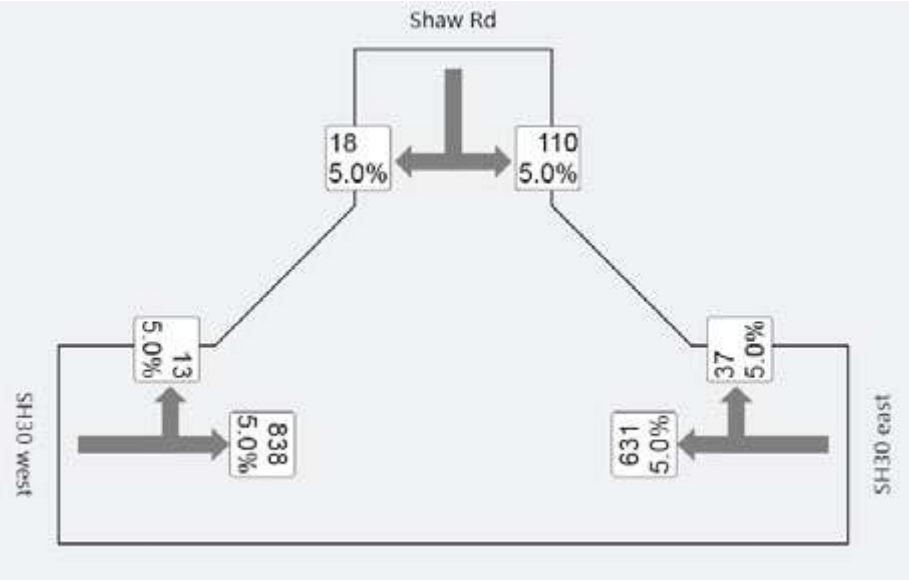


**Figure 4: Shaw Road AM Peak Hour Modelled Flows**

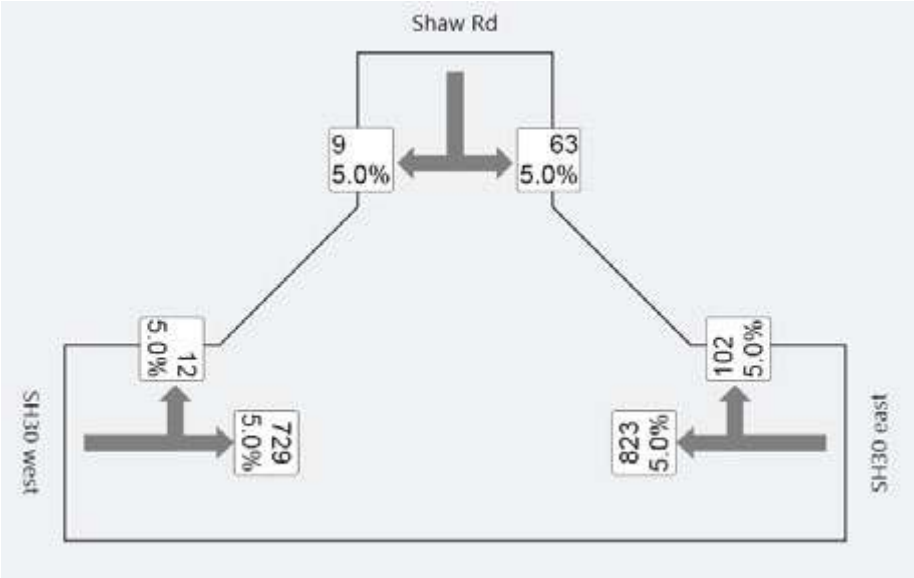


**Figure 5: Shaw Road PM Peak Hour Modelled Flows**

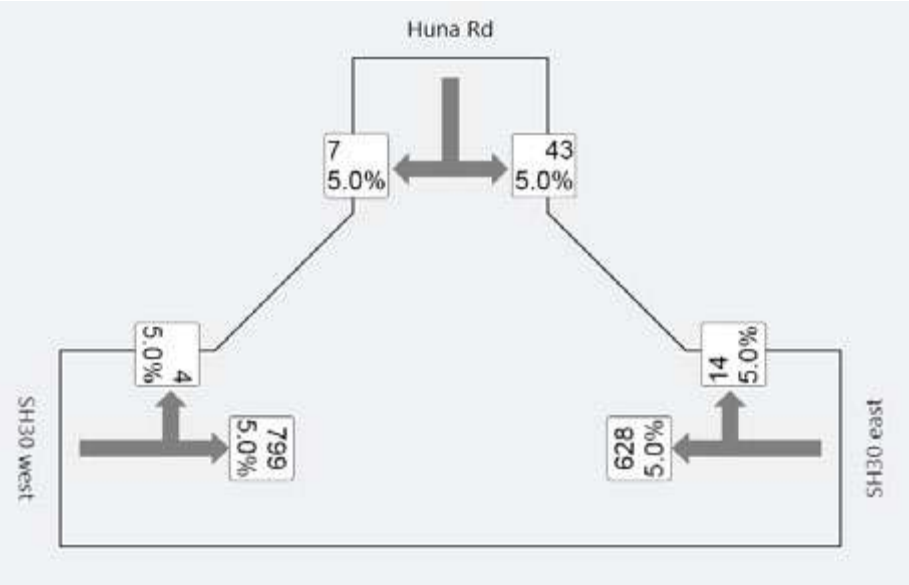


**Figure 6: Huna Road AM Peak Hour Modelled Flows**

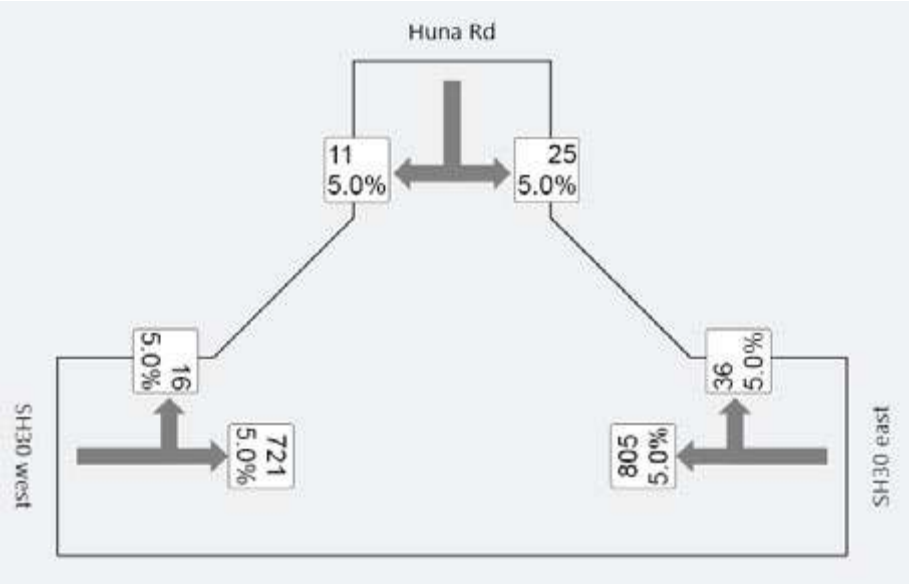


**Figure 7: Huna Road PM Peak Hour Modelled Flows**

The following tables summarise the results of the intersection modelling.

## MOVEMENT SUMMARY

**Site: SH30 - Shaw Rd 2022 AM**

**Movement Performance - Vehicles**

| Mov ID | Turn | Demand Flow | HV | Deg. Satn | Average Delay | Level of Service | 95% Back of Queue Vehicles | 95% Back of Queue Distance | Prop. Queued | Effective Stop Rate | Average Speed |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | veh/h | % | v/c | sec | | veh | m | | per veh | km/h |
| East: SH30 east | | | | | | | | | | | |
| 5 | T | 664 | 5.0 | 0.352 | 0.0 | LOS A | 0.0 | 0.0 | 0.00 | 0.00 | 60.0 |
| 6 | R | 39 | 5.0 | 0.095 | 16.3 | LOS C | 0.3 | 2.3 | 0.74 | 0.92 | 41.5 |
| Approach | | 703 | 5.0 | 0.352 | 0.9 | NA | 0.3 | 2.3 | 0.04 | 0.05 | 58.6 |
| North: Shaw Rd | | | | | | | | | | | |
| 7 | L | 116 | 5.0 | 0.382 | 24.6 | LOS C | 1.5 | 10.7 | 0.83 | 1.06 | 37.0 |
| 9 | R | 19 | 5.0 | 0.188 | 46.0 | LOS E | 0.5 | 3.9 | 0.92 | 1.01 | 27.0 |
| Approach | | 135 | 5.0 | 0.382 | 27.6 | LOS D | 1.5 | 10.7 | 0.84 | 1.06 | 35.2 |
| West: SH30 west | | | | | | | | | | | |
| 10 | L | 14 | 5.0 | 0.475 | 8.4 | LOS A | 0.0 | 0.0 | 0.00 | 1.09 | 49.0 |
| 11 | T | 882 | 5.0 | 0.475 | 0.0 | LOS A | 0.0 | 0.0 | 0.00 | 0.00 | 60.0 |
| Approach | | 896 | 5.0 | 0.475 | 0.1 | NA | 0.0 | 0.0 | 0.00 | 0.02 | 59.8 |
| All Vehicles | | 1734 | 5.0 | 0.475 | 2.6 | NA | 1.5 | 10.7 | 0.08 | 0.11 | 56.3 |

**Table 7: Shaw Road AM Peak Model Summary**

## MOVEMENT SUMMARY

**Site: SH30 - Shaw Rd 2022 PM**

**Movement Performance - Vehicles**

| Mov ID | Turn | Demand Flow | HV | Deg. Satn | Average Delay | Level of Service | 95% Back of Queue Vehicles | 95% Back of Queue Distance | Prop. Queued | Effective Stop Rate | Average Speed |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | veh/h | % | v/c | sec | | veh | m | | per veh | km/h |
| East: SH30 east | | | | | | | | | | | |
| 5 | T | 866 | 5.0 | 0.459 | 0.0 | LOS A | 0.0 | 0.0 | 0.00 | 0.00 | 60.0 |
| 6 | R | 107 | 5.0 | 0.208 | 14.7 | LOS B | 0.8 | 5.6 | 0.69 | 0.91 | 42.8 |
| Approach | | 974 | 5.0 | 0.459 | 1.6 | NA | 0.8 | 5.6 | 0.08 | 0.10 | 57.5 |
| North: Shaw Rd | | | | | | | | | | | |
| 7 | L | 66 | 5.0 | 0.169 | 18.7 | LOS C | 0.6 | 4.1 | 0.71 | 1.00 | 40.9 |
| 9 | R | 9 | 5.0 | 0.128 | 55.7 | LOS F | 0.3 | 2.5 | 0.94 | 1.00 | 24.1 |
| Approach | | 76 | 5.0 | 0.169 | 23.4 | LOS C | 0.6 | 4.1 | 0.74 | 1.00 | 37.6 |
| West: SH30 west | | | | | | | | | | | |
| 10 | L | 13 | 5.0 | 0.413 | 8.4 | LOS A | 0.0 | 0.0 | 0.00 | 1.09 | 49.0 |
| 11 | T | 767 | 5.0 | 0.413 | 0.0 | LOS A | 0.0 | 0.0 | 0.00 | 0.00 | 60.0 |
| Approach | | 780 | 5.0 | 0.413 | 0.1 | NA | 0.0 | 0.0 | 0.00 | 0.02 | 59.8 |
| All Vehicles | | 1829 | 5.0 | 0.459 | 1.9 | NA | 0.8 | 5.6 | 0.07 | 0.10 | 57.2 |

**Table 8: Shaw Road PM Peak Model Summary**

## MOVEMENT SUMMARY

**Site: SH30 - Huna Rd 2022 AM**

**Movement Performance - Vehicles**

| Mov ID | Turn | Demand Flow | HV | Deg. Satn | Average Delay | Level of Service | 95% Back of Queue Vehicles | 95% Back of Queue Distance | Prop. Queued | Effective Stop Rate | Average Speed |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | veh/h | % | v/c | sec | | veh | m | | per veh | km/h |
| East: SH30 east | | | | | | | | | | | |
| 5 | T | 661 | 5.0 | 0.350 | 0.0 | LOS A | 0.0 | 0.0 | 0.00 | 0.00 | 60.0 |
| 6 | R | 15 | 5.0 | 0.032 | 15.0 | LOS B | 0.1 | 0.8 | 0.69 | 0.88 | 42.6 |
| Approach | | 676 | 5.0 | 0.350 | 0.3 | NA | 0.1 | 0.8 | 0.02 | 0.02 | 59.5 |
| North: Huna Rd | | | | | | | | | | | |
| 7 | L | 45 | 5.0 | 0.134 | 20.2 | LOS C | 0.4 | 3.1 | 0.75 | 1.00 | 39.8 |
| 9 | R | 7 | 5.0 | 0.082 | 39.2 | LOS E | 0.2 | 1.3 | 0.89 | 1.00 | 29.5 |
| Approach | | 53 | 5.0 | 0.134 | 22.9 | LOS C | 0.4 | 3.1 | 0.77 | 1.00 | 38.0 |
| West: SH30 west | | | | | | | | | | | |
| 10 | L | 4 | 5.0 | 0.448 | 8.4 | LOS A | 0.0 | 0.0 | 0.00 | 1.10 | 49.0 |
| 11 | T | 841 | 5.0 | 0.448 | 0.0 | LOS A | 0.0 | 0.0 | 0.00 | 0.00 | 60.0 |
| Approach | | 845 | 5.0 | 0.448 | 0.0 | NA | 0.0 | 0.0 | 0.00 | 0.01 | 59.9 |
| All Vehicles | | 1574 | 5.0 | 0.448 | 0.9 | NA | 0.4 | 3.1 | 0.03 | 0.04 | 58.6 |

**Table 9: Huna Road AM Peak Model Summary**

## MOVEMENT SUMMARY

**Site: SH30 - Huna Rd 2022 PM**

**Movement Performance - Vehicles**

| Mov ID | Turn | Demand Flow | HV | Deg. Satn | Average Delay | Level of Service | 95% Back of Queue | | Prop. Queued | Effective Stop Rate | Average Speed |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | Vehicles | Distance | | | |
| | | veh/h | % | v/c | sec | | veh | m | | per veh | km/h |
| East: SH30 east | | | | | | | | | | | |
| 5 | T | 847 | 5.0 | 0.449 | 0.0 | LOS A | 0.0 | 0.0 | 0.00 | 0.00 | 60.0 |
| 6 | R | 38 | 5.0 | 0.073 | 14.1 | LOS B | 0.3 | 1.8 | 0.65 | 0.89 | 43.4 |
| Approach | | 885 | 5.0 | 0.449 | 0.6 | NA | 0.3 | 1.8 | 0.03 | 0.04 | 59.0 |
| North: Huna Rd | | | | | | | | | | | |
| 7 | L | 26 | 5.0 | 0.066 | 18.2 | LOS C | 0.2 | 1.5 | 0.68 | 1.00 | 41.3 |
| 9 | R | 12 | 5.0 | 0.159 | 48.9 | LOS E | 0.4 | 2.6 | 0.92 | 1.00 | 26.1 |
| Approach | | 38 | 5.0 | 0.159 | 27.6 | LOS D | 0.4 | 2.6 | 0.76 | 1.00 | 35.1 |
| West: SH30 west | | | | | | | | | | | |
| 10 | L | 17 | 5.0 | 0.411 | 8.4 | LOS A | 0.0 | 0.0 | 0.00 | 1.09 | 49.0 |
| 11 | T | 759 | 5.0 | 0.411 | 0.0 | LOS A | 0.0 | 0.0 | 0.00 | 0.00 | 60.0 |
| Approach | | 776 | 5.0 | 0.411 | 0.2 | NA | 0.0 | 0.0 | 0.00 | 0.02 | 59.7 |
| All Vehicles | | 1699 | 5.0 | 0.449 | 1.0 | NA | 0.4 | 2.6 | 0.03 | 0.05 | 58.4 |

**Table 10: Huna Road PM Peak Model Summary**

The modelling shows that all movements operate with a good level of service at both intersections during both peaks, with the exception of the right turn exit movement from both local roads onto SH30.

This (right turn out) movement is operating with delays that range between 39 seconds at Huna Road during the AM peak to 56 seconds at Shaw Road during the PM peak. However at both intersections the movement has very low volumes, and consequently queuing and intersection capacity are not problematic. At Shaw Road during the PM peak (the worst performing scenario) the 95$^{th}$ percentile queue is 0.3 vehicle lengths, and the movement volume to capacity ratio is 0.13.

Notwithstanding the low level of queuing and spare capacity, a 56 second delay is in the LOS F category and is less acceptable. While the capacity of the right turn movement is not a specific concern due to low volumes, such as is the case at the subject sites, the primary concern that remains as delay increases is that road safety will be adversely affected.

A further two scenarios have been modelled at the intersection of Shaw Road and SH30 for the 2022 design year, PM peak, being:

- without any subdivision traffic; and
- a 100 lot subdivision.

The results of this further modelling are summarised below.

## MOVEMENT SUMMARY

**Site: SH30 – No Subdivision Shaw Rd 2022 PM**

**Movement Performance - Vehicles**

| Mov ID | Turn | Demand Flow | HV | Deg. Satn | Average Delay | Level of Service | 95% Back of Queue Vehicles | 95% Back of Queue Distance | Prop. Queued | Effective Stop Rate | Average Speed |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | veh/h | % | v/c | sec | | veh | m | | per veh | km/h |
| East: SH30 east | | | | | | | | | | | |
| 5 | T | 837 | 5.0 | 0.443 | 0.0 | LOS A | 0.0 | 0.0 | 0.00 | 0.00 | 60.0 |
| 6 | R | 18 | 5.0 | 0.033 | 13.6 | LOS B | 0.1 | 0.8 | 0.63 | 0.83 | 43.8 |
| Approach | | 855 | 5.0 | 0.443 | 0.3 | NA | 0.1 | 0.8 | 0.01 | 0.02 | 59.5 |
| North: Shaw Rd | | | | | | | | | | | |
| 7 | L | 14 | 5.0 | 0.033 | 17.6 | LOS C | 0.1 | 0.8 | 0.66 | 0.98 | 41.7 |
| 9 | R | 3 | 5.0 | 0.032 | 43.7 | LOS E | 0.1 | 0.6 | 0.91 | 1.00 | 27.8 |
| Approach | | 17 | 5.0 | 0.033 | 22.5 | LOS C | 0.1 | 0.8 | 0.71 | 0.98 | 38.1 |
| West: SH30 west | | | | | | | | | | | |
| 10 | L | 3 | 5.0 | 0.399 | 8.4 | LOS A | 0.0 | 0.0 | 0.00 | 1.10 | 49.0 |
| 11 | T | 749 | 5.0 | 0.399 | 0.0 | LOS A | 0.0 | 0.0 | 0.00 | 0.00 | 60.0 |
| Approach | | 753 | 5.0 | 0.399 | 0.0 | NA | 0.0 | 0.0 | 0.00 | 0.00 | 59.9 |
| All Vehicles | | 1624 | 5.0 | 0.443 | 0.4 | NA | 0.1 | 0.8 | 0.01 | 0.02 | 59.4 |

**Table 11: Shaw Road PM Peak –No Subdivision Model Summary**

## MOVEMENT SUMMARY

**Site: SH30 - Shaw Rd 2022 PM with 100 lots**

**Movement Performance - Vehicles**

| Mov ID | Turn | Demand Flow | HV | Deg. Satn | Average Delay | Level of Service | 95% Back of Queue Vehicles | 95% Back of Queue Distance | Prop. Queued | Effective Stop Rate | Average Speed |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | veh/h | % | v/c | sec | | veh | m | | per veh | km/h |
| East: SH30 east | | | | | | | | | | | |
| 5 | T | 852 | 5.0 | 0.451 | 0.0 | LOS A | 0.0 | 0.0 | 0.00 | 0.00 | 60.0 |
| 6 | R | 63 | 5.0 | 0.119 | 14.1 | LOS B | 0.4 | 3.1 | 0.66 | 0.89 | 43.3 |
| Approach | | 915 | 5.0 | 0.451 | 1.0 | NA | 0.4 | 3.1 | 0.05 | 0.06 | 58.5 |
| North: Shaw Rd | | | | | | | | | | | |
| 7 | L | 40 | 5.0 | 0.099 | 18.2 | LOS C | 0.3 | 2.3 | 0.69 | 1.00 | 41.2 |
| 9 | R | 6 | 5.0 | 0.074 | 49.3 | LOS E | 0.2 | 1.5 | 0.92 | 1.00 | 25.9 |
| Approach | | 46 | 5.0 | 0.099 | 22.4 | LOS C | 0.3 | 2.3 | 0.72 | 1.00 | 38.2 |
| West: SH30 west | | | | | | | | | | | |
| 10 | L | 8 | 5.0 | 0.406 | 8.4 | LOS A | 0.0 | 0.0 | 0.00 | 1.10 | 49.0 |
| 11 | T | 758 | 5.0 | 0.406 | 0.0 | LOS A | 0.0 | 0.0 | 0.00 | 0.00 | 60.0 |
| Approach | | 766 | 5.0 | 0.406 | 0.1 | NA | 0.0 | 0.0 | 0.00 | 0.01 | 59.9 |
| All Vehicles | | 1727 | 5.0 | 0.451 | 1.2 | NA | 0.4 | 3.1 | 0.04 | 0.06 | 58.2 |

**Table 12: Shaw Road PM Peak Hour 100 Lot Subdivision Model Summary**

This modelling shows that under the base scenario with no subdivision, the right turn from Shaw Road is expected to be operating with a 44 second delay and LOS E by 2022. Therefore the proposed 200 lot subdivision results in a 12 second increase in the delay experienced by this movement as compared to the pre-existing base case.

Under the scenario with a 100 lot subdivision the right turn from Shaw Road is expected to be operating with a 49 second delay and LOS E by 2022. Therefore a staged 100 lot subdivision results in a 5 second increase in the delay experienced by the right turn out movement as compared to the pre-existing base case.

Based on the modelling done it is assessed that a 100 lot subdivision should be acceptable up to the design planning horizon of 2022, and that the 200 lot subdivision is marginally acceptable over the same time period.

However it must be acknowledged that the SH30 volumes are such that a right turn movement

from either Shaw Road or Huna Road onto the highway will be becoming difficult in the year 2022 regardless of whether this subdivision occurs or not. Further to this, if additional growth is to occur elsewhere or the underlying growth in traffic volumes exceeds the modelled value, then the intersections will come under additional pressure. It is further noted that any significant change to the assessed trip distribution, i.e. increase in traffic split to/from the west, would further increase the volume and delay for right turn out movements. Further sensitivity tests are necessary to understand the effects of such changes to the distribution pattern.

Ideally a strategy for the highway is needed that considers all expected growth in the area, and a Structure Plan for the development of the surrounding land.

Further work is required to determine the timing / trigger for the need to upgrade access from the subdivision site onto SH30. This upgrade to the access onto SH30 could involve the upgrade of one or both intersections, or alternatively the construction of a single new intersection to replace them both. Any need for an upgrade of the intersections would therefore result from increases to SH30 volumes as much as the proposed development itself.

In addition to determining the location of any upgraded access to the highway there are also options regarding the intersection form, with either a seagull type Tee intersection or a roundabout being viable alternatives.

Seagull intersections permit the right turn exit from a side road to be undertaken in two stages, with drivers required to give way to traffic from their right first, to reach a sheltered median, after which they utilise an acceleration lane before merging with the traffic from their left.

A Seagull type Tee intersection would provide no effective delay to the highway through traffic and would suit a situation such as exists at the subject intersections, where the right turn volumes are relatively low, but receiving a poor LOS due to limited gaps in the priority traffic stream.

A roundabout is a more significant investment and more substantial intersection form. A roundabout is typically suited to situations where the intersecting roads have more evenly matched volumes of traffic and importance in a roading hierarchy. At a roundabout all traffic is delayed to some degree due to the geometry and low volume roads typically operate with a high LOS. Roundabouts are widely acknowledged to be the safest form of at-grade road intersection.

## 9.3 Wider Network Effects

### 9.3.1 Current Flow Observations

Whakatane District Council have advised that morning, evening and holiday peak times traffic queues are intermittently extending back well in advance of the Landing Road bridge and in the morning peak, traffic coming into town can be queued for up to 2km west of the Landing Road roundabout (i.e. beyond the bridge and two existing roundabouts on SH30). In the evening peak, queues have been observed for a similar distance, from the Landing Road bridge, back down the entire length of Landing Road and Domain Road, and back down McAlister Street.

### 9.3.2 Previous Studies

It is stated in the *Whakatane Integrated Urban Growth Strategy* (2010) that previous modelling has shown that *"A new State Highway river crossing will be required regardless of options. The Whakatane River bridge will require additional capacity by 2016*".

Furthermore, the draft "*Whakatane Access and Security Scoping Study*" concluded that:

"*Forecast growth rates from both historic trends and the WRTM model show modest increases in traffic in the future, and so significant widespread congestion is not likely to occur based on the evidence presented to date. The exception to this is the Landing Road/Domain Road roundabout where the existing queues may start to effect capacity at adjoining intersections and could produce some wider delays if not addressed*".

### 9.3.3 Effects of Huna Road / Shaw Road Development

The residential growth demand for the district has been forecast by WDC and the "medium growth" scenario has been previously modelled using the Regional Model (WRTM). The addition of 200 lots as proposed by this development will fulfil some of that already anticipated future growth.

On this basis, it is considered the wider network effects of this development have already been assessed as part of the WRTM modelling to date which have identified that upgrades will be required in the future at the Landing Road Bridge and the Bridge/Landing Road roundabout.

The timing of those required upgrades may be brought forward by the proposed development if the development progresses faster than has been previously modelled for residential development as a whole on the west side of the river. To understand the effect of the development on the timing of the upgrades will require further assessment of the WRTM modelled assumptions with respect to the location and growth expectations.

Notwithstanding the growth already anticipated within the district, the proposed development when complete will potentially add an additional 200 veh/h to the state highway network in the peak hour periods. Not all of this traffic will use the Landing Road Bridge as "The Hub" retail area will attract some of the trips and reduce the need to travel further with others travelling to/from the west. Conservatively based on 80% of the traffic crossing the bridge, and based on the distribution described in this report. The morning peak increase in eastbound movements at the bridge is 120 veh/h. This additional generation compares with the existing eastbound flow of 811 veh/h (June 2012 morning peak). While this increase in flow can be expected to increase the queuing and delay at the Landing Road roundabout, the flow is within the expected capacity of the two way bridge itself. Again the timing of any bridge upgrade will be sensitive to the future growth rates.

Any increase in queuing or level of service reduction will likely result in peak spreading or a natural adjustment of the trip distribution times as a result of the poor level of service with the net effect being a lengthening of the peak flow periods over which the high flows occur.

It is noted that the peak hour flows have actually been decreasing over recent years, however it is recognised that there are still times of particularly heavy traffic flow such as Easter weekend and other holiday periods where the capacity of the existing SH30 network between Keepa Road and Landing Road is reached or exceeded.

## 9.4 NZTA Consultation

A draft of the Scoping Report was sent to NZTA for comment, and the following key issues summarise the feedback received from NZTA on 30 October 2012:

- A suggestion that Whakatane District Council and NZTA work together to develop a network master plan for the area (Urban Growth areas) that would identify traffic growth

and the impacts of that growth for the wider area, including appropriate mitigation and funding mechanisms;

- NZTA advised they may be able to support this current proposal once the network plan was complete and a clear investment strategy for the local network developed;
- Specific comments related to the Shaw/ Huna Road development:
  i) Retain current "t"- intersection layouts as neither seagulls nor a roundabout are supported due to safety concerns. A widened shoulder is preferred with costs to be met by the developer;
  ii) Internal link road between Huna and Shaw Roads is supported;
  iii) Query with respect to the provision for cyclists and pedestrian movements between the development and Whakatane or the Hub retail centre.

# 10. Conclusion

This Scoping Assessment has been prepared with the objective of identifying any significant transportation issues that require either further investigation or potential mitigation in order to manage the additional traffic associated with the development of a 200 lot residential subdivision on a 21ha block bounded by Huna Road, Shaw Road and SH30.

The proposed residential site is located within a larger area referred to as "*West of Keepa Road* " that has been identified by Whakatane District Council as a future residential expansion area in the *Whakatane Integrated Urban Growth Strategy.* However no structure plan or analysis of the growth west of Keepa Road has been undertaken at this stage and therefore this current proposal has been assessed in isolation of the necessary development of Council's long term growth plans for the area.

In the absence of a Structure Plan for the site, this report has assessed and recommended feasible access location options for the site based on the current road environment. Locations have been identified for both Huna Road and Shaw Road access which shows that access is feasible from the side roads, although the District Plan requirements for intersections would assist to cover the specific requirements for design.

The effects of the development on the adjacent road network have been assessed and the following issues have been identified as either requiring further investigation and/or mitigated in order for the transportation effects of the development to be adequately managed:

- Shaw Road to be upgraded between SH30 and the subdivision access to either rural collector standard (7.2m carriageway) or, alternatively, urban standard if the subdivision was to have direct property access to the local road. In either case a footpath and kerb and channel is desirable on the frontage of the subdivision. Similarly, Huna Road whilst already meeting a rural standard should be considered for an urbanised road frontage.
- Modelling of the SH30 intersection with Shaw Road based on forecast flows for 2022 indicates that right turn movements without the subdivision are operating with high delays due to the high volume of traffic on SH30. Further investigation is recommended to confirm the forecast flows and growth on SH30. The addition of 200 lots increases the volume of right turn movements and hence delays to a Level of Service F. The number of vehicles queued is small however the length of delay is of potential safety concern which will only grow as traffic on SH30 increases as expected in future years. If the subdivision was approved on the basis of insignificant traffic flows now or even as a staged approach (modelling has shown that a 100 lot stage could be managed in the current environment up to 2022), at some stage in the future an intersection upgrade would likely be necessary.

  Should an improved access be necessary to accommodate the existing and proposed flows, this can be located at either Shaw Road, Huna Road or somewhere between with a connection between Shaw and Huna Roads upstream of the intersection. Each has its own issues and merits with Shaw Road being closest to town and a combined intersection requiring re-routing of Shaw and Huna Road traffic through a residential subdivision. Whichever option is adopted should consider the implications of further growth west of Keepa Road as noted in the growth study.

  Several options have been considered for the form of intersection upgrade include either a seagull channelized layout or a roundabout. Additional investigation required to identify the preferred option. The seagull layout will likely be the most cost efficient to achieve while a roundabout is regarded as the safest option but will require a large inscribed diameter and therefore may require extensive widening and/or land requirements.

However, a major change to the intersection form would introduce potential delays to the highway traffic and the dis-benefits are likely to outweigh the benefits to a small volume of right turning traffic. NZTA have advised that they support retention of the current intersection layout with some minor shoulder widening.

Further work, including sensitivity testing, is required to determine the necessity for and timing of any proposed upgrade of the SH30 intersections with consideration to the likely programme for the site development and the overall strategy for the highway including all expected growth in the area.

- The effects on the wider arterial network between the subdivision and CBD have been previously recognised in previous investigations by Council. The residential growth demand for the district has been forecast by WDC and the "medium growth" scenario has been previously modelled using the Regional Model (WRTM). The addition of 200 lots as proposed by this development will fulfil some of that already anticipated future growth.

  On this basis, it is considered the wider network effects of this development have already been assessed as part of the WRTM modelling to date which have identified that upgrades will be required in the future at the Landing Road Bridge and the Bridge/Landing Road roundabout. This development may potentially accelerate the timing of these upgrades.

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October 2012

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Dear Michal

**Shaw Road/Huna Road Residential Subdivision**
**Traffic Effects Assessment Addendum: State Highway Access**

Further to the Traffic Design Group Ltd Scoping Assessment for the proposed Huna/Shaw Road Residential zoning (October 2012), we have given further and more detailed consideration to the recommended SH30 intersection layouts appropriate to accommodate the expected traffic generated by the proposal.

The scope of this assessment is intended as an addendum to the original Scoping Assessment with a focus on the two following matters as requested:

- Recommendations as to how Shaw Rd and Huna Rd should be upgraded to cater for the proposal; and
- Recommendations as to whether the internal linkage between Shaw Rd and Huna Rd is fundamental to the proposal and any traffic solutions.

## 1. Previous Assessment

Modelling of the SH30 intersection with Shaw Road and Huna Road based on forecast flows for 2022 indicates that right turn movements in the base case scenario (no development) would operate with high delays due to the high volume of traffic on SH30. All other movements were demonstrated to perform satisfactorily for the proposed 200 lot development.

The addition of 200 residential lots increases expected average delays for right turn out movements in the evening peak to around 56 seconds based on the default SIDRA modelled equations. However, the number of vehicles undertaking this movement is small and the 95%ile queue is less than one vehicle.

The Scoping Assessment concluded that on the basis of the expected insignificant traffic flows that would be undertaking the critical right turn out movement, the subdivision effects could be managed in the current environment up to 2022. At some stage in the future an intersection upgrade would likely be necessary.

However, it was acknowledged that the SH30 volumes are such that a right turn movement from either Shaw Road or Huna Road onto the highway will be increasingly difficult in the

year 2022, and beyond, regardless of whether this subdivision occurs or not. Further to this, if additional growth is to occur elsewhere or the underlying growth in traffic volumes exceeds the modelled value, then the intersections will come under additional pressure.

Ideally a strategy for the highway is needed that considers all expected growth in the area, and a Structure Plan for the development of the surrounding land.

Further work was recommended to determine the timing / trigger for the need to upgrade access from the subdivision site onto SH30, including sensitivity tests to understand the effects of potential changes in the distribution pattern, state highway traffic growth and modelling parameters.

NZTA have advised that they support retention of the current intersection layout with some minor shoulder widening as detailed in the following response:

1. *Retain 'T intersections' with Shaw and Huna Road. Seagulls with acceleration lanes or a roundabout are not supported due to safety concerns. A widened shoulder is preferred along Huna Road and Shaw Road to accommodate turning movements. The cost of these works should be met by the developer.*
2. *The NZTA supports the proposal for a local road linkage between Shaw and Huna Roads as it recognised that this will reduce the impact of development generated traffic on SH30.*
3. *There is no current provision for pedestrians or cyclists to and from the Hub or Whakatane, how will this be accommodated?*

## 2. Modelling Sensitivity

The modelling of priority controlled intersections is less than straight forward - particularly when modelling intersections with high through flows such as currently experienced at both Huna Road and Shaw Road intersections with SH30.

The intersection modelling is sensitive to a number of parameters which are required to be assessed in the absence of site specific data, including the gap acceptance parameters (*critical gap* and *follow up headway)* and the type of capacity model adopted. The model outputs are, in turn, subject to different interpretation as to their relevance. The importance of these assessments is expanded on below:

- Gap acceptance and follow up headways vary considerably depending on the available source of data. Austroads guides provide one source although not without confusion: the Austroads Road Design Guide 4A (2010) is based on earlier Austroads guides (2002 and 2005) while the Austroads Traffic Management Guide is based on Highway Capacity Manual (HCM) 2010. The US HCM 2010 are notably greater than the Austroads values, however driver behaviour at priority controlled intersections in the USA (which has numerous four way priority controlled junctions) differs from New Zealand and Australia.
- Gap acceptance parameters could reasonably be expected to decrease with increasing opposing flows such as experienced on the heavily loaded SH30
- Various capacity models result in differing intersection performance outputs. SIDRA Intersection software includes four of these models ranging from the "traditional" Austroads (ex HCM) model, to the Akcelik models which include both a simple exponential model and a bunched exponential model. SIDRA recommends use of the bunched exponential as a default but recognises that this model has one of the most conservative capacity forecasts of any of the models.

- The delay thresholds that are acceptable for intersection performance vary depending on jurisdiction, and the maximum acceptable delay is subjective and needs to consider all variables including flow and intersection specific layout.

The numerous modelling parameters that must be assessed increases the uncertainty of any particular result. Calibration of the intersection modelling is the best way to improve certainty, although this is difficult to achieve with the current low volumes of side road traffic which reduces the sample size.

To account of this uncertainty, a number of sensitivity checks have now been undertaken for the modelled results previously reported. Sensitivity of the modelling in terms of the capacity equation, the gap acceptance parameters and flow distribution has been investigated:

- Use of the traditional simple exponential models reduces the critical movement (right turn out) average delay to 30 – 35sec (from the 56sec noted above for the base model)
- Variation of the gap acceptance parameters (80% to 90%) reduces average delay to 30 and 40 sec respectively. Figure 1 below depicts the delay performance sensitivity with respect to gap acceptance parameters. Notably the 80% gap acceptance parameter matches closely the recommended values for a three way give way controlled intersection with two approach lane which the current layout is expected to function in a similar fashion.
- Variation of the critical movement flow has been analysed based on an increase in movements to/from the west to 30% which has resulted in less than 5% change to the modelled delays. This reflects the expectation that intersection delay is more sensitive to the state highway flows than to variation in side road flows at these levels.

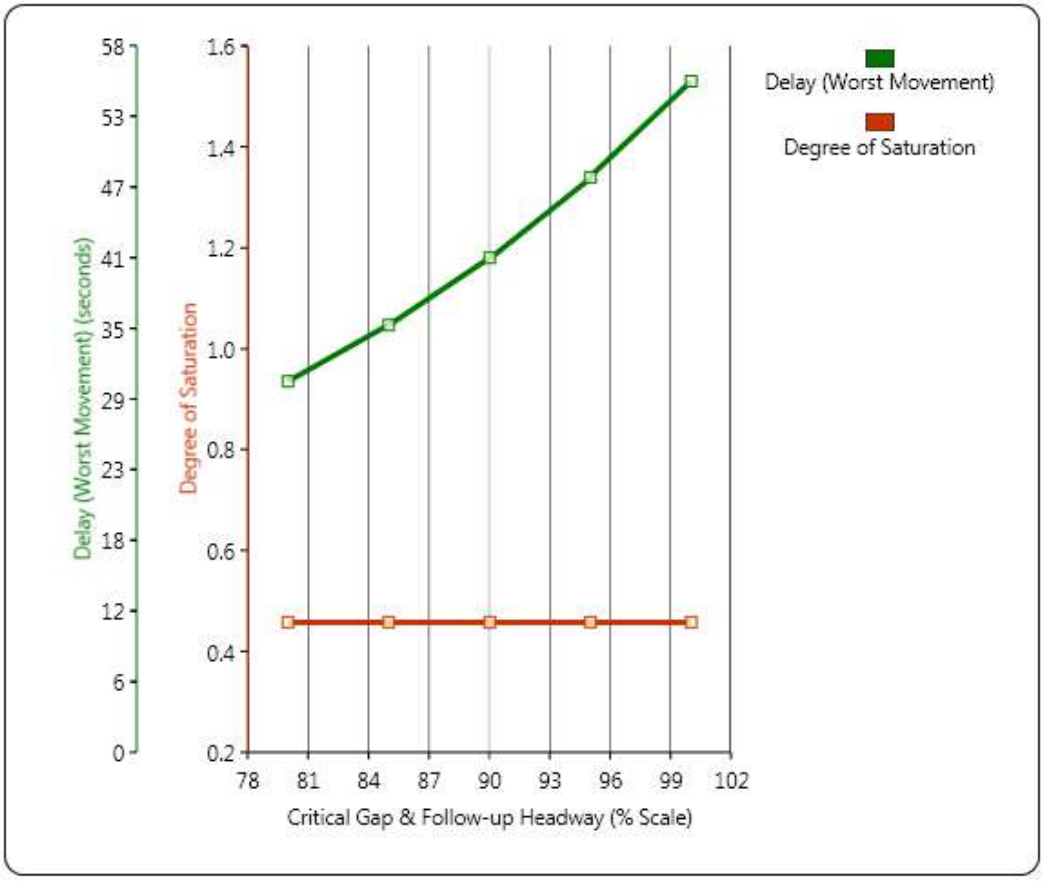


**Figure 1: Sensitivity of Average Delay to variation in Gap Acceptance Parameters**

## 3. Discussion and Conclusion

### 3.1 SH30 Access Mitigation

Further to the initial Scoping Assessment, additional sensitivity tests have been undertaken to verify the report conclusions. Based on the above assessment and consideration of the likely network operation and driver behaviour it is concluded that the current SH30 intersection form (for both Shaw Road and Huna Road) will be capable of managing the traffic flows associated with the development.

NZTA have commented on the side road shoulder width on the approach to SH30 and the potential need for shoulder widening. The combined land and shoulder width on the Shaw Road approach is nominally 6.0m over the first 20m from the limit line which would be adequate to cater for a long truck with room to be passed by another vehicle, and no further widening is considered necessary. The Huna Road approach has narrower shoulders and a short section of widening is necessary to cater for passing of a long truck stopped at the limit line. It is therefore recommended that a short section of shoulder widening is undertaken on the Huna Road approach (to provide a width of 6.0m over a minimum length of 20m from the limit line).

In coming to the above conclusion, the following relevant factors have been considered:

- The volume of right turning out traffic is very small at around 10 to 15 veh/hour in the evening peak and the average queue is less than one vehicle. Therefore, while delays to this small number of vehicles are relatively high, the pressure on drivers to make an unsafe turn, is limited.
- The modelling is considered conservative. In reality actual gap acceptance parameters adopted by drivers are likely to reflect that there is a short merge area available for right turn out movements enabling a driver to accept shorter gaps in the westbound traffic stream and potentially a two stage movement. Further, as noted above, gap acceptance parameters can be expected to decrease as traffic volumes increase, particularly as volumes approach saturation, and there are several alternative (and less conservative) capacity models that may be considered to that used in the default SIDRA analysis.
- The proposed internal access road provides an internal connection between Huna Road and Shaw Road. This enables drivers to adopt whichever state highway intersection route is most efficient at the time, whether due to greater pressure on one or other, or based on his destination. In this manner, Shaw Road is likely to be preferred for Whakatane orientated movements and Huna Road for west orientated movements. Further, the road network provides alternative, albeit longer, routes to Thornton Road in the west and Keepa Road to the east. Alternatively, traffic is able to left turn out of the side road and travel to the next state highway intersection to make a turn if necessary.
- NZTA have advised that they prefer to see the current intersection form retained. Furthermore, it is understood that NZTA are considering a reduction of the speed limit through these intersections to 80km/h and such a reduction would improve safety at the intersections.

### 3.2 SH30 Access – No Internal Link Road

In the scenario where no internal subdivision road was to be developed the linkage between Huna Road and Shaw Road would not be available. In this case the flexibility of traffic from the development to use either SH30 intersection or alternative routes would be reduced.

This scenario is based on approximately half of the development site, or 100 lots, to be accessed from each of Huna and Shaw Roads respectively. On this basis there would be an even distribution of traffic to each intersection as opposed to the west and east bias that would result from the availability of an internal connection. However, depending on final distributions, the change in right turn movements is less than 10 veh/h. This change, as before, coupled with the other considerations outlined in 3.1 above, is less than minor and therefore similar mitigation could reasonably be expected for this scenario as for the base scenario above.

However, in recognition of the lack of flexibility in route choice with this option, it is recommended that a review clause is included with this option that requires monitoring of the performance of both intersections following development with the implementation of improvements to the intersections if the review determined the necessity. The improvements would involve widening of the state highway to lengthen the right turn out merge lane to around 80m or more. It is suggested that the exact wording of such a review clause would need to be discussed and agreed with NZTA.

Apart from the effects of the omission of an internal road connection on the state highway intersections, the lack of internal connection is not desirable from an urban design perspective. The segregation of the development into two separate entities would require trips between the neighbourhoods whether by vehicle, cycle or foot to have to utilise the state highway network instead. While less desirable than an internal connection, as a minimum in this scenario an additional cycle / pedestrian linkage is recommended to be provided along the state highway frontage linking the two neighbourhoods.

Wherever possible good linkage is provided internally within urban zoned land and desirably reliance should not be placed on the state highway to achieve this linkage. Therefore, it is recommended that provision should be made in the structure plan for this residential zone for an internal link to be completed even if that link is developed in stages as each landowner develops their particular lot.

Notwithstanding the conclusions of this addendum, it is reiterated that ideally a strategy for the area is needed that considers all expected growth in the west, and a Structure Plan for the development of the surrounding land, if any. Development of a strategy would necessarily include consideration of wider network effects including the Landing Road Bridge and adjacent intersections, and confirmation that the growth from this development is reasonably reflected in Council's current transport model and adopted growth scenario. It has previously been identified that future upgrades will be required at the Landing Road Bridge and the Bridge/Landing Road roundabout and therefore this development may potentially accelerate the timing of these upgrades.

Yours faithfully
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# Stormwater Management
# Feasibility Study
## For
## Shaw Road, Whakatane

PREPARED BY

**STORMWATER SOLUTIONS CONSULTING LTD**

5TH FEBRUARY 2013

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Project information

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Date: 5th February 2013

Reference: 1368- Shaw Road

Status: SW management feasibility study

Project Manager:

Bronwyn Rhynd - Director

# 1 Introduction

The properties located adjacent to State Highway 30 (SH30) between Huna and Shaw Road, Whakatane, are currently under application for zone change from Rural 1 (Plains) to Residential. A feasibility study and options assessment has been undertaken with respect to the stormwater management system for these properties.

The purpose of this report is to support the zone change application by Whakatane District Council (WDC). Treatment, conveyance and discharge to the Kopeopeo Canal are the components of the stormwater management system, which will be described in general form along with the associated Environment Bay of Plenty (EBoP) and Whakatane District Council consent requirements.

This feasibility and options assessment has been commissioned by PAG Enterprises Ltd, who is the owner of 220 SH30.

# 2 Description of the site and proposed plan change

The 21.75ha site, subject of the re-zoning application, is located between Huna and Shaw Roads, bound by State Highway 30 to the south and rural land to the north. The location is shown indicatively in Figure 1, and more specifically within Ross Overington Surveyors Ltd drawing 2910/2, appendix A.

**Figure 1 : Site location[1]**

The site is located in the Kopeopeo East Canal catchment, which is part of the overall Rangitaiki Drainage District and Whakatane Waimana Rivers Scheme. The majority of the site falls towards Marshalls Drain which runs adjacent to State Highway 30 along the southern boundary. This drain discharges through a culvert, under Shaw Road, to Kopeopeo East Canal (the canal). The canal ultimately discharges through floodgates and pumps to the lower tidal reach of the Whakatane River.

[1] Reference: Google maps www.google.co.nz

The site is currently utilised for various purposes, with Julian's Berry farm located in the western portion and PAG Enterprises olive grove in the eastern portion. The remainder of the area is a mix of agricultural and residential. The ground contour is low lying adjacent to SH30 at RL1.5m and rises to RL7.0m (Moturiki Datum) to accommodate a low sand dune feature which is aligned with the northern boundary. There is a small low lying area in the north eastern corner which is north of the sand dune feature.

The sub soil characteristics follow the old sand dune system in the higher locations with Kopeopeo sand loam and in the lower area soils are Paroa mottled silt loam.[2]

WDC have proposed a zone change for this area from Rural 1 (plains) to residential which is to be notified early 2013. This zone change is to meet the residential demands for the estimated growth within the Whakatane district.

## *2.1 Stormwater runoff (Peak flows and volumes)*

An assessment of the peak flows and volumes of stormwater runoff generated from this site under its current usage, being rural purposes, has been undertaken. The modelling was undertaken based on 1 hour duration storm event using the Rational Method, as per EBoP and WDC guidelines. The outcomes of this assessment are presented in Table 1.

**Table 1: Existing peak flows**

| Rainfall event | Peak flow ($m^3/s$) |
|---|---|
| 10yr ARI (10% AEP) | 0.6 |
| 100yr ARI (1% AEP)[1] | 1.01 |

Notes:

1. 100yr ARI rainfall event including the effects of climate change

# 3 Receiving environment

The receiving environment for the stormwater discharge from the site and catchment is the Kopeopeo East Canal, see Photos 1 and 2. The existing discharge pipeline outlet is located approximately 3000m upstream of the confluence with the Orini Canal. The 25m wide Kopeopeo Canal is stop banked waterway which approximately 10 km long and joins the Orini canal 500m upstream of a flood gated outlet to the Whakatane River. The Whakatane River then meanders a further 1000m downstream through estuarine environment to meet the coast line.

[2] Coffey Geotechnics Ltd, geotechnical assessment, November 2012 (Coffey 2012)

**Photo 1: Kopeopeo Canal at Shaw Road & SH30 intersection**

**Photo 2: Kopeopeo Canal approximately 800m downstream of site**

The Kopeopeo Canal forms part of the Rangitaiki Plains Drainage scheme. It is controlled by a series of pumps and floodgates with the main control at the floodgates 500m downstream of the confluence with Orini Canal. This flood gate controls the tidal influence for the Whakatane River and also the discharge from the drainage scheme. The Kopeopeo Pumpstation is located approximately 1000m upstream of the floodgates, within a side canal and adjacent to the Whakatane River stop bank. This pumpstation controls flows from this canal system during rainfall events, together with the operation of the flood gates further downstream (as previously mentioned).

The Whakatane River is noted for presence of the following flora and fauna (EBoP Proposed Land Management Plan); koaro, water birds, whitebait and

trout. In addition there are banded dotterel breeding grounds and whitebait spawning sites. The discharge from the Kopeopeo Canal to the Whakatane River is blocked due to the presence of a flood gate for fauna migratory purposes.

# 4 Proposed site development

A typical residential layout has been developed for the purposes of assessing the stormwater management options. The layout is shown in Ross Overington Surveyors Ltd drawing 2910/2, Appendix A.

The site has a lower lying area adjacent to State Highway 30, which also has a 40m buffer for Transit NZ purposes. This area will be raised slightly in the earthworks phase of the development to provide for stormwater management which includes the storage of runoff during large rainfall events.

Another area where surface levels will be raised is in the north eastern corner, refer to drawing 1368/SK01, Appendix A. This portion of the site is separated in level from the southern area by the "sand-dune" that runs across the site in an east-west direction. Therefore to allow for conveyance of runoff to the Kopeopeo Canal this area will need to be altered (with respect to surface levels).

## *4.1 Stormwater runoff (Peak volumes and flows)*

The stormwater runoff generated from the site under fully developed (residential) conditions has been assessed based on the following design parameters:

- NIWA HIRDs rainfall data[3]
- Site specific temporal rainfall pattern and unit hydrograph
  - Temporal rainfall pattern guidance provided by Roger Waugh, Principal Technical Engineer, EBoP
- 70% impervious coverage
- 100yr rainfall event, including effects of climate change

The outcomes of the peak flows and volumes are presented in Table 2.

**Table 2: Fully developed site peak flows and volumes**

| Rainfall event | Peak flow ($m^3/s$) | Volume ($m^3$) |
|---|---|---|
| 10yr ARI (10% AEP) | 4.05 | 20963 |
| 100yr ARI (1% AEP) | 8.10 | 42584 |
| Water quality[1] | - | 3714 |

Notes:
1. Water quality event is the 90 percentile rainfall event for this area

[3] NIWA High Intensity Rainfall System V3

# 5 Stormwater management

Stormwater management options have been developed for the whole proposed residential zone area (site), being 21.75 ha. The options presented give guidance as to appropriate stormwater management applications and can be considered as a tool box of options that can be applied to a residential development within the site.

The development of the stormwater management options has taken into consideration good engineering practice principles as well as the following:

- Environment Bay of Plenty (EBoP)
    - Draft Hydrological and Hydraulic guidelines
    - Draft Stormwater Management Guidelines for the Bay of Plenty Region
- Whakatane District Council (WDC)
    - Engineering Code of Practice

The proposed stormwater management system consists of treatment, conveyance, as well as discharge to the Kopeopeo East canal in the vicinity of State Highway 30 (SH30) and Shaw Road intersection. Details of the conveyance and treatment components are presented in the following sub-sections with discharge to Kopeopeo Canal in Section 6.

## *5.1 Stormwater Treatment*

The stormwater runoff from the proposed fully developed site is to be treated for removal of suspended solids and other contaminants associated with a residential development that can be adsorbed to the suspended solids or removed by bio-retention, such as nitrogen reduction.

Various stormwater treatment devices can be adopted for the stormwater treatment within the site, which include but not limited to the following:

- Stormwater pond
- Raingardens
- Swales
- Proprietary units, such as StormFilters
- Source control for the roof runoff

A preliminary assessment has been undertaken for the above devices to establish rough order dimensions for each device. It is to be noted that these ballpark figures are for information only when they are considered in the overall scheme planning.

### 5.1.1 Stormwater pond

A stormwater treatment pond can be installed at the low point of the site to provide end of pipe treatment for the entire site. The typical pond characteristics are shown in Table 3.

**Table 3: Typical Pond Characteristics**

| **Pond Characteristics** | |
|---|---|
| Pond footprint[1] | 6000m$^2$ |
| Levels | |
| *Top of the pond* | *RL2.0m* |
| *Permanent water level* | *RL1.0m (assumed)* |
| *Base of the pond* | *RL0.0m* |
| Permanent storage volume | 3700m$^2$ (min) |
| Side slope | 5(H) to 1(V) |

Notes:

1. The pond foot print is at RL 2m and assumes filling of surrounding low lying area. Storage volume at foot print area will allow for a certain amount of detention volume

It is to be noted that the pond can be constructed in stages to accommodate the progression of development within the site, shall that occurs. An indicative location for a pond is shown in drawing 1368/SK01, Appendix A.

### 5.1.2 Raingardens

Raingardens can be installed within the proposed road reserve to provide treatment for the road runoff. A preliminary assessment based on the Stormwater Management Guidelines for the Bay of Plenty Region has revealed a raingarden with surface area of 30m$^2$ is required to treat every 2000m$^2$ of the contributing catchment, which equates to 100m length of road reserve, with 80% impervious cover. A typical raingarden schematic is shown in Figure 2.

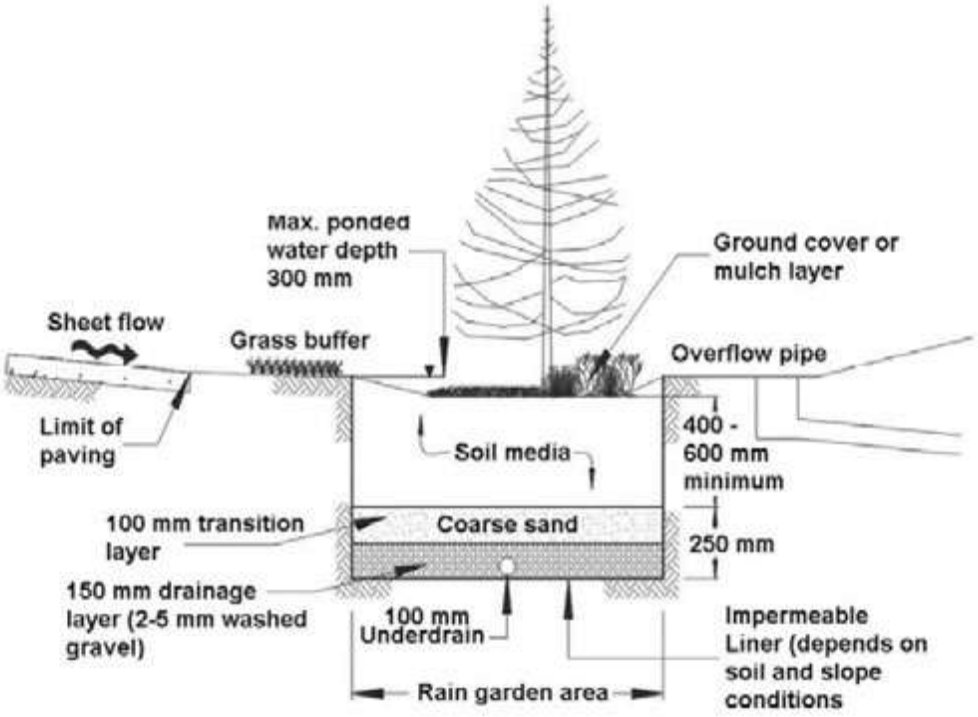



**Figure 2: Typical Raingarden Schematic[4]**

It is to be noted that the impermeable liner and/or the underdrain may not be required for the site, depending on the natural subsoil percolation rate. This is to be determined in the detailed design stage.

### 5.1.3 Swales

Swales can be installed along the road reserve to provide the same level of stormwater treatment as raingardens. The typical swale characteristics are shown in Table 4, with typical cross section shown in Figure 3.

[4] Refer to EBoP Stormwater Management Guidelines Figure 9.12

**Table 4 : Typical Swale Characteristics**

| **Swale Characteristics** | |
|---|---|
| Typical Length | 40m |
| Typical Width | |
| *Top width* | *2.6m* |
| *Base width* | *1.0m* |
| Typical Depth | 0.2m |
| Longitudinal Slope | To suit road gradient |
| Side slope | 4(H) to 1(V) |
| **Catchment Serviced** | |
| Catchment Area | 2000m$^2$ |
| Impervious coverage | 80% |

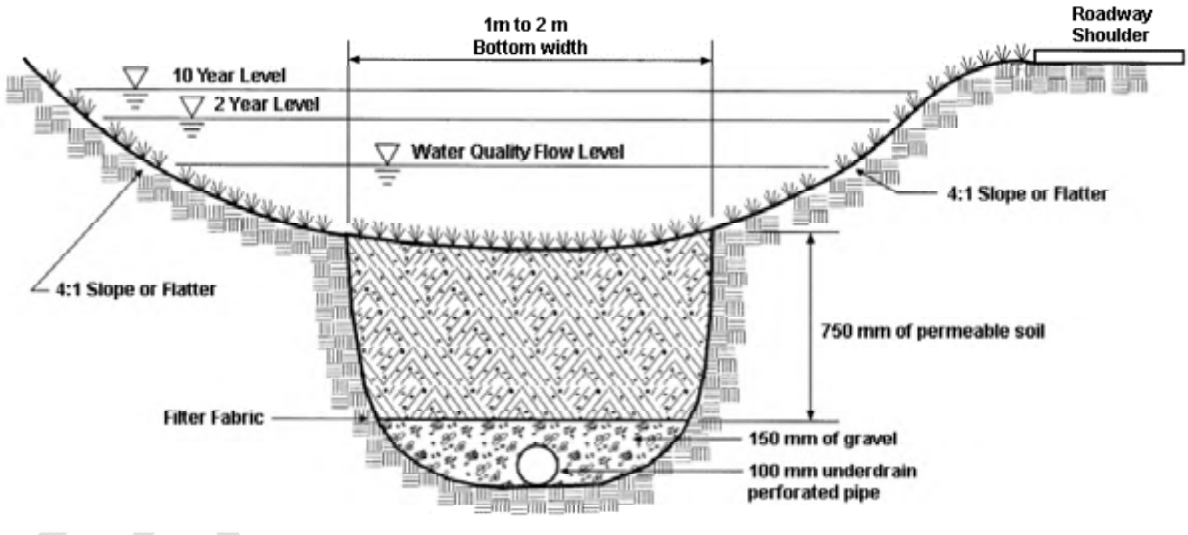



**Figure 3: Typical Swale Schematic[5]**

[5] Refer to EBoP Stormwater Management Guidelines Figure 9.3

### 5.1.4 Proprietary units

Proprietary units such as StormFilters supplied by Stormwater360 can be adopted for the road runoff treatment. The StormFilter units have been approved by Auckland Council as ARC TP10 compliant and can be buried underground.

### 5.1.5 Source control for the roof runoff

The roof runoff will be controlled through the selection of roofing materials that are of low contaminant generation, such as painted zinc roofing or Colorsteel. Therefore, no additional treatment is required for the roof runoff.

## *5.2 Stormwater Conveyance*

A stormwater conveyance system will service the fully developed site, which consists of the following:

- Primary network
- Secondary overland flow paths

This system will be designed to accommodate the downstream tail water effects from the Kopeopeo Canal during various flow situations. The discharge of stormwater runoff from the site (to the canal) can be by gravity or pumped to overcome the head difference between the site and the canal.

The internal conveyance system is discussed in the following sub sections with the discharge to Kopeopeo Canal within Section 6. It is to be noted that the conveyance systems will be refined during the detailed design stage for any proposed residential development within the site.

### 5.2.1 Internal site conveyance

#### 5.2.1.1 Primary network

A primary network is proposed to service the site during the 10yr ARI (10% AEP) rainfall event, as per WDC Engineering Code of Practice. A typical layout of the primary system will follow the road layout and be located within the road reserve. This network can be either by surface conveyance, such as swales, or a piped system. The nature of the topography will usually dictate which system is most appropriate.

This site has been assessed as a piped system as there is a surface level difference from north to south however very little difference (in surface level) from west to east, thus making it difficult for surface swales to convey runoff to the point of discharge from the site. A typical piped network is shown in drawing 1368/SK01, Appendix A

The pipelines within the southern portion of the site may have surcharge due to the high tail water conditions within the Kopeopeo Canal. The piped network

design can accommodate this (effect) by increasing the pipe sizes, and/or providing onsite detention, to ensure the surcharge will not reach the manhole lid levels. The detail of piped network will be assessed at the design stage of developments within this plan change area.

#### 5.2.1.2 Secondary overland flow paths

Overland flow paths are proposed for the site to provide a conveyance system for runoff generated during large rainfall events, with the design standard being 100yr ARI (1% AEP) including the effects of climate change. The runoff will be conveyed in both the primary system, with allowance for blockage, and overland flow paths.

The flow paths will utilise the road reserve with design parameters to provide for access during flood events, such as minimum levels of flow within the carriageway. This will ensure that there is *"... safe and comfortable vehicle and pedestrian access across and along road reserves..."*[6] during these larger rainfall events.

The overland flow paths will discharge to the pond located within the 40m buffer strip adjacent to SH30. This will allow for storage and ponding in this area prior to discharge to the Kopeopeo Canal via gravity.

The overland flow path conveyance for the north eastern corner can be achieved with a shallow swale adjacent to the road reserve. A typical layout and cross section is shown in drawings 1368/SK01 and SK11 respectively.

[6] Section 4.1.7 Performance standards, Chapter 4, WDC Engineering Code of Practice, Issue 8 April 2008

# 6 Discharge to Kopeopeo Canal

The receiving environment for the discharge of stormwater runoff from this site is the Kopeopeo Canal. This canal is part of the overall Rangitaiki Drainage District and Whakatane Waimana Rivers Scheme, which provide flood control for the natural and built environment. Therefore the water levels within the canal affect the discharge regime from the development within the site.

The discharge from the site can be either via a gravity system or pumped to accommodate the flood levels during various rainfall events. A pumped system will need to be sized in accordance with the storage available and the peak flows during various rainfall events. This assessment does not go into detail of the pumped system as the ability to provide gravity discharge is achievable and ultimately the preferred option by the long term asset owner, being WDC.

The discharge to the canal is also governed by the effects of the magnitude of the flows. Therefore this assessment is based on 2 scenarios as follows:

- Existing flow rate scenario
    - Discharge at existing situation (rural zone) flow rates rates
    - Flow rates are attenuated prior to discharge
- Partial attenuation scenario
    - Provide some level of attenuation prior to discharge but not fully restricted to existing flow rates
    - Mitigate effects within Kopeopeo Canal by altering the canal conveyance hydraulics such as:
        - increasing pump capacity at pumpstation
        - altering start-stop levels within the pumpstation operation
        - assessing flood gate levels

Details of the assessment of both scenarios is presented in the following sub sections

## *6.1 Existing flow rate scenario*

The overall design criterion for this scenario is to provide sufficient storage on site to allow for attenuation of flows in order to discharge at pre-development rates to the canal.

The storage can be provided in a variety of locations such as:

- Treatment pond
- Within road reserves
- Above ground tanks

During the 10yr ARI rainfall event, which is the primary level of service (w.r.t. drainage), the storage will need to allow for the fully functioning conveyance system. Therefore the storage is limited to pond and above ground tanks, as the road reserves will need to provide for other social activities.

The storage required is based on the difference between the volume that is generated under the existing site conditions (being rural activity status) and that for the fully developed site together with the discharge levels at no greater than the maximum allowable as per WDC requirements.

The storage can be provided within a pond and the low lying areas of the site. For this assessment a pond will be constructed within the 40m strip adjacent to SH30, below RL1.5m. The remainder of the low lying area between RL1.5m and RL2.5m will provide additional storage under the natural contour

A rough order of runoff volume has been investigated for a 3 day rainfall period together with the effects of storage within a pond, during 10yr ARI rainfall event, and low lying areas for 100yr ARI event. This investigation includes the discharge to the canal through a new 900mm diameter discharge pipeline with 600mm orifice plate. The outcomes are presented in Table 5

**Table 5: Storage, Peak flows and top water levels on site**

| **Rainfall event** | **3 day rainfall depth(mm)** | **$Q_p$ ($m^3/s$)** | **Runoff volume ($m^3$)** | **Storage required ($m^3$)** | **Top water level (RLm)** |
|---|---|---|---|---|---|
| 10yr | 190.3 | 0.6 | 29006 | 17355 | 2.16 |
| 100yr CC | 379.4 | 0.62 | 58607 | 27955 | 2.36 |

During 10yr event the pond storage will be augmented within the low lying area of the 40m buffer strip adjacent to SH30, with a footprint of approximately 4ha. For

the larger rainfall event the storage will encroach further into the site with a footprint of approximately 7.5ha

The peak flows from the site will be no greater than those experienced under the current situation for 10yr event and 80% of the peak during 100yr event. The comparison of flows together with the level between the site and the canal top water during various rainfall events is presented in Table 6

**Table 6: Peak flows and top water levels – attenuated flow**

| Rainfall event (ARI) | $Q_p$ ($m^3/s$) | | Top water level (RLm) | |
|---|---|---|---|---|
| | existing | proposed | Site | Canal |
| 10yr | 0.6 | 0.6 | 2.16 | 1.63 |
| 100yr CC | 1.01 (0.81 @ 80%) | 0.62 | 2.36 | 1.81 |

## *6.2 Partial attenuation scenario*

The fully developed site will increase the peak flows and volume of runoff (during rainfall events) as there is an increase of impervious surface coverage. The design criterion for this scenario is that the flows will not be attenuated to pre-development rates however there will be some attenuation as the flow is discharged through the treatment facilities on site. The effect of the stormwater discharge flows can be mitigated by altering the hydraulic grade line to accommodate additional flow within the conveyance capacity of the canal. This can be achieved by the following:

- Re-configuration of the pumpstation, and/or
- Increasing the existing pumpstation capacity by additional pump/s

By increasing the capacity of the canal system the top water level within the canal can be maintained (as under the current conditions), therefore this assessment is based on the tail water conditions equalling that of the current top water levels.

As with the previous scenario the treatment pond will be utilised, thus provide for a certain amount of attenuation, prior to discharge to the canal. The pond will gravity discharge through 2 x 1200mm diameter pipelines with flap gates, to prevent back flow to the site. The outcomes of the assessment of this scenario is presented within Table 7

**Table 7: Peak flows and top water level**

| Rainfall event (ARI) | Peak flow ($m^3/s$) | Top water level (RLm) | |
|---|---|---|---|
| | | **Site** | **Canal** |
| 10yr | 3.19 | 1.79 | 1.63 |
| 100yr CC | 5.05 | 2.18 | 1.81 |

Notes:
1. Pond dimensions as per table 3, Section 5.1.1
2. Low lying areas within the site are taken as RL2.0m or below

During the 10yr event the top water level is within the pond, as the top of the pond is at RL2.0m. During the larger rainfall event of 100yr ARI (including climate change) the top water level will reach RL2.18m, resulting in the low lying areas of the site will be inundated up to 180mm (0.18m). The extent of inundation is shown in drawing number 1368/SK03. A schematic cross section through the treatment pond, site and canal showing top water levels is shown in drawing 1368/SK11, Appendix A.

A sensitivity analysis was undertaken as to the effect of additional pipes discharging to the canal on the top water levels within the site. The outcome of 3 x 1200mm diameter pipeline is that the top water level (within the site) would be RL2.10m during 100yr event.

This option is considered as the best practical option as there is less area affected within the site by the top water levels for the larger rainfall events. Therefore the potential risk to the built environment is less with this scenario than the existing flow rate scenario.

### 6.2.1 Kopeopeo Canal pump station upgrade options

To accommodate the flows entering the canal from the fully developed site the Kopeopeo Canal pumpstation may need to be upgraded. This option has been discussed with EBoP and WDC[7] and is preferred from an asset ownership and operational point of view.

At present the Kopeopeo Canal pumpstation capacity operates at between $5m^3/s$ and $6m^3/s$ during large rainfall events, as required. The catchment demand for this pump's flow rate will need to be confirmed[8] . However, irrespective of the current flow demand the hydraulic grade of the canal can be altered to accommodate the increase in flow (within the canal) due to the discharge from the site. Once confirmation of flow rates the pumpstation either has capacity to

[7] Discussions were held with Roger Waugh, EBoP, and Santha Agas, WDC, on 18th January 2012.
[8] At the time of writing the catchment flow rate demand at Kopeopeo Canal pump station was not available.

accommodate the increase of flow or additional (capacity) can be provided by providing an additional pump/s.

It is to be noted that the staging of the development within the site will determine when and if the canal's hydraulic grade will need to be altered or when and if additional pumps within the station are required.

# 7 Staging of the Development

This assessment has considered the whole of the site being developed, being 21.75ha, however with multiple ownership this could be undertaken in stages. A stormwater management system designed for a portion of the site would need to accommodate the fully developed site's needs however not be fully constructed for the full site development.

As an example of a first stage of development could be Lot 2 DPS 387805, 220 Shaw Road. The stormwater management system would include the following:

- Pond for treatment purposes
- Piped conveyance
- Outlet to the canal

The treatment pond would be located such that it can be increased in size or interconnected to another pond to accommodate further development stages. The piped conveyance system would consider upstream flows and the outlet to the canal would ensure that the future flows can be accommodated without detrimental effects to the receiving environment.

The initial stages (of the site's development) could be undertaken without the need to upgrade the Kopeopeo Canal pumpstation. This would need to be assessed and approved by EBoP and WDC prior to finalisation of detailed infrastructure design.

# 8 EBoP and WDC consent requirements

The EBoP Regional Plans and bylaws have been reviewed with respect to compliance together with WDC District Plan. Resource consents for both authorities will need to be applied for with respect to the stormwater discharge from a fully developed residential site.

## *8.1 EBoP consents*

The discharge of stormwater, to the Kopeopeo Canal, from a residential development within the site would be considered under EBoP Regional Water and Land Plan, 1 December 2008 (EBoP RWLP). The discharge does not comply with permitted activity rule 30, as follows,

“The rate of discharge shall not exceed 125 litres per second for 10 minute duration 10% AEP storm event (10 year return period storm)”

The 10year ARI peak rate of discharge is in the order of 3.19$m^3$/s for the total site which includes impervious coverage of 70%, (under fully developed residential scenario). Therefore it becomes a restricted discretionary activity under Rule 30A (as follows) and requires resource consent.

“the discharge of stormwater to surface water…where the rate of discharge is greater than 125 litres per second for a 10 minute duration 10% AEP storm event.....”:

The culverts for discharge to the Kopeopeo Canal will require approval as they will be within the stop banks of the canal. Approval will need to be sought from EBoP with the application detailing the long term effects to the canal stop banks as well as (effects the stop bank) during construction of the culverts & outlet protection.

## *8.2 WDC approvals*

The proposed development will need to meet the objectives and policies of the operative WDC District Plan, 15th October 2010. This includes the compliance to the Engineering Code of Practice during the design phase of the development.

The stormwater discharge will need to have a current resource consent prior to any engineering approval being granted for works within the site. The design of any infrastructure will need to meet the performance standards set out in the WDC Engineering Code of Practice.

# 9 Summary

To support the zone change from Rural 1 (Plains) to Residential for the 21.75ha site between Huna and Shaw Roads adjacent to SH30 a feasibility study has been undertaken. Various stormwater management options have been assessed as to the viability and best practical application for a residential development. The effect on the receiving environment has also been considered.

The stormwater management for a fully developed residential zone will need to include conveyance, treatment and discharge to the Kopeopeo East Canal. The internal conveyance can be either a traditional piped solution or surface channels, such as swales. The piped solution suits this site the best in the lower lying areas due to the lack of surface level difference in the west-east direction.

The treatment of stormwater runoff prior to discharge can be achieved by many devices. However for this site the utilisation of a pond within the 40m buffer zone from SH30 makes for visual enhancement and economical sense. The location of the pond in this area will provide for a visual amenity, as landscaping will be undertaken, and no development can take place adjacent to the public highway...

The discharge to the Kopeopeo Canal is the largest constraint on the site with respect to stormwater management. The preference of EBoP and WDC is to have a gravity discharge from the site and mitigate the effects on the canal. The upgrading of the Kopeopeo Canal, downstream of the site, can be undertaken if necessary to allow for the increase of discharge from this site. Utilising the treatment pond option would allow partial attenuation of flow and provide gravity discharge to the canal. However the pumpstation will need to be evaluated as to the best practical option with respect to upgrade, whether it is additional pump/s or alteration to the existing pump operating regime.

The site has multiple ownership, therefore the staging of a stormwater management system would need to be considered. The initial stages will need to consider the integration of the fully developed area with respect to conveyance, treatment and discharge to the canal.

EBoP consents will need to be applied for with respect to discharge to the Kopeopeo Canal and construction of discharge pipelines etc within the stop banks of the canal. These consents are required by WDC prior to any approvals are granted for engineering and associated works.

# 10 References and Bibliography


Coffey 2012

*Geotechnical Assessment report for State Highway 30 Shaw & Huna Roads, Whakatane, PAG* Enterprises Ltd, 27 November 2012

EBoP HHGL

*Draft Hydrological and Hydraulic Guidelines for the Bay of Plenty Region,* prepared by Environmental Hazards Group, Environment Bay of Plenty, May 2012.

EBoP RWLP

*Bay of Plenty Regional Water and Land Plan,* Environment Bay of Plenty, December 2008, (updated 2 March 2010).

EBOP SWGL

*Stormwater management guidelines for the Bay of Plenty Region, DRAFT,* Environment Bay of Plenty 15th June 2012.

Geosciences 2012

*Preliminary site investigations for properties located at 220 and 234 State Highway 30, Whakatane (contamination report),* Geosciences Ltd, 13th December 2012

SKM 2012

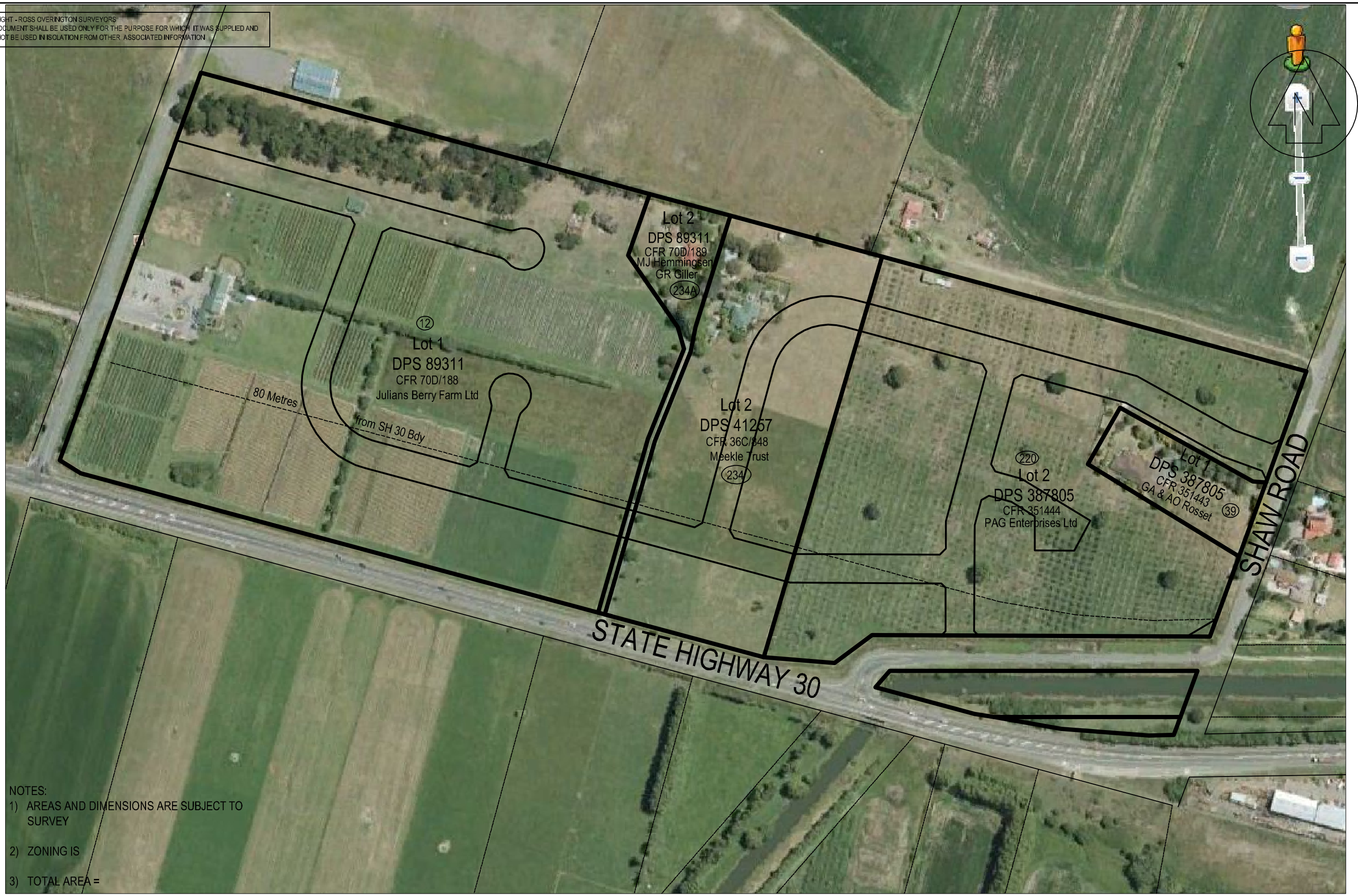
*Kopeopeo Canal – Assessment of Environmental Effects, Part A: Resource consent application, DRAFT, Revision 2,* Sinclair Knight Mertz, 15th June 2012.

WDC CoP

*Engineering Code of Practice, Issue 8,* Whakatane District Council, April 2008.

# APPENDIX A

# DRAWINGS




NOTES:
1) AREAS AND DIMENSIONS ARE SUBJECT TO SURVEY

2) ZONING IS

3) TOTAL AREA =

PREPARED FOR:
PAG ENTERPRISES LIMITED
PROPOSED RESIDENTIAL ZONE
SHAW ROAD
WHAKATANE

# CONCEPT PLAN
# POSSIBLE ROADING LAYOUT

WHAKATANE DISTRICT

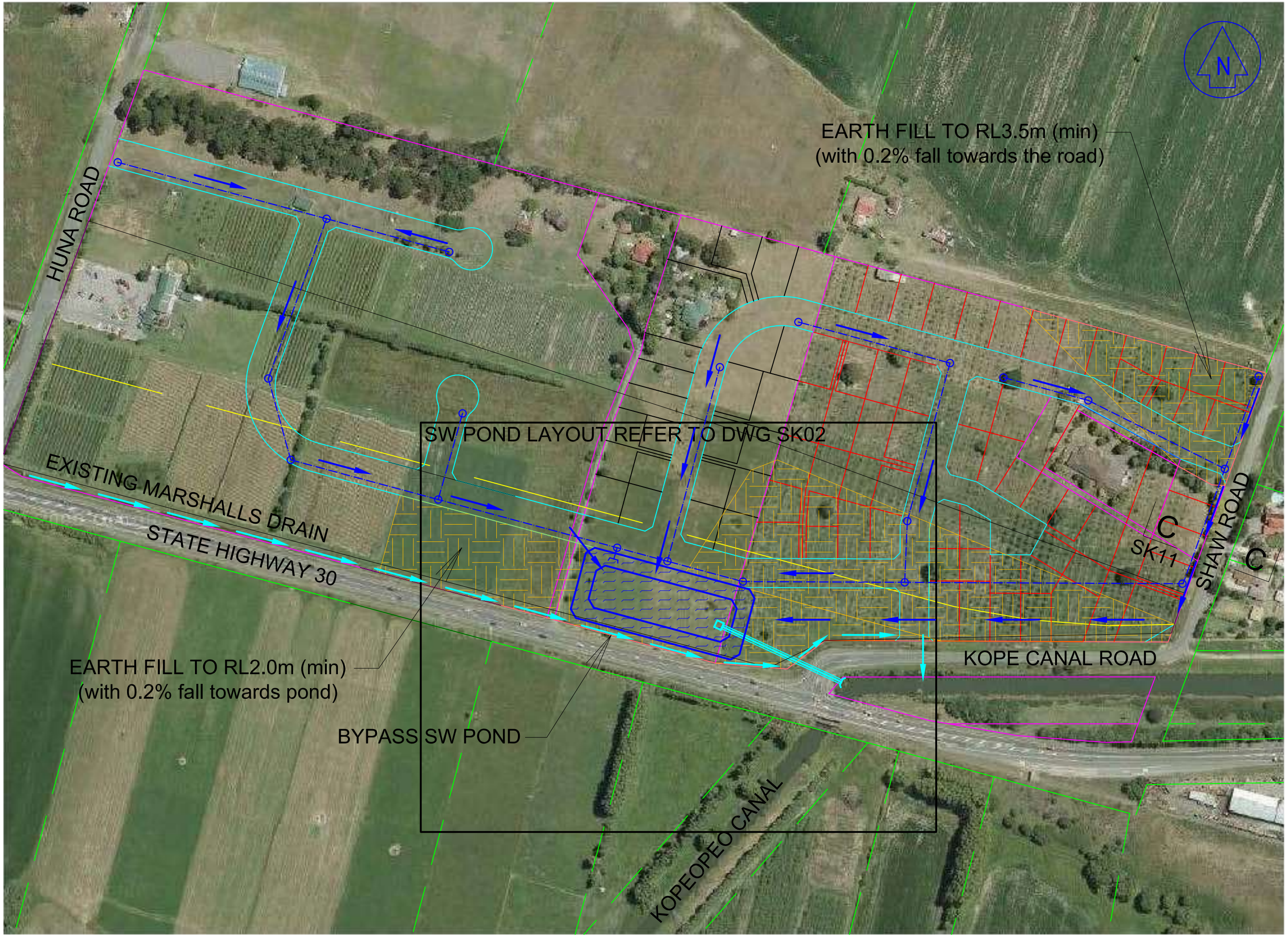
DATE: SEPTEMBER 2012
SCALES: 1:2500 (A3)

| No. | Date | Revision |
|---|---|---|
| | | |
| | | |
| | | |

Boulder Consultants (CNI) Limited, trading as
ROSS OVERINGTON SURVEYORS
PO Box 600 Phoenix House, Pyne Street, WHAKATANE
Phone: 07 308 6384 Email: whakatane@boulderconi.co.nz

DESIGNED | SURVEYED
DRAWN

DRAWING No. 2910/2
CAD No.

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3. REFER TO ROSS OVERINGTON SURVEYORS CONCEPT PLAN

**LEGEND:**

- EARTH FILL
- STORMWATER POND
- OVERLAND FLOW PATH
- PRIMARY PIPE NETWORK
- EXISTING MARSHALLS DRAIN

| REV | DATE: | AMENDMENT: |
|---|---|---|
| | | |
| | | |
| | | |
| | | |
| | | |

CLIENT:
PAG Enterprises Ltd

Stormwater SOLUTIONS
Stormwater Solutions Consulting Ltd
PO Box 25 598, St Heliers, Auckland 1740
www.stormwatersolutions.co.nz

FOR PLAN CHANGE APPLICATION ONLY

PROJECT:
PROPOSED RESIDENTIAL ZONE
SHAW ROAD
WHAKATANE

TITLE:
STORMWATER MANAGEMENT LAYOUT PLAN (PRELIMINARY)

| DATE: | SCALE: |
|---|---|
| JAN 2013 | 1:2500 @A3 |

| DRAWING NO: | REVISION |
|---|---|
| 1368/SK01 | |

| DRAWN: | CHECKED: |
|---|---|
| W.L. | B.R. |


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**LEGEND:**

EARTH FILL

STORMWATER POND

| REV | DATE: | AMENDMENT: |
|---|---|---|
| | | |
| | | |
| | | |
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| | | |

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PROJECT:
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SHAW ROAD
WHAKATANE

TITLE:
SW POND LAYOUT PLAN (PRELIMINARY)

| DATE: | SCALE: |
|---|---|
| JAN 2013 | 1:1000 @A3 |

| DRAWING NO: | REVISION |
|---|---|
| 1368/SK02 | |

| DRAWN: | CHECKED: |
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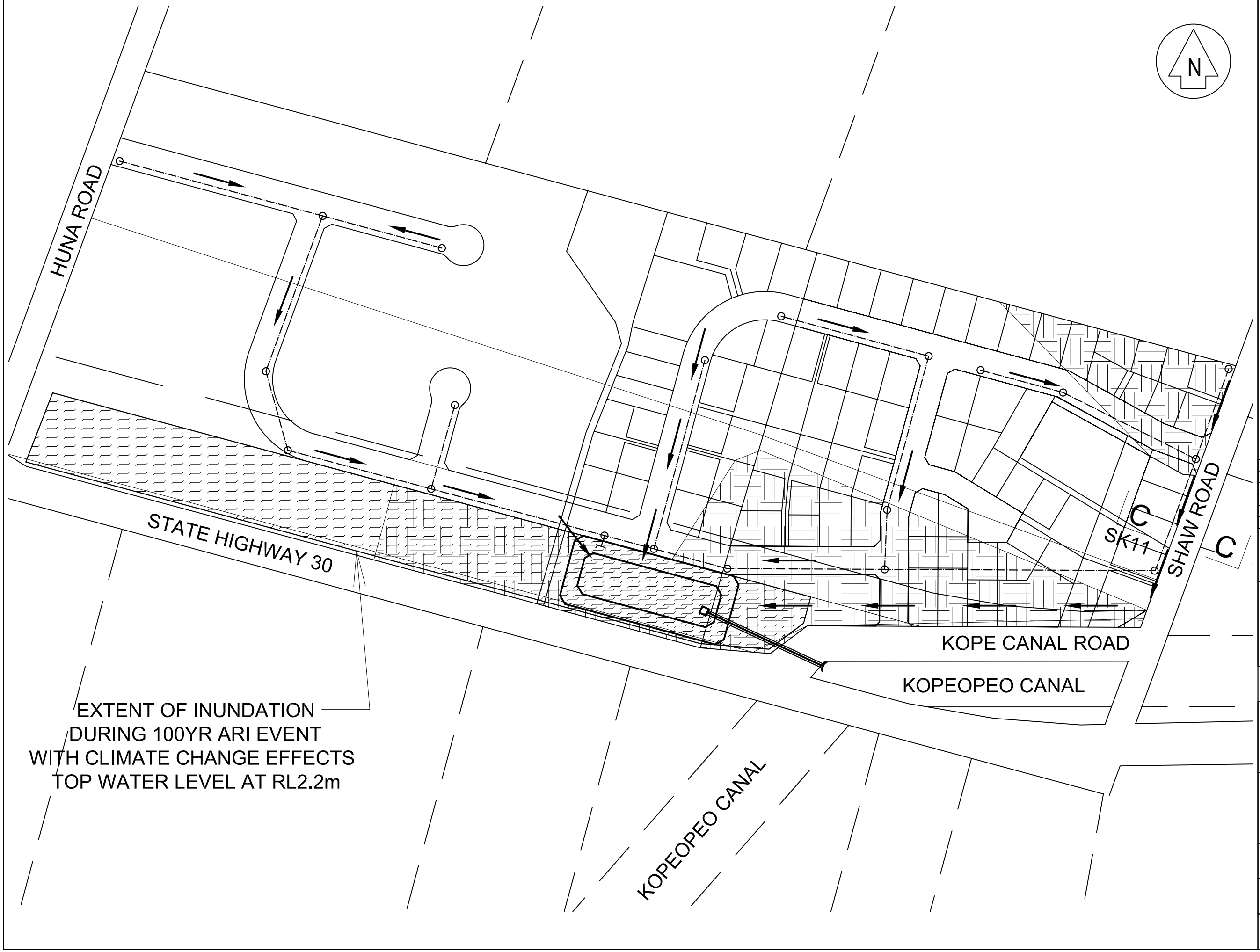
| W.L. | B.R. |

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**LEGEND:**

- EARTH FILL
- EXTENT OF INUNDATION
- OVERLAND FLOW PATH
- PRIMARY PIPE NETWORK

| REV | DATE: | AMENDMENT: |
|---|---|---|
| | | |
| | | |
| | | |
| | | |
| | | |

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PAG Enterprises Ltd

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FOR PLAN CHANGE APPLICATION ONLY

PROJECT:
PROPOSED RESIDENTIAL ZONE
SHAW ROAD
WHAKATANE

TITLE:
Extent of Inundation
100yr ARI Climate Change
(Partial Attenuation Scenario)

| DATE: | SCALE: |
|---|---|
| JAN 2013 | 1:2500 @A3 |

| DRAWING NO: | REVISION |
|---|---|
| 1368/SK03 | |

| DRAWN: | CHECKED: |
|---|---|
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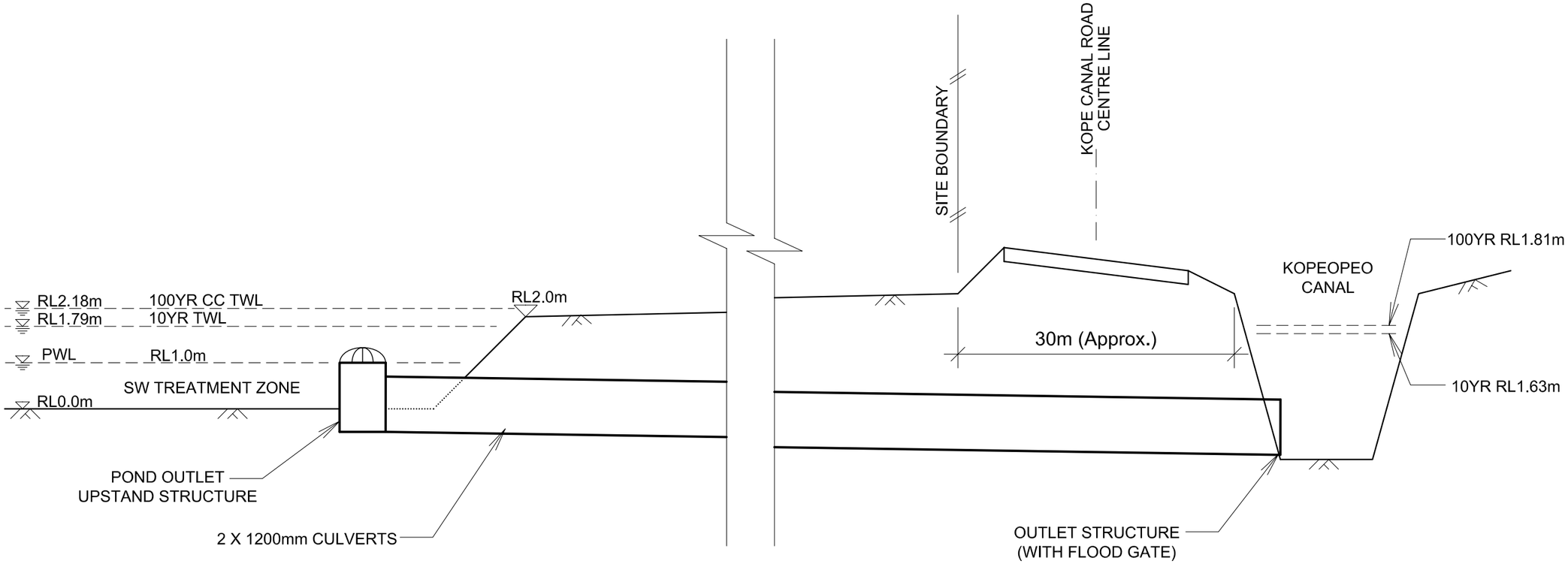
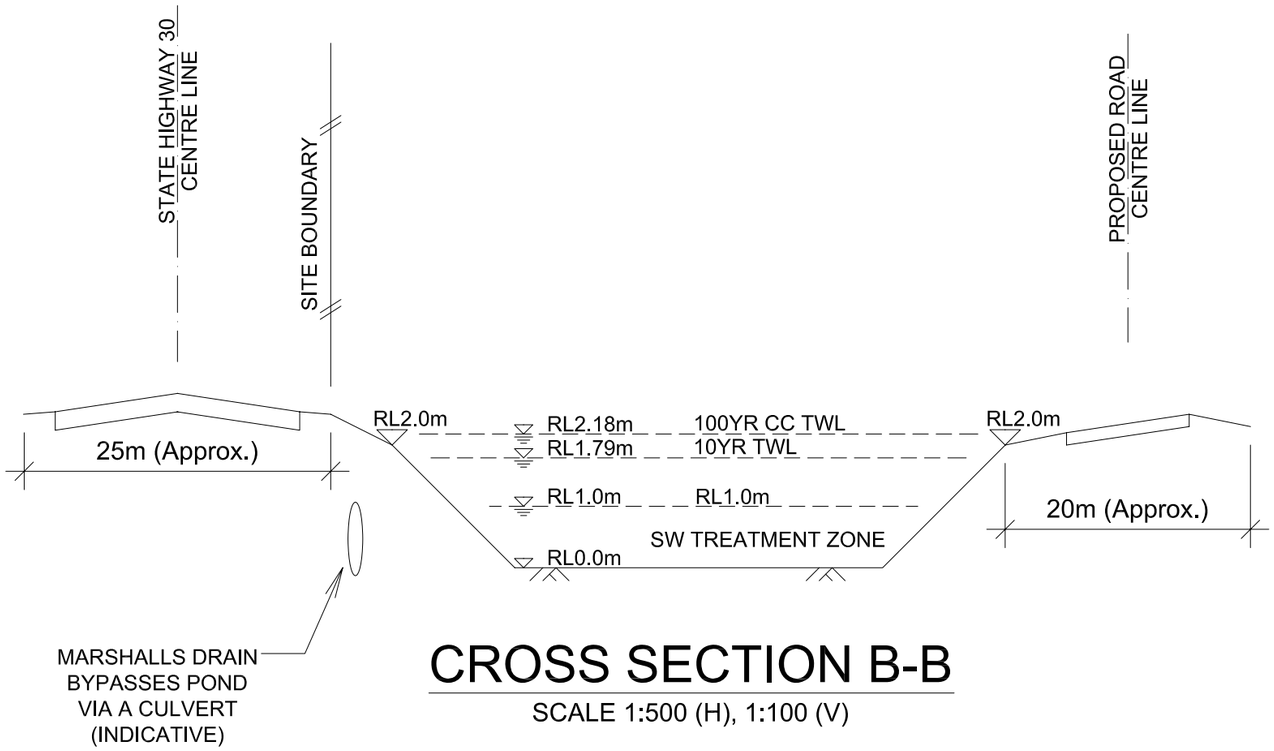



# CROSS SECTION A-A

SCALE 1:500 (H), 1:100 (V)




# CROSS SECTION B-B

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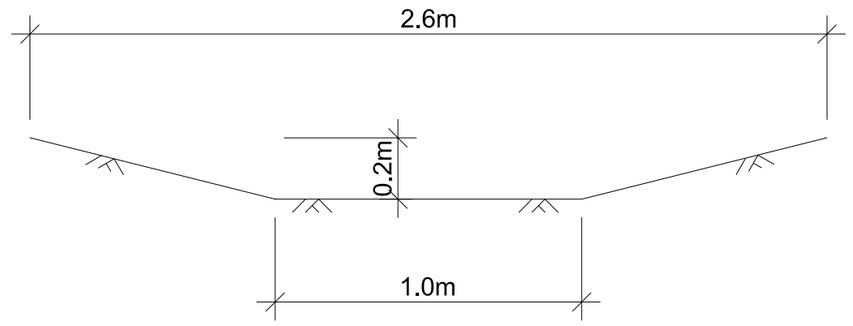


# CROSS SECTION C-C

SCALE 1:20

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3. REFER TO ROSS OVERINGTON SURVEYORS CONCEPT PLAN

**LEGEND:**

| | |
|---|---|
| TWL | TOP WATER LEVEL |
| PWL | PERMANENT WATER LEVEL |
| SW | STORMWATER |
| CC | CLIMATE CHANGE |

| REV | DATE: | AMENDMENT: |
|---|---|---|
| | | |
| | | |
| | | |
| | | |
| | | |

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SHAW ROAD
WHAKATANE

TITLE:
CROSS SECTIONS (PRELIMINARY)

| DATE: | SCALE: |
|---|---|
| JAN 2013 | AS SHOWN |

| DRAWING NO: | REVISION |
|---|---|
| 1368/SK11 | |

| DRAWN: | CHECKED: |
|---|---|
| W.L. | B.R. |

# APPENDIX B

# CALCULATIONS

- Peak flow – existing & proposed
- Stormwater Treatment options
  - At Source
  - End of pipe
- Internal conveyance assessment
- Site specific temporal rainfall pattern and unit hydrograph
- MOUSE modelling
  - Discharge from fully developed site
  - Effects on Kopeopeo canal

Jlcd bf

17/01/2013 Shaw Rd. Job No. 1368 1/2

Aim: Peak flows and volumes.

1 Ref: SW Management Guidelines
Storm duration used for peak flow control is the 1 hr storm.

2 Ref: NIWA HIRDS V3 (current rainfall)

| ARI | 1 hr | 24 hr | 72 hr |
|---|---|---|---|
| 2yr | 25.3 | 90.6 | 123.9 |
| 10yr | 38.9 | 139.1 | 190.3 |
| 100yr | 66.6 | 237.5 | 326.8 |

2.1°C temperature increase (climate change)

| ARI | 1 hr | 24 hr | 72 hr |
|---|---|---|---|
| 2yr | 28.9 | 98.8 | 133 |
| 10yr | 44.9 | 157.5 | 213.9 |
| 100yr | 77.8 | 277.4 | 379.4 |

3. Peak discharge.

3.1. Existing.

Catchment area = 21.75 ha

Impervious percentage = 100% pervious

Runoff coefficient. 0.2 Medium soakage soil types. Cultivated.

0.3. Medium soakage soil type. pasture and grass cover

⇒ Average : 0.25 ✓

⇒ $Q_{p2} = 21.75 \times 10000 \times 0.25 \times 25.3/3600 = 382$ L/S ✓

$Q_{p10} = 21.75 \times 10000 \times 0.25 \times 38.9/3600 = 588$ L/S ✓

$Q_{p100} = 21.75 \times 10000 \times 0.25 \times 66.6/3600 = 1006$ L/S ⇒ 80% $Q_{p100} = 805$ L/S

3.2. Proposed.

Assume 70% impervious cover. C = 0.9 for imp. area. ✓

⇒ Current climate. $Q_{p2} = (21.75 \times 10000 \times 70\% \times 0.9 + 21.75 \times 10000 \times 30\% \times 0.3) \times 25.3/3600 = 1101$ L/S

$Q_{p10} = 1692$ L/S ✓

$Q_{p100} = 2897$ L/S ✓

Climate change $Q_{p2} = 1257$ L/S

$Q_{p10} = 1933$ L/S

$Q_{p100} = 3384$ L/S

4. Volumes.

4.1. Water quality volume.

(Ref: SW Management guidelines.)

$A = 0.9 \times 70\% \times 21.75\,ha + 0.15 \times 30\% \times 21.75\,ha = 14.7\,ha$

90 percentile storm = 2 yr 1hr rainfall depth = $25.3\,mm$

$\Rightarrow$ WQV. $V = 25.3\,mm \times 14.7\,ha = 3719\,m^3$ ✓

4.2 2/10/100 runoff volume. (3 day storm)

use runoff coefficient of 0.3 for pervious area

$\Rightarrow$ Equivalent $A = (0.9 \times 70\% + 0.3 \times 30\%) \times 21.75\,ha = 15.66\,ha$

$\Rightarrow$ Current rainfall events.

2yr $V_2 = 15.66 \times 123.9 = 19400\,m^3$ ✓

10yr $V_{10} = 15.66 \times 190.3 = 29800\,m^3$

100yr. $V_{100} = 15.66 \times 324.8 = 50864\,m^3$

With climate change effects.

2yr $V_{2c} = 15.66 \times 133 = 20830\,m^3$

10yr $V_{10c} = 15.66 \times 213.9 = 33500\,m^3$

100yr $V_{100c} = 15.66 \times 379.4 = 59414\,m^3$

4.3. Current rural standard for runoff discharge at a rate of 28 mm/day over 3 day period.

$\Rightarrow$ Total runoff discharge $= 21.75\,ha \times 28 \times 3 = 18270\,m^3$ ✓

$\Rightarrow$ Onsite additional storage required for the 100yr event

Current rainfall: $V = 50864 - 18270 = 32594\,m^3$

Climate change: $V = 59414 - 18270 = 41144\,m^3$

calcd ft.

22/01/2013 Shaw Rd. Job No. 1368. 1/5

Aim: Stormwater treatment options (at source)

1. At source treatment . to provide SW treatment for road reserve only.
   - Swales
   - Raingardens
   - Proprietary units ✓

1.1. Swales. (per 100m length 20m width road reserve)

Catchment area = 2000 m²

Assume 80% impervious cover

⇒ Imp. = 2000 × 80% = 1600 m²

Per = 2000 × 20% = 400 m²

Runoff coefficient (Ref. SW Management Guideline, Table 7.1)

Imp. = 0.85 (paved surface)

Per. = 0.30 (grass cover)

WQ rainfall intensity = 25.3 mm (90 percentile storm 2yr 1hr)

⇒ $Q_{WQ} = (1600 \times 0.85 + 400 \times 0.3) \times 25.3 / 3600 = 10.4$ L/s ✓

10yr 1hr rainfall intensity = 38.9 mm

⇒ $Q_{10} = (1600 \times 0.85 + 400 \times 0.3) \times 38.9 / 3600 = 16.0$ L/s ✓

Swale sizing.

Assume: longitudinal slope = 1%

Side slope = 4H:1V

depth of flow = 100 mm

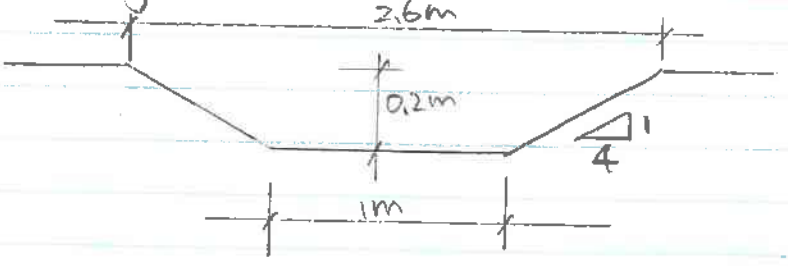
Manning's coefficient: WQ storm = 0.25 ✓

10yr storm = 0.03 ✓

⇒ Sheet 2. Swale base width = 1m ✓

Swale length = 10m ✓

Typical x-section. (See below and also Sheet 3)




depth of flow 100mm. for water quality.

Stormwater Solutions Consulting Limited
PO Box25 598, St Heliers
Ph: 09 974 2170
Fax: 09 929 3050

# Swale design spreadsheet - *as per SW Management Guidelines for Bay of Plenty region*

| | | | |
|---|---|---|---|
| Job name | Shaw Rd | File Name | Treatment options |
| Job No. | 1368 | Sheet Name | Std Swale |
| Date | 22/01/2013 | Path | S:\SS\Ongoing Projects\1368 |

**Swale sizing**

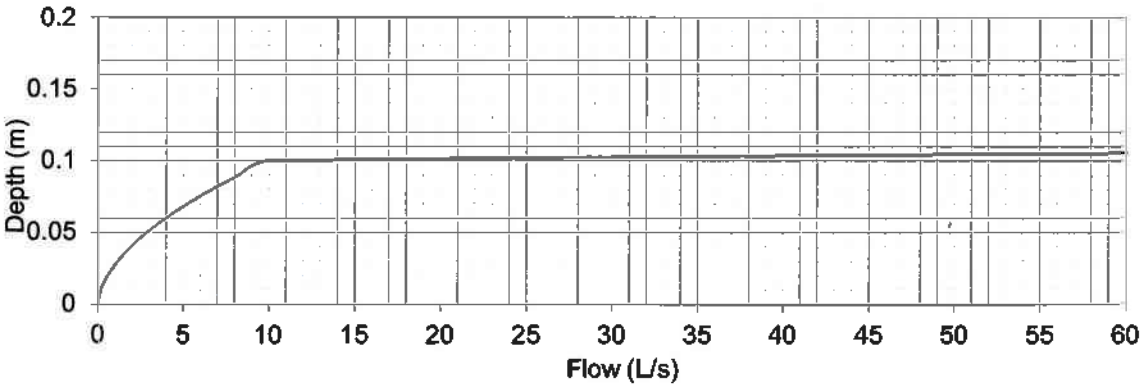
| | | | |
|---|---|---|---|
| Z-horiz slope | 4 | swale depth | 0.2 m |
| base width | 1 m | slope m/m | 0.010 |
| top width | 2.6 m | res time | 9 min |

grass height 100 mm

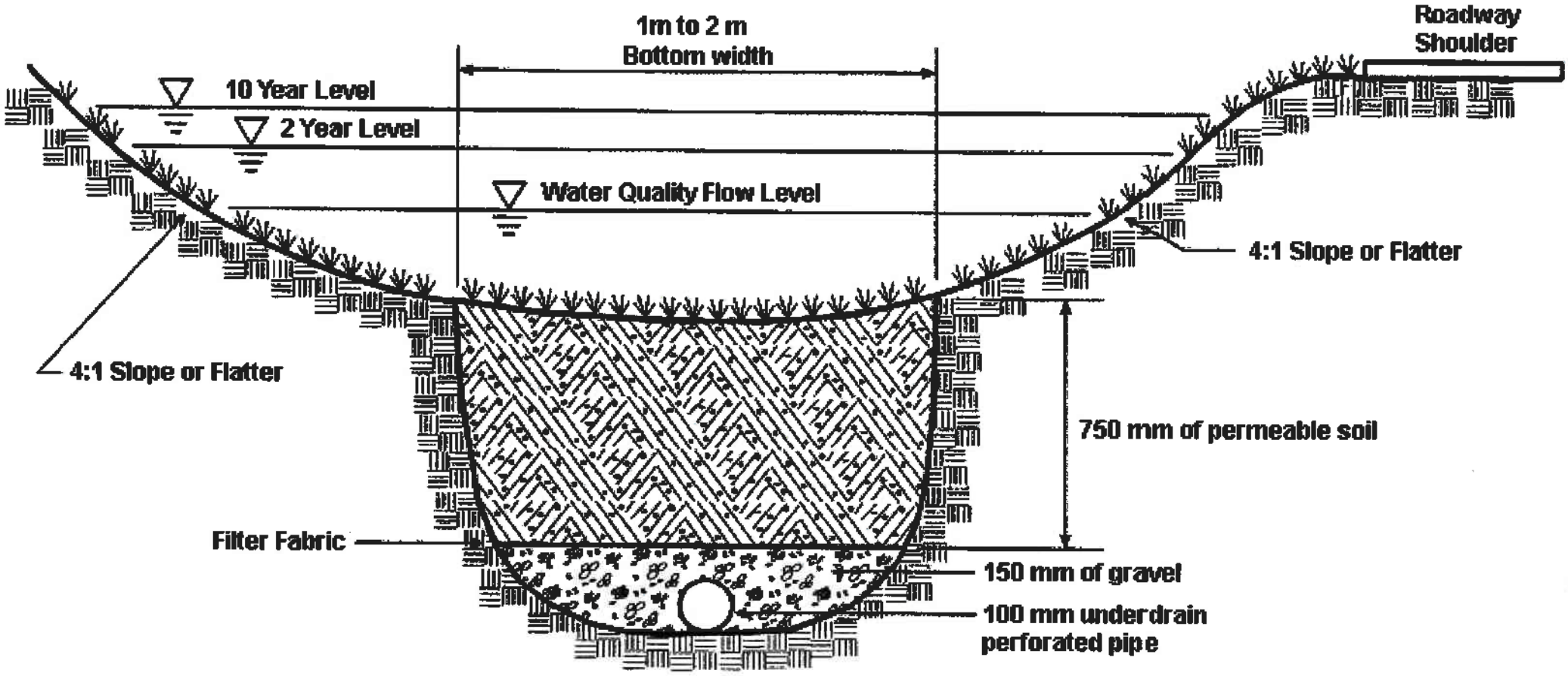
0.25 mannings coefficient for WQ storm
0.03 for submerged flow 10yr storm

| depth | n | A | R | V | Q (L/s) | Length | |
|---|---|---|---|---|---|---|---|
| 0 | 0.25 | 0.00 | 0.00 | 0.00 | 0.0 | 0.0 | |
| 0.01 | 0.25 | 0.01 | 0.01 | 0.02 | 0.2 | 9.6 | |
| 0.02 | 0.25 | 0.02 | 0.02 | 0.03 | 0.6 | 14.9 | |
| 0.03 | 0.25 | 0.03 | 0.03 | 0.04 | 1.2 | 19.2 | |
| 0.04 | 0.25 | 0.05 | 0.03 | 0.04 | 2.0 | 22.8 | |
| 0.05 | 0.25 | 0.06 | 0.04 | 0.05 | 2.9 | 26.0 | |
| 0.06 | 0.25 | 0.07 | 0.05 | 0.05 | 4.0 | 28.9 | |
| 0.07 | 0.25 | 0.09 | 0.06 | 0.06 | 5.2 | 31.6 | |
| 0.075 | 0.25 | 0.10 | 0.06 | 0.06 | 5.9 | 32.9 | |
| 0.08 | 0.25 | 0.11 | 0.06 | 0.06 | 6.7 | 34.1 | |
| 0.09 | 0.25 | 0.12 | 0.07 | 0.07 | 8.3 | 36.5 | |
| 0.1 | 0.25 | 0.14 | 0.08 | 0.07 | 10.0 | 38.7 | **Treatment** |
| 0.11 | 0.03 | 0.16 | 0.08 | 0.63 | 99.7 | | |
| 0.12 | 0.03 | 0.18 | 0.09 | 0.66 | 117.3 | | |
| 0.13 | 0.03 | 0.20 | 0.10 | 0.69 | 136.4 | | |
| 0.14 | 0.03 | 0.22 | 0.10 | 0.72 | 157.1 | | |
| 0.15 | 0.03 | 0.24 | 0.11 | 0.75 | 179.3 | | |
| 0.16 | 0.03 | 0.26 | 0.11 | 0.77 | 203.1 | | |
| 0.17 | 0.03 | 0.29 | 0.12 | 0.80 | 228.6 | | |
| 0.18 | 0.03 | 0.31 | 0.12 | 0.83 | 255.7 | | |
| 0.19 | 0.03 | 0.33 | 0.13 | 0.85 | 284.5 | | |
| 0.2 | 0.03 | 0.36 | 0.14 | 0.88 | 315.1 | | |

**Swale depth vs. flow**



# Figure 9.3 – Swale schematic showing soils and underdrain



1.2. Raingarden

Assume the same catchment area as swale.

WQV = (1600 × 0.85 + 4500 × 0.3) × 25.3 / 1000 = 37.4 m³ ✓

Raingarden surface area required = WQV · drg / K(h + drg) + trg.

Where drg = planting soil depth = 0.8 m

K = coefficient of permeability = 0.75 m/d

h = Average water height = 0.15 m

trg = time of pass WQV = 1.5 days

⇒ A = 37.4 × 0.85 / (0.75 × (0.8 + 0.15) × 1.5) = 28.3 m² ✓

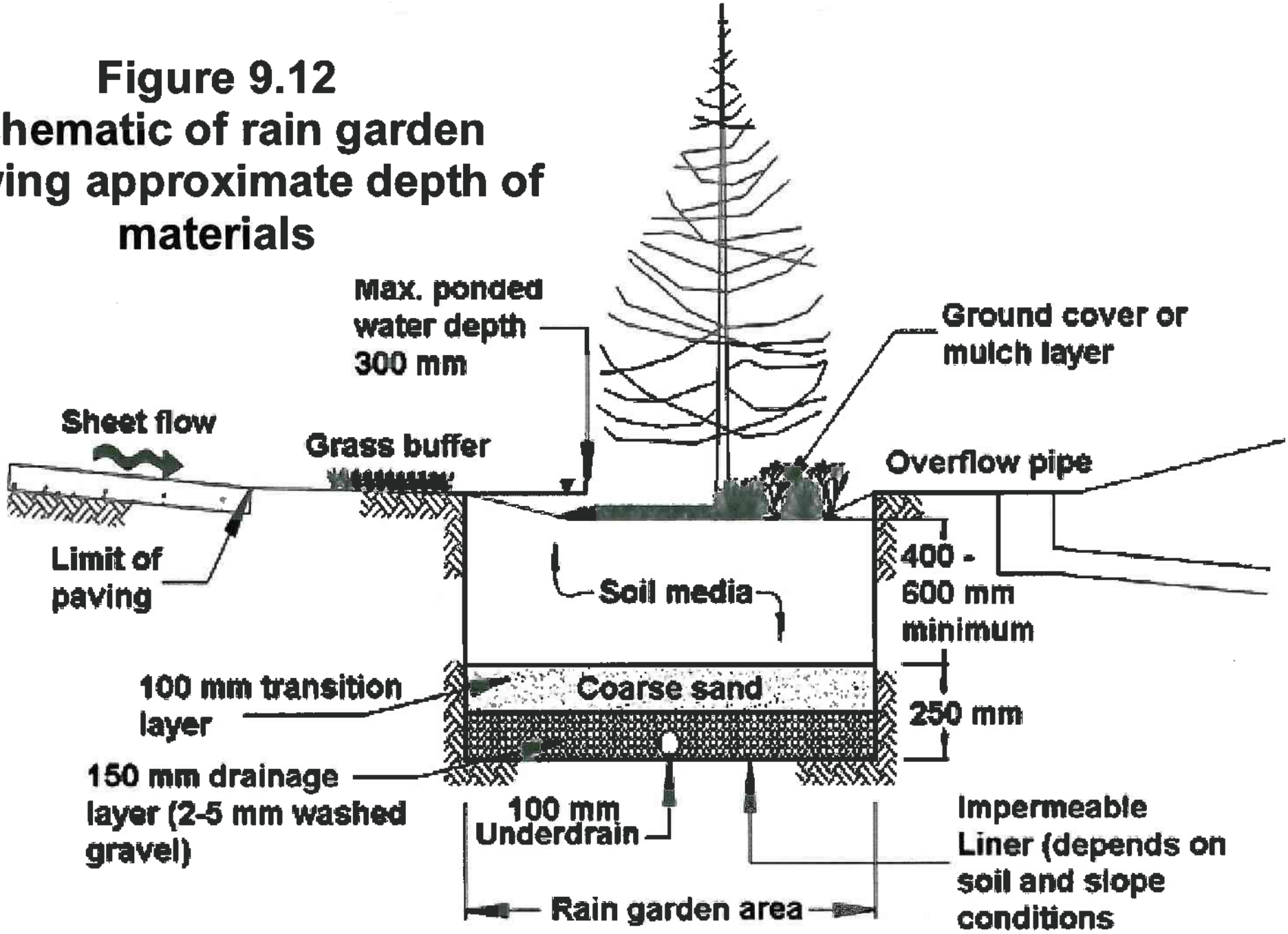
⇒ Up to 30 m² raingarden is required to provide treatment for a typical road reserve area of 2000 m²

Typical raingarden x-section is shown in sheet 5.

1.3. Proprietary Units

Runoff from the road reserve can also be treated by proprietary units such as stormfilters supplied by Stormwater 360.

# Figure 9.12
# Schematic of rain garden showing approximate depth of materials



Draft.

22/01/2013 · Shaw Rd Job No. 1368 1/2

Aim: Stormwater treatment options (end of pipe)

1. Water Quality Volume for the entire site

Site area = 21.75 ha. assume 70% impervious cover

Ref: SW Management Guidelines.

$A_{WQ} = (0.9 \times 70\% + 0.15 \times 30\%) \times 21.75 \times 10000 = 146812.5\ m^2$

$WQV = 146812.5 \times 25.3/1000 = 3714\ m^3$ ✓

2. Treatment via a SW pond.

⇒ Pond footprint = 120m x 50m = 6000 $m^2$

Permanent water level = RL 1.0m

Pond base = RL 0.0m

Permanent storage provided in the pond = 3638 $m^3$ close to WQV ⇒ OK

# *Onsite Pond Sizing*

| | | | |
|---|---|---|---|
| Job name | Shaw Rd | File name | Pond Option |
| Job number | 1368 | Path | S:\SS\Ongoing Projects\1368 |
| Date | 22/01/2013 | Sheet name | Treatment Pond |

Onsite Pond

| | | |
|---|---|---|
| Surface Level, RL | 2 m | |
| Permanent Water Level | 1 m | (assumed) |
| Top Length | 120 m | |
| Top Width | 50 m | |
| Side slopes | 5 H to 1 V | |

| Pond Level | Pond Depth | Length, m | Width, m | Area, m2 | Volume, m3 | Total Volume, m3 |
|---|---|---|---|---|---|---|
| 2 | 2 | 120 | 50 | 6000 | 2794 | 8875 |
| 1.5 | 1.5 | 115 | 45 | 5175 | 2394 | 6081 |
| 1 | 1 | 110 | 40 | 4400 | 2019 | 3688 |
| 0.5 | 0.5 | 105 | 35 | 3675 | 1669 | 1669 |
| 0 | 0 | 100 | 30 | 3000 | 0 | 0 |

| | | | |
|---|---|---|---|
| Permanent Storage | 3688 | $m^3$ | (From RL0 to RL1.0m) |
| Live Storage | 5188 | $m^3$ | (From RL1.0 to RL2.0m) |

# MOUSE Runoff Computation Engine v2008 Release Version (10.0.0.1330)

## MOUSE Runoff Model A Status Report

### File Overview

| | |
|---|---|
| Working dir : | Whakatane\Modelling\MOUSE\Shaw Run05\ |
| Hydrological data file : | Whakatane.HGF |
| Sewer network data : | Whakatane.UND |
| Result File : | TD Qp10 24hr.CRF |

### Time Overview

| | |
|---|---|
| Simulation start date : | 1/01/2013 0:00 |
| Simulation end date : | 2/01/2013 0:00 |
| Simulation time step [s] : | 10 |

### Dry Weather Periods

| | |
|---|---|
| Initial loss recovery rate [m/hour] : | 0.00005 |

## Simulation Result Summary

## Catchment Result Summary

### Catchment runoff hydrograph summary

| | Rain Event | Minimum | Maximum | Flow - Accumulated | Time - Minimum | Time - Maximum |
|---|---|---|---|---|---|---|
| | | [m3/s] | [m3/s] | m3 | | |
| Shaw Rd IMP | WDC Rain / TD 10yr 24hr | 0 | 3.631 | 18344.571 | 1/01/2013 0:00 | 1/01/2013 19:14 |
| Shaw Rd PER | WDC Rain / TD 10yr 24hr | 0 | 0.418 | 2618.363 | 1/01/2013 0:00 | 1/01/2013 19:14 |
| Total : | | | 4.049 | 20962.934 | | |

# MOUSE Runoff Computation Engine v2008 Release Version (10.0.0.1330)

## MOUSE Runoff Model A Status Report

### File Overview

| | |
|---|---|
| Working dir : | Whakatane\Modelling\MOUSE\Shaw Run05\ |
| Hydrological data file : | Whakatane.HGF |
| Sewer network data : | Whakatane.UND |
| Result File : | TD Qp100 24hr.CRF |

### Time Overview

| | |
|---|---|
| Simulation start date : | 1/01/2013 0:00 |
| Simulation end date : | 2/01/2013 0:00 |
| Simulation time step [s] : | 10 |

### Dry Weather Periods

| | |
|---|---|
| Initial loss recovery rate [m/hour] : | 0.00005 |

## Simulation Result Summary

## Catchment Result Summary

### Catchment runoff hydrograph summary

| | Rain Event | Minimum | Maximum | Flow - Accumulated | Time - Minimum | Time - Maximum |
|---|---|---|---|---|---|---|
| | | [m3/s] | [m3/s] | m3 | | |
| Shaw Rd IMP | WDC_Rain / TD 100yr24hr | 0 | 7.262 | 37264.737 | 1/01/2013 0:00 | 1/01/2013 19:14 |
| Shaw Rd PER | WDC_Rain / TD 100yr24hr | 0 | 0.835 | 5318.961 | 1/01/2013 0:00 | 1/01/2013 19:14 |
| Total : | | | 8.097 | 42583.698 | | |

Chtd 11.

25/01/2013 Shaw Rd. Job№ 1368. 1/1

Aim: Internal conveyance for norther eastern portion runoff discharge

1 The majority of the site can discharge to the proposed SW treatment pond near SH30, except the north eastern corner of the site where the existing ground level is at RL 1.5m

2. Conveyance options

2.1. discharge to SW pond (with surcharge)

Pond invle RL = 0 m. (approx)

Length of pipe = 500m (approx)

If the pipe slope = 0.5% ⇒ level drop = 500 × 0.5% = 2.5m

Plus invtor pipe size and cover

⇒ Fill the low lying area to RL 3.5m (2m fill)

22 discharge to the Kope canal via pond outlet pipe

This option will require onsite storage to accommodate tail water effect from Kope Canal

23. discharge to SW pond via swale

A shallow gradient swale to convey runoff to SW pond

Pond top RL = 2.0m

swale gradient = 0.2%

⇒ 500 × 0.2% = 1.0m

⇒ Swale base at northeastern corner = 3.0 m

⇒ Fill the low lying area to RL 3.5m (approx.)

31/01/2013. Shaw Rd. Job No 1368. 1/1

Aim: MOUSE modelling (Run 6)
- Modified from Run 05
- Increase pond outlet weir length to 10m

1. Modelled results.

| | 10yr current rainfall | | 100yr climate change | |
|---|---|---|---|---|
| | 24hr | 72hr | 24hr | 72hr |
| Scenario | 1 | 2 | 3 | 4 |
| Tailwater level | 1.63 | 1.63 | 1.81 | 1.81 |
| Pond outlet structure | | | | |
| weir length. | 10m | 10m | 10m | 10m |
| outlet pipe | 1200 x 2 | 1200 x 2 | 1200 x 2 | 1200 x 2 |
| Onsite pond top water level | 1.79m | 1.79m | 2.18m | 2.18m |
| Weir discharge | 3.19m³/s | 3.19m³/s | 5.05m³/s | 5.05m³/s |

Scenario 5: Modified from Scenario 3.
Add another 1200Φ pipe ⇒ 1200 x 3. Add weir length to 15m
⇒ Onsite pond-top water level = 2.10m
Weir discharge = 6.62 m³/s

Note: Weir added to MOUSE model to represent upstand structure

1/2/2013. Shaw Rd Job No. 1368. Checked. 1/2

Aim: MOUSE modelling (Run08)
- Modified from Run07
- Onsite detention and discharge at existing rate

1. Ref: calcs dated 17/01/2013. aimed peak flows and volumes
   ⇒ Allowable peak discharge rates post development
   $Q_{p10}$ = 588 L/s
   $Q_{p100}$ = 805 L/s - 80% of 100yr ARI event.

2. Pond geometry.
   To provide more detention volume. onsite pond will be achieved to as sheet 2.

3. Pond outlet structure
   A 900mmΦ pipe with a 600mmΦ orifice plate at inlet

4. Modelled results.

| | 10yr 24hr | 10yr 72hr | 100yr CC 24hr | 100yr CC 72hr |
|---|---|---|---|---|
| Onsite catchment | | | | |
| Qp imp. | 3.63 m³/s | 3.63 m³/s | 7.26 m³/s | 7.26 m³/s |
| Qp per | 0.42 m³/s | 0.42 m³/s | 0.84 m³/s | 0.84 m³/s |
| Runoff volume | 20963 m³ | 29006 m³ | 42584 m³ | 58607 m³ |
| Onsite pond | | | | |
| top water level | 2.16m | 2.16m | 2.36m | 2.36m |
| peak discharge | 0.60 m³/s | 0.60 m³/s | 0.62 m³/s | 0.62 m³/s |
| Onsite storage provided | 17355 m³ | 17355 m³ | 27955 m³ | 27955 m³ |
| Extent of ponding (approx.) | 4 ha | 4 ha | 7.5 ha | 7.5 ha |

# *Onsite Pond Sizing*

| | | | |
|---|---|---|---|
| Job name | Shaw Rd | File name | Pond Option |
| Job number | 1368 | Path | S:\SS\Ongoing Projects\1368 |
| Date | 1/02/2013 | Sheet name | Detention Treatment Pond |

Onsite Pond

| | | |
|---|---|---|
| Surface Level, RL | 2 m | |
| Permanent Water Level | 1 m | (assumed) |
| Side slopes | 5 H to 1 V | (below RL1.5m) |

| Pond Level | Pond Depth | Length, m | Width, m | Area, m2 | Volume, m3 | Total Volume, m3 |
|---|---|---|---|---|---|---|
| 2.5 | 2.5 | | | 100000 | 30000 | 42375 |
| 2 | 2 | | | 20000 | 6294 | 12375 |
| 1.5 | 1.5 | 115 | 45 | 5175 | 2394 | 6081 |
| 1 | 1 | 110 | 40 | 4400 | 2019 | 3688 |
| 0.5 | 0.5 | 105 | 35 | 3675 | 1669 | 1669 |
| 0 | 0 | 100 | 30 | 3000 | 0 | 0 |

| | | | |
|---|---|---|---|
| Permanent Storage | 3688 | $m^3$ | (From RL0 to RL1.0m) |
| Live Storage | 8688 | $m^3$ | (From RL1.0 to RL2.0m) |
| 100yr | 30000 | $m^3$ | Above RL2.0 |

Reflects the lay'g surface geometry.

checked.

30/01/2013. Shaw Rd Job No. 1368. 1/5

Aim: MOUSE modelling (Run 05)
- Modified from Run 02
- HIRDS rainfall temporal distribution
- Extended pond top RL to 2.5m (using 40m set back = 200m)

1. Rainfall pattern
Use 24hr and 72hr rainfall depth as per NIWA HIRDS
Use rainfall temporal distribution pattern as per EoPRC instruction (sheet 2)
⇒ Sheets 3-4. Unit hydrographs ✓

2. Tailwater within Kope Canal
10yr = 1.63m
100yr = 1.81 m

3. Pond size (Sheet 5).

4. Modelled results

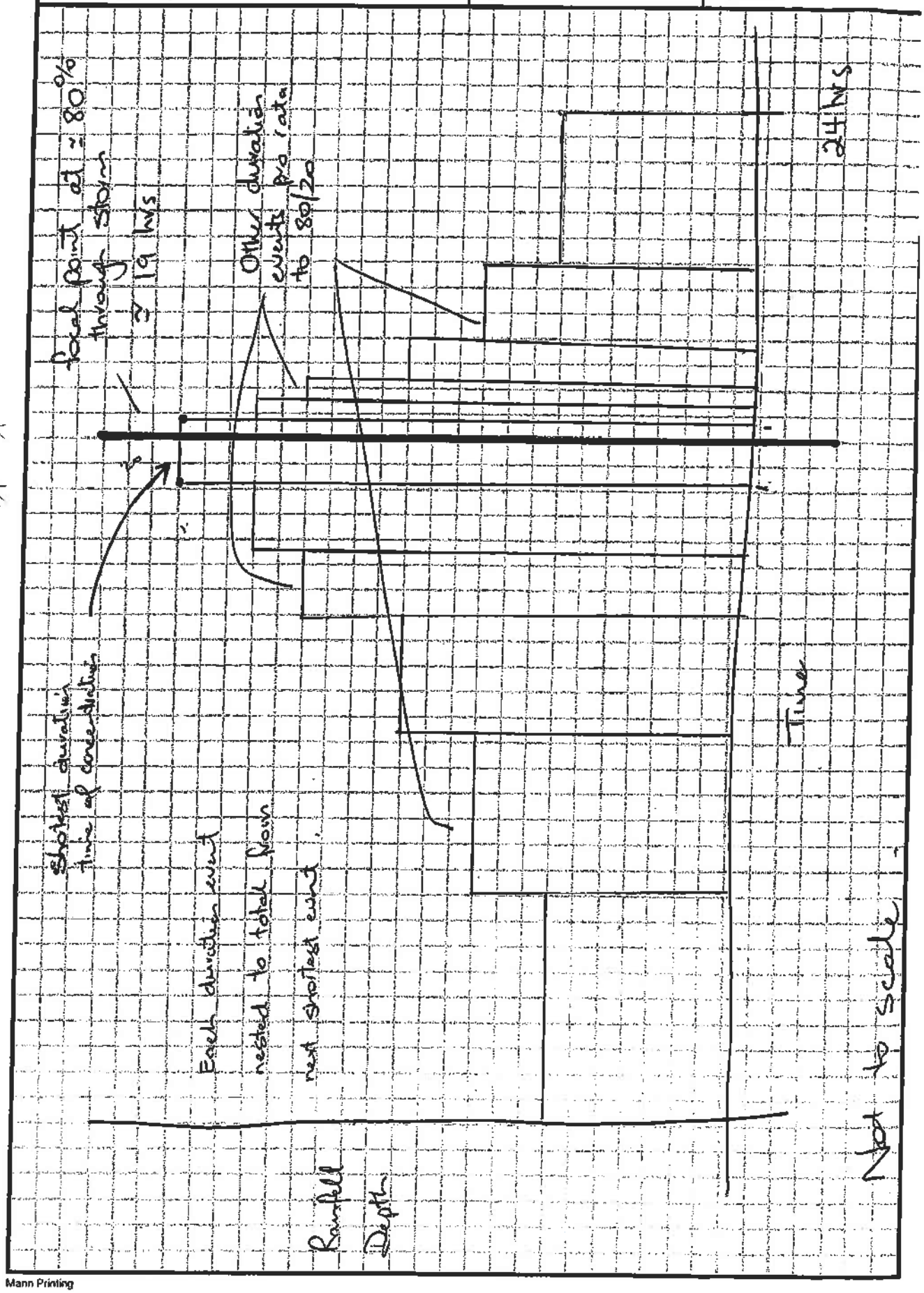
| Scenario | 1 | 2 | 3 | 4 | 5* | 6 |
|---|---|---|---|---|---|---|
| Scenario name | 10yr 24hr | 10yr 72hr | 100yr cc 24hr | 100yr cc 72hr | 100yr cc 24hr | |
| Tailwater level. | 1.63 | 1.63 | 1.81 | 1.81 | 1.81. | |
| Pond outlet pipe 1200ϕ | x2 | x2 | x2 | x2 | x3 | |
| Onsite pond top water level | 1.82 | 1.82 | 2.20 | 2.20 | 2.13 | |
| Pond Qp to Kope Canal | 1.53x2 3.06 m³/s | 1.53x2 3.06 m³/s | 2.44 x 2 4.88 m³/s | 2.44x2 4.88 m³/s | 2.21 x3 6.63 m³/s | |

Note: Scenario 5 pond outlet weir length. 10m

**Environment B·O·P**
(Bay of Plenty Regional Council)
P.O. Box 364
Whakatane

| JOB NAME | PAGE NO. |
|---|---|
| SECTION | |
| JOB NO. | DESIGNED |
| DATE | CHECKED |

CALCULATION SHEET



10yr current rainfall

| Duration, hr | Duration, min | Rainfall depth | Rainfall intensity, mm/min | Focal point. Min | Off peak rainfall depth | Off peak duration | Off peak intensity | 24hr rain Start | 24hr rain Stop | 72hr rain Start | 72hr rain Stop |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | 10 | 15.9 | 1.59 | 8 | | | | 1144 | 1154 | 3448 | 3458 |
| | 20 | 22.5 | | 16 | 6.6 | 10 | 0.66 | 1136 | 1156 | 3440 | 3460 |
| | 30 | 27.6 | | 24 | 5.1 | 10 | 0.51 | 1128 | 1158 | 3432 | 3462 |
| 1 | 60 | 38.9 | | 48 | 11.3 | 30 | 0.376666667 | 1104 | 1164 | 3408 | 3468 |
| 2 | 120 | 51.4 | | 96 | 12.5 | 60 | 0.208333333 | 1056 | 1176 | 3360 | 3480 |
| 6 | 360 | 79.8 | | 288 | 28.4 | 240 | 0.118333333 | 864 | 1224 | 3168 | 3528 |
| 12 | 720 | 105.4 | | 576 | 25.6 | 360 | 0.071111111 | 576 | 1296 | 2880 | 3600 |
| 24 | 1440 | 139.1 | | 1152 | 33.7 | 720 | 0.046805556 | 0 | 1440 | 2304 | 3744 |
| 48 | 2880 | 169.5 | | 2304 | 30.4 | 1440 | 0.021111111 | | | 1152 | 4032 |
| 72 | 4320 | 190.3 | | 3456 | 20.8 | 1440 | 0.014444444 | | | 0 | 4320 |

24hr rain

| From, min | To, min | |
|---|---|---|
| 0 | 576 | 0.046805556 |
| 577 | 864 | 0.071111111 |
| 865 | 1056 | 0.118333333 |
| 1057 | 1104 | 0.208333333 |
| 1105 | 1128 | 0.376666667 |
| 1129 | 1136 | 0.51 |
| 1137 | 1144 | 0.66 |
| 1145 | 1154 | 1.59 |
| 1155 | 1156 | 0.66 |
| 1157 | 1158 | 0.51 |
| 1159 | 1164 | 0.376666667 |
| 1165 | 1176 | 0.208333333 |
| 1177 | 1224 | 0.118333333 |
| 1225 | 1296 | 0.071111111 |
| 1297 | 1440 | 0.046805556 |

72hr rain

| From, min | To, min | |
|---|---|---|
| 0 | 1152 | 0.014444444 |
| 1153 | 2304 | 0.021111111 |
| 2305 | 2880 | 0.046805556 |
| 2881 | 3168 | 0.071111111 |
| 3169 | 3360 | 0.118333333 |
| 3361 | 3408 | 0.208333333 |
| 3409 | 3432 | 0.376666667 |
| 3433 | 3440 | 0.51 |
| 3441 | 3448 | 0.66 |
| 3449 | 3458 | 1.59 |
| 3459 | 3460 | 0.66 |
| 3461 | 3462 | 0.51 |
| 3463 | 3468 | 0.376666667 |
| 3469 | 3480 | 0.208333333 |
| 3481 | 3528 | 0.118333333 |
| 3529 | 3600 | 0.071111111 |
| 3601 | 3744 | 0.046805556 |
| 3745 | 4032 | 0.021111111 |
| 4033 | 4320 | 0.014444444 |

16.6c
26.9c
22.2c
9.0c

100yr with climate change

| Duration, hr | Duration, min | Rainfall depth | Rainfall intensity, mm/min | Focal point. Min | Off peak rainfall depth | Off peak duration | Off peak intensity | 24hr rain | | 72hr rain | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | Start | Stop | Start | Stop |
| | 10 | 31.8 | 3.18 | 8 | | | | 1144 | 1154 | 3448 | 3458 |
| | 20 | 45 | | 16 | 13.2 | 10 | 1.32 | 1136 | 1156 | 3440 | 3460 |
| | 30 | 55 | | 24 | 10 | 10 | 1 | 1128 | 1158 | 3432 | 3462 |
| 1 | 60 | 77.8 | | 48 | 22.8 | 30 | 0.76 | 1104 | 1164 | 3408 | 3468 |
| 2 | 120 | 102.6 | | 96 | 24.8 | 60 | 0.413333333 | 1056 | 1176 | 3360 | 3480 |
| 6 | 360 | 159.2 | | 288 | 56.6 | 240 | 0.235833333 | 864 | 1224 | 3168 | 3528 |
| 12 | 720 | 210.1 | | 576 | 50.9 | 360 | 0.141388889 | 576 | 1296 | 2880 | 3600 |
| 24 | 1440 | 277.4 | | 1152 | 67.3 | 720 | 0.093472222 | 0 | 1440 | 2304 | 3744 |
| 48 | 2880 | 338 | | 2304 | 60.6 | 1440 | 0.042083333 | | | 1152 | 4032 |
| 72 | 4320 | 379.4 | | 3456 | 41.4 | 1440 | 0.02875 | | | 0 | 4320 |

24hr rain

| From, min | To, min | |
|---|---|---|
| 0 | 576 | 0.093472222 |
| 577 | 864 | 0.141388889 |
| 865 | 1056 | 0.235833333 |
| 1057 | 1104 | 0.413333333 |
| 1105 | 1128 | 0.76 |
| 1129 | 1136 | 1 |
| 1137 | 1144 | 1.32 |
| 1145 | 1154 | 3.18 |
| 1155 | 1156 | 1.32 |
| 1157 | 1158 | 1 |
| 1159 | 1164 | 0.76 |
| 1165 | 1176 | 0.413333333 |
| 1177 | 1224 | 0.235833333 |
| 1225 | 1296 | 0.141388889 |
| 1297 | 1440 | 0.093472222 |

72hr rain

| From, min | To, min | |
|---|---|---|
| 0 | 1152 | 0.02875 |
| 1153 | 2304 | 0.042083333 |
| 2305 | 2880 | 0.093472222 |
| 2881 | 3168 | 0.141388889 |
| 3169 | 3360 | 0.235833333 |
| 3361 | 3408 | 0.413333333 |
| 3409 | 3432 | 0.76 |
| 3433 | 3440 | 1 |
| 3441 | 3448 | 1.32 |
| 3449 | 3458 | 3.18 |
| 3459 | 3460 | 1.32 |
| 3461 | 3462 | 1 |
| 3463 | 3468 | 0.76 |
| 3469 | 3480 | 0.413333333 |
| 3481 | 3528 | 0.235833333 |
| 3529 | 3600 | 0.141388889 |
| 3601 | 3744 | 0.093472222 |
| 3745 | 4032 | 0.042083333 |
| 4033 | 4320 | 0.02875 |

# *Onsite Pond Sizing*

| | | | |
|---|---|---|---|
| Job name | Shaw Rd | File name | Pond Option |
| Job number | 1368 | Path | S:\SS\Ongoing Projects\1368 |
| Date | 22/01/2013 | Sheet name | Treatment Pond |

Onsite Pond

| | | |
|---|---|---|
| Surface Level, RL | 2 m | |
| Permanent Water Level | 1 m | (assumed) |
| Top Length | 120 m | |
| Top Width | 50 m | |
| Side slopes | 5 H to 1 V | |

| Pond Level | Pond Depth | Length, m | Width, m | Area, m2 | Volume, m3 | Total Volume, m3 |
|---|---|---|---|---|---|---|
| 2.5 | 2.5 | 500 | 40 | 20000 | 6500 | 15375 |
| 2 | 2 | 120 | 50 | 6000 | 2794 | 8875 |
| 1.5 | 1.5 | 115 | 45 | 5175 | 2394 | 6081 |
| 1 | 1 | 110 | 40 | 4400 | 2019 | 3688 |
| 0.5 | 0.5 | 105 | 35 | 3675 | 1669 | 1669 |
| 0 | 0 | 100 | 30 | 3000 | 0 | 0 |

| | | | |
|---|---|---|---|
| Permanent Storage | 3688 | $m^3$ | (From RL0 to RL1.0m) |
| Live Storage | 5188 | $m^3$ | (From RL1.0 to RL2.0m) |
| 100yr | 8875 | $m^3$ | Above RL2.0 |

↳ accomodated within low lying areas adjacent to SH30.

Stormwater Solutions Consulting Ltd
PO Box 25 598, St Heliers, Auckland
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# MEMO

To: David Bewley

From: Bronwyn Rhynd

CC: Barney Gray, John Hessling

Date: 20th February 2013

Re: Shaw Road Stormwater Strategy - addendum

With reference to your email, dated 12th February 2013, regarding further information with respect to the stormwater strategy for the proposed Shaw Road plan change application we have reviewed the staging and updated the assessment. The outcomes of this review are presented in this addendum to the previous report titled "*Stormwater Management Feasibility Study for Shaw Road, Whakatane"*, with supporting calculation appended for reference.

## 1 Staging of the development

Staging of the re-zoned land can be undertaken as the land become available. At present the owner of 220 State Highway 30 (SH30) is interested in starting on a residential development. This stage has been considered at Stage 1 of the re-zoned land.

Stage 1 is 7.16ha in total and consists of the following Lots:

- 220 SH30; Lot 2 DPS 387805
- 39 Shaw Road; Lot 1 DPS 387805

## 2 Stormwater strategy for Stage 1

The stormwater strategy for Stage 1 follows that of the overall re-zoned area, being utilization of a pond for treatment and partial attenuation of flows with gravity discharge to Kopeopeo Canal.

The pond will be located adjacent to the western boundary and in a position that can be incrementally increased in size once further parcels of land are ready for development, as shown in drawing 1368/SK04 attached. The pond will have an outlet structure which will have the conveyance capacity for the fully developed total re-zoned area.

The pond will treat the runoff within the permanent water volume. The required volume is 1222m$^3$ for treatment purposes based on rainfall depth of 25.3mm and impervious/pervious ratio of 70/30. The pond characteristics are shown in Table 1.

**Table 1: Pond characteristics**

| Pond Characteristics | |
|---|---|
| Pond footprint[1] | 2800m$^2$ |
| Levels | |
| *Top of the pond* | *RL2.0m* |
| *Permanent water level* | *RL1.0m (assumed)* |
| *Base of the pond* | *RL0.0m* |
| Permanent storage volume | 1313m$^3$ (min) |
| Side slope | 5(H) to 1(V) |

Notes:

1. The pond foot print is at RL 2m and assumes filling of surrounding low lying area. Storage volume at foot print area will allow for a certain amount of detention volume

The remainder of the re-zoned land will discharge to Marshalls Drain which is an open drain adjacent to SH30. Once Marshalls Drain meets the location of the pond it can be discharged to the Kopeopeo Canal via 900mm diameter pipeline, as shown in drawing 1368/SK04.

## 2.1 Stage 1 discharge to Kopeopeo Canal

Based on the partial attenuation scenario[1] for the discharge to the Kopeopeo Canal and assessment of top water levels and peak flow rates has been undertaken. The outcomes of this assessment are presented in Table 2.

**Table 2: Peak flows and top water level**

| Rainfall event (ARI) | Peak flow (m$^3$/s) | | Top water level (RLm) | |
|---|---|---|---|---|
| | Existing | Proposed | Site (proposed) | Canal |
| 10yr | 0.46 | 1.36 | 1.67 | 1.63 |
| 100yr CC | 0.92 | 2.52 | 1.91 | 1.81 |

**Notes:**

1. Pond dimensions as per Table 1,

During the 10yr and 100yr rainfall events the top water levels are within the pond, as the top of the pond is at RL2.0m. There is sufficient head within the pond to provide gravity flow through the twin 1200mm diameter outlet pipes, which are flap gated to prevent backflow (into the pond).

[1] See section 6 of Stormwater Management Feasibility Study report.

The remainder of the area for re-zone will bypass the pond into Marshalls Drain, which in turn enters the Kopeopeo Canal through 900mm diameter culvert.

## 2.2 Effect of discharge to Kopeopeo Canal

The increase of peak flow for the fully developed Stage 1 can be accommodated within the Kopeopeo Canal which is controlled by the flood gates downstream at the confluence with the Whakatane River and the Kopeopeo Canal pump station. During rainfall events the operation of the pump station is determined by the top water level within the canal and the operative settings of the pump.

This assessment is based on the top water level within the canal remaining as per current situation and the conveyance of the additional flow being accommodated within the canal. At the time of writing the pump capacity and flow rates at the pump station were not available

However for guidance an assessment could be based on a first principle approach of the effect of the increase of runoff from the change to residential zoning. The catchment for the Kope-Orini canal system is 4960ha, of which the re-zoned land area is 21.75ha of which Stage 1 is 7.16ha. Therefore the increase of runoff would be for an equivalent area of rural state land[2]. An assessment of the equivalent rural state land area is presented in Table 3.

**Table 3: Equivalent land area assessment**

| **Characteristic** | **Area (ha)** | | |
|---|---|---|---|
| | **Total area as zoned** | **Rural zoned land equivalent (post residential development)** | |
| | | **area** | **% of total area** |
| Kope-Orini catchment | 4690 | 4690 | 100 |
| Shaw Road re-zone land | 21.75 | 50.46 | 1 |
| Stage 1 – Shaw Road | 7.16 | 16.61 | 0.3 |

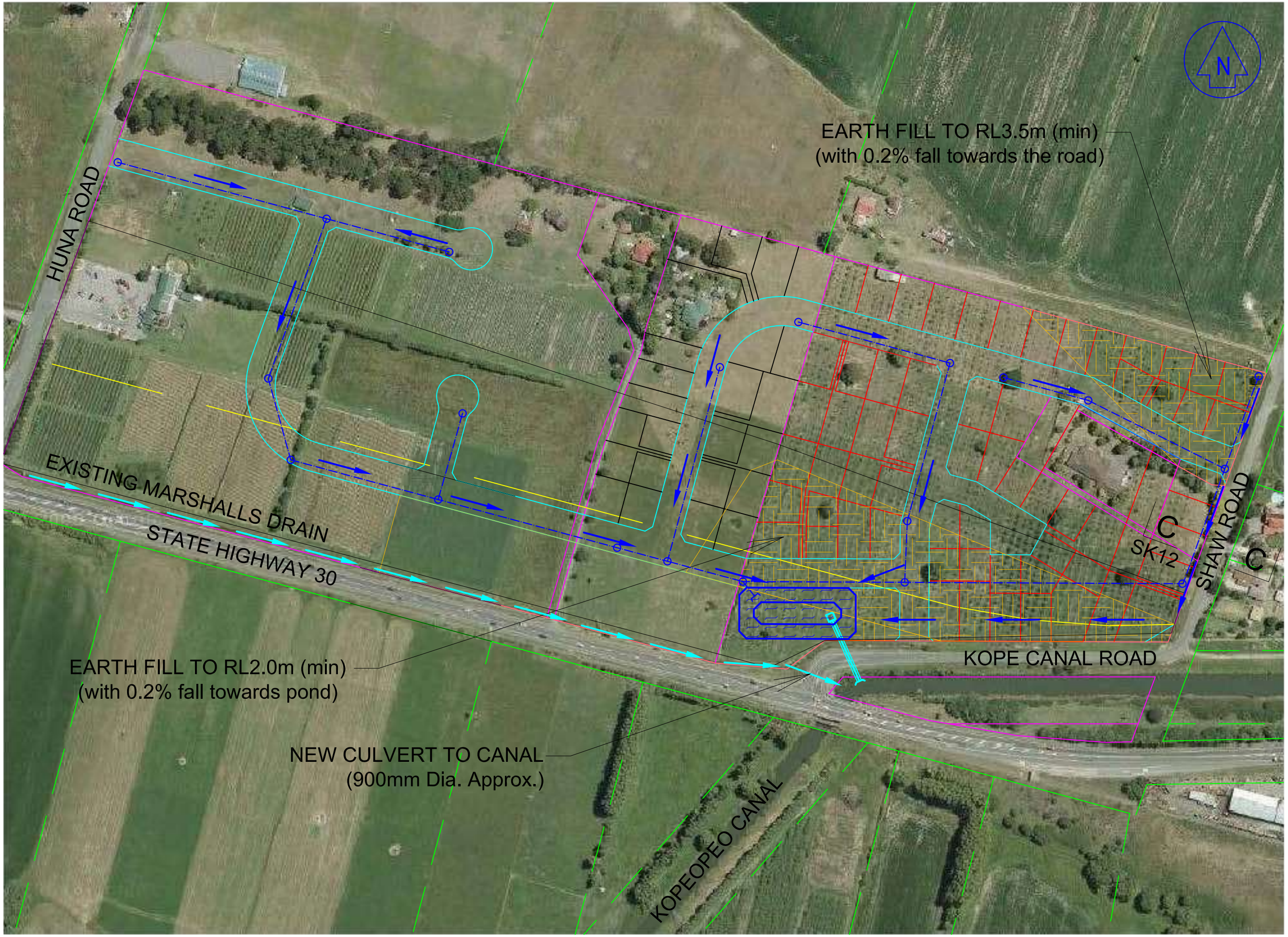
The total re-zoned Shaw Road area would have an area which is equivalent to 1% of the total Kope-Orini canal system catchment, whilst the Stage 1 is 0.3%. Therefore it is likely that the canal and pump station would have capacity for the increase in flow and for both Stage 1 and the fully developed Shaw Road area.

# 3 Costs of stormwater management

The costs of the stormwater strategy can be estimated for the works within the re-zoned land, with respect to the pond development and outlet structure. However the increase in pump capacity at the Kopeopeo Pump station need to be based on the increase in demand required. This is not available at the time of writing.

[2] Equivalent area of rural state land = fully developed residential area produces increase in runoff at a rate equivalent to the ratio of runoff coefficients between residential and rural, i.e. 0.58/0.25. Therefore this directly corresponds increasing the rural land by the same ratio

# Appendix A - Drawings

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2. CONTRACTOR TO CHECK AND VERIFY ALL DIMENSIONS, LEVELS AND ANGLES ON SITE PRIOR TO COMMENCING WORK. REFER FIGURED DIMENSIONS ONLY **DO NOT SCALE**. ANY DISCREPANCIES MUST BE REPORTED TO THE PROJECT MANAGER.
3. REFER TO ROSS OVERINGTON SURVEYORS CONCEPT PLAN

**LEGEND:**

- EARTH FILL
- STORMWATER POND
- OVERLAND FLOW PATH
- PRIMARY PIPE NETWORK
- EXISTING MARSHALLS DRAIN

| REV | DATE: | AMENDMENT: |
|---|---|---|
| | | |
| | | |
| | | |
| | | |
| | | |

CLIENT:
PAG Enterprises Ltd

Stormwater SOLUTIONS
Stormwater Solutions Consulting Ltd
PO Box 25 598, St Heliers, Auckland 1740
www.stormwatersolutions.co.nz

FOR PLAN CHANGE APPLICATION ONLY

PROJECT:
PROPOSED RESIDENTIAL ZONE
SHAW ROAD
WHAKATANE

TITLE:
STORMWATER MANAGEMENT LAYOUT PLAN (PRELIMINARY)

| DATE: | SCALE: |
|---|---|
| Feb 2013 | 1:2500 @A3 |

| DRAWING NO: | REVISION |
|---|---|
| 1368/SK04 | |

| DRAWN: | CHECKED: |
|---|---|
| W.L. | B.R. |

# Appendix B - Calculations

#1368
18/2/13



Attn: Peak flows & volumes for Carey's land only.

1. Lot 2 DP 387805. 220 SH30
   Lot 1 " " 39 Shaw Road.

(a) Lot 2 = 220 SH30 = 6.5870 ha } ref Overton
(b) Lot 1 = 39 Shaw Road = 0.57 ha } Survey plan
assessment 28/11/12

2. WQV.
(a) rainfall depth = 25.3mm.
(b) Area x C ⇒ C = 0.9 impervious
C = 0.15 pervious.

(c) volume ⇒ 0.9 x 70% x 7.157 ha + 0.15 x 30% x 7.157
CA = 4.83
V = 4.83 x 25.3mm = 1222 m³.

3. Pond sizing
Available area at south western corner as per scheme plan
⇒ 100m x 35m
⇒ Pond sizing (sheet 2)
⇒ At above RL 20m. extent of ponding will encroach into the low lying area of neighbouring properties.

4. Peak flow for Stage 1 for existing scenario.
(a) 14.5927 ha = 0.93 m³/s 10yr & 1.87 m³/s 100yr CC
(see MOUSE modelling Run 09)
(b) 7.157 / 14.5927 x 0.93 m³/s = 0.456 m³/s 10yr
(c) 7.157 / 14.5927 x 1.87 m³/s = 0.917 m³/s

(d) difference between existing & proposed.

| ARI | Existing | Proposed. | Diff. |
|---|---|---|---|
| 10 | 0.456 | 1.36 | 0.904 |
| 100yr CC | 0.917 | 2.52 | 1.603 |

# *Onsite Pond Sizing*

| | | | |
|---|---|---|---|
| Job name | Shaw Rd | File name | Pond Option |
| Job number | 1368 | Path | S:\SS\Ongoing Projects\1368 |
| Date | 18/02/2013 | Sheet name | Treatment Pond (220 SH30) (2) |

| Onsite Pond | | |
|---|---|---|
| Surface Level, RL | 2 m | |
| Permanent Water Level | 1 m | (assumed) |
| Top Length | 80 m | |
| Top Width | 35 m | |
| Side slopes | 5 H to 1 V | |

| Pond Level | Pond Depth | Length, m | Width, m | Area, m2 | Volume, m3 | Total Volume, m3 |
|---|---|---|---|---|---|---|
| 2.5 | 2.5 | 500 | 50 | 25000 | 6950 | 10525 |
| 2 | 2 | 80 | 35 | 2800 | 1263 | 3575 |
| 1.5 | 1.5 | 75 | 30 | 2250 | 1000 | 2313 |
| 1 | 1 | 70 | 25 | 1750 | 763 | 1313 |
| 0.5 | 0.5 | 65 | 20 | 1300 | 550 | 550 |
| 0 | 0 | 60 | 15 | 900 | 0 | 0 |

| | | | |
|---|---|---|---|
| Permanent Storage | 1313 | $m^3$ | (From RL0 to RL1.0m) |
| Live Storage | 2263 | $m^3$ | (From RL1.0 to RL2.0m) |
| 100yr | ~~3575~~ 6950 | $m^3$ | Above RL2.0 |

Aim: MOUSE modelling (Run09)
- Modified from Run06
- Stage 1 development

1. Stage 1 development
220 Sh130. Area = 6.5850 ha } ⇒ total = 7.157 ha. Assume 70% imp
39 Shaw Rd = 0.57 ha } ⇒ imp = 5.0019 per = 2.1471

Remaining catchment = 10.7935 + 0.517 + 3.2817 = 14.5927 ha. Assume 100% per.

2. SW discharge
Stage 1 ⇒ Onsite SW pond → Kope Canal
Remainder → Marshalls Drain → Kope

Update onsite pond size as per sheet 2

3. Modelled results

| | 10yr 2hr | 10yr 72hr | 100yr CC 2hr | 100yr CC 72hr |
|---|---|---|---|---|
| ISP | | | | |
| Stage 1 imp. | 1.20 m³/s | 1.20 m³/s | 2.39 m³/s | 2.39 m³/s |
| Stage 1 per | 0.14 m³/s | 0.14 m³/s | 0.28 m³/s | 0.28 m³/s |
| Remaining area | 0.93 m³/s | 0.93 m³/s | 1.87 m³/s | 1.87 m³/s |
| Total runoff volume | 12714 m³ | 17632 m³ | 25908 m³ | 35666 m³ |
| | | | | |
| Onsite pond | | | | |
| top water level | 1.57 m | 1.67 m | 1.91 m | 1.91 m |
| Peak discharge via 1200 | 1.36 m³/s | 1.36 m³/s | 2.52 m³/s | 2.52 m³/s |

# *Onsite Pond Sizing*

| | | | |
|---|---|---|---|
| Job name | Shaw Rd | File name | Pond Option |
| Job number | 1368 | Path | S:\SS\Ongoing Projects\1368 |
| Date | 18/02/2013 | Sheet name | Treatment Pond (220 SH30) (2) |

Onsite Pond

| | | |
|---|---|---|
| Surface Level, RL | 2 m | |
| Permanent Water Level | 1 m | (assumed) |
| Top Length | 80 m | |
| Top Width | 35 m | |
| Side slopes | 5 H to 1 V | |

| Pond Level | Pond Depth | Length, m | Width, m | Area, m2 | Volume, m3 | Total Volume, m3 |
|---|---|---|---|---|---|---|
| 2.5 | 2.5 | 500 | 50 | 25000 | 6950 | 10525 |
| 2 | 2 | 80 | 35 | 2800 | 1263 | 3575 |
| 1.5 | 1.5 | 75 | 30 | 2250 | 1000 | 2313 |
| 1 | 1 | 70 | 25 | 1750 | 763 | 1313 |
| 0.5 | 0.5 | 65 | 20 | 1300 | 550 | 550 |
| 0 | 0 | 60 | 15 | 900 | 0 | 0 |

| | | | |
|---|---|---|---|
| Permanent Storage | 1313 | $m^3$ | (From RL0 to RL1.0m) |
| Live Storage | 2263 | $m^3$ | (From RL1.0 to RL2.0m) |
| 100yr | 6950 | $m^3$ | Above RL2.0 |

# 1368.

24/2/13

Am: Equivalent Rural Land run off for Shaw Road re-zone land.

1. Total Kope-Oriel Canal catchment = 4960ha.

2. Rezone area
   (a) total 21.75ha
   (b) Stage 1 7.16ha.

3. Runoff co-efficients
   C = 0.25 Rural
   C = 0.9 Impv.
   C = 0.3 Perv. post development.

4. Imp/pervious 1960 70/30

5. Eqv. rural land.
   (a) 21.75 x 0.7 = 15.225 ha

   (b) ∴ ave. C for 21.75ha = $\frac{15.225 \times 0.7 + 6525 \times 0.3}{21.75}$
   = 0.58

   (c) 0.58/0.25 ⇒ 2.32

   (d) 21.75ha x 2.32 = 50.46 ha
   7.16 x 2.32 = 16.61 ha.

6. Percentage of total catchment.
   (a) total Kope-Oriel = 4960 ha
   (b) 21.75/4960 = 0.4% or 50.46/4960 = 1%
   (c) 7.16/4960 = 0.1% or 16.61/4960 = 0.3%.

13 December 2012

Ref: LtR-0230/Dec 2012

Ross Overington Surveyors
PO Box 600
Phoenix House
WHAKATANE

Attention: John Hesseling

Dear John,

**Re: Preliminary site investigation for properties located at 220 and 234 State Highway 30, Whakatane, (Lot 3 DP 387805 and Lot 2 DPS 41267).**

At the request of Ross Overington Surveyors, Geosciences Ltd conducted a preliminary site investigation for the above properties. The investigation includes two properties located at 220 and 234 State Highway 30, legally described as Lot 3 DP 387805 and Lot 2 DPS 41267. The two properties will be referred to as "the site" in this report.

The investigation described in this report acknowledges the requirements of the National Environmental Standard (NES) (Reference 1) that came into effect on 1 January 2012. It was conducted in general accordance with the Ministry for the Environment's (MfE) Contaminated Land Management Guidelines *(CLMG) No. 1 'Guidelines for Reporting on Contaminated Sites in New Zealand'* and No. 5 *'Site Investigation and Analysis of Soils'* (References 3 and 4 respectively).

## 1.0 Introduction

Under the NES regulations, land is considered to be actually or potentially contaminated if an activity or industry on the Hazard Activities or Industries List (HAIL) has been, is or is more likely than not to have been undertaken on the land.

Horticultural activities are listed in A.10 on the HAIL and include market gardens, orchards, glasshouses and spray sheds. An assessment of the potential for soil contamination on site is therefore mandatory before a change in land use, subdivision and / or redevelopment of the site can be approved.

Studies conducted by various Regional Councils and District Health Boards have indicated that arsenic, copper, lead and organochlorine pesticides (including DDT) are the main contaminants detected on properties used for horticultural activities.

## 2.0 Objectives

The objectives of this investigation were to:

- assess the likelihood of any other HAIL activities occurring on site.
- to assess whether the surface soil at the site has elevated concentrations of organochlorine pesticides, arsenic, copper, and/or lead as a result of historical horticultural activities; and

- to assess the potential risk to human health and the environment as a result of potential residues of agrichemicals in soil on the site.

## 3.0 Scope of works

To achieve the objectives of the above mentioned investigation, the following scope of works was undertaken:

- a review of historical aerial photographs of the site and surroundings;
- a search of the property file and Land Information Memorandum (conducted by Ross Overington Surveyors);
- an inspection of the site;
- the collection of six composite soil samples from the site;
- laboratory analysis of soil samples for arsenic, copper, lead, cadmium and organochlorine pesticides (OCPs);
- a comparison of the laboratory results with the relevant national and international risk-based assessment guideline criteria; and
- preparation of a letter report summarising the results and recommendations of the investigation.

## 4.0 Site location and description

The site is located at the corner of SH30 and Shaw Rd, Whakatane (see Figure 1). The property at 220 SH30 is covered in olive trees and covers an area of 6.59 hectares. The property at 234 SH30 is paddock and covers an area of 3.28 hectares. A residential dwelling and associated out buildings are located in the north western corner of this property.

Surrounding landuse includes a berry farm to the west of the site and a combination of horticultural and farming activities to the east, north, and south of the site. The majority of the site is bordered by roads and driveways.

## 5.0 Soils, topography and drainage

The majority of the soil on site is described as Rawatu fine sandy loam. The parent material comprises alluvium and colluvium over laying a thin layer of Kaharoa Tephra.

The topography is generally flat with a very gentle rise running east-west in the north of the site. An open watercourse runs along the southern border of the site.

## 6.0 Historical investigation

A history of the site has been obtained by discussions with current landowners, a review of the historical aerial photographs and a search of the property file and Land Information Memorandum for the site.

### 6.1. Land owner information

Lot 2 DPS 41267 / 234 SH30

The western lot contains the site of the original house on the land. The present house is in very much the same location.

The land has always been used for grazing, originally by the Goiles and subsequently by the Christiansens (1991 – 1993) and the Meekels (1993 to present). Animals grazed have principally been cattle with occasional sheep. Cattle are presently grazed on the land. There is no evidence of any activity or industry on the Hazardous Activities or Industries List (HAIL) being undertaken on this land.

Personal communications with the Meekel family were conducted during the fieldwork for the investigation. Mr Meekel built the existing loading yard over twenty years ago and it was purpose built to load/unload stock onto trucks. There was no known sheep dip located on the land and no pesticide spraying of stock was carried out.

Historical aerial photographs confirm the above site history.

Lot 3 DP 387805 / 220 SH30

The eastern lot was originally owned and farmed by the Goiles family until 1987 when it was transferred to CI Abel. It was subsequently transferred to the Yates in 1991, to the Jensens in 1995 and to the Grays in 2006. The land was grazed until about 15 years ago when it was planted in olives by the Jensens.

Dry stock grazed have principally been cattle and occasionally sheep. There was no known sheep dip or cattle race located on the land. Cattle and horses are presently grazed on the land in conjunction with the olive trees. The only buildings on the land are some storage sheds, a pump shed and a water supply borehole. The grass around the olive trees was recently mowed.

A resource consent was granted to the Jensens in 2004 to construct and operate nine accommodation chalets with associated shops and offices. This was never pursued. The land was subdivided by the Grays in 2006 to create the present title.

Mr Gray has advised that the only chemical sprays used on the olive trees are Roundup, copper based sprays and hormone sprays.

Historical aerial photographs confirm the above site history

### 6.2. Property file and Land Information Memorandum

A search of the property files and LIMs was conducted by John Hesseling from Ross Overington Surveyors. No information referring to potentially contaminating activities or the use of hazardous substances on either of the two properties were found as part of these searches

### 6.3. Historical aerial photographs

Historical aerial photographs from 1944, 1987, 1995, 2002 and 2003 were viewed as part of the historical investigation. Copies of the historical aerial photographs are provided in Appendix A.

**1944**

It is difficult to determine distinctive features in the 1944 aerial photograph; however it appears that the site is open pasture. There is a house in the same location as the house currently occupying 234 SH30. Surrounding land use appears to be a mix of horticultural and farming activities.

**1987**

The 1987 aerial photograph is similar to that of 1944 except that the property at 220 SH30 is separated into three distinct paddocks. Surrounding land use appears to be a mix of horticultural and farming activities.

**1995/2002**

The 1995 and 2002 aerial photographs show the site is still open pasture. The property at 220 SH30 has been further divided into six paddocks. Surrounding land use is similar to that of the previous aerial photographs.

**2003**

The property at 220 SH30 has now been planted in olive trees while 234 SH30 remains as pasture. No other developments can be identified on site.

## 7.0 Site inspection

Geosciences Ltd undertook a site investigation on 26 November 2012. The site features were similar to that of the 2003 aerial photograph. The property at 220 SH30 consisted of a mature olive grove, with overgrown grass and weeds covering the ground.

The property at 220 SH30 was grassed pasture apart from the residential dwelling, associated buildings and the stock loading yard.

## 8.0 Potential for contamination

Apart from the olive grove, the historical investigation did not reveal any other (former or current) contaminating site activities on site. There was no evidence of fuel stored on site or any visible evidence of chemical storage sheds, old sheep dips or cattle races.

The potential for contamination is therefore considered to be from the use of agrichemicals associated with horticultural activities only. The main contaminants of concern are defined by Council as arsenic, copper, lead, cadmium and organochlorine pesticides.

## 9.0 Soil sampling and analysis

Soil sampling was conducted on the same day as the site inspection. Soil sampling locations are shown in Figure 2.

### 9.1. Soil sampling

Soil samples were taken from the top 75 mm of topsoil by means of a stainless steel corer with an inside diameter of 2.5 cm. Sampling equipment was decontaminated between each sample in accordance with our internal Quality Control procedures.

The sampling protocol followed was in accordance with the *Contaminated Land Management Guidelines (CLMG) No. 5 – Site Investigation and Analysis of Soils"* (Reference 4). According to this guideline the potential for contamination from horticultural activities is generally regarded to be low and uniform. Consequently, five composite soil samples comprising three sub samples were collected in a grid based sampling pattern across the olive grove (Lot 3 DP 387805).

One composite soil sample comprising three sub samples was collected from the pasture (Lot 2 DPS 41267).

A brief sample description was recorded in the field at the time of sample collection. Each sample core was placed in a plastic zipper bag with the date, sample identification number, location, and initials of sampler noted on the bag. The composite soil samples were mixed in the field.

### 9.2. Laboratory analysis and quality control

Sample bags were placed in a chilly bin with a chain of custody form (COC) indicating the analysis to be preformed. Soil samples were dispatched to RJ Hill Laboratories Ltd in Hamilton for analysis of arsenic, copper, lead, cadmium and OCPs including DDT. RJ Hill Laboratories are accredited by International Accreditation New Zealand for the analysis undertaken. On receipt of the samples Hill laboratories returned a copy of the COC to us.

## 10.0 Acceptance criteria and relevant guidelines

The Resource Management (*National Environmental Standard for Assessing and Managing Contaminants in Soil to Protect Human Health*) Regulations 2011 came into effect on 1 January 2012. The NES mandates soil contaminant standards for the protection of human health for twelve priority pollutants for various land use criteria.

## 11.0 Analytical results

A comparison of the analytical results with the relevant guideline criteria is provided in Table 1 below. Human health criteria were taken from the *National Environmental Standard for Assessing and Managing Contaminants in Soil to Protect Human Health (2011)*. Copies of the original laboratory transcripts are attached to this report (Appendix B).

Although all samples collected as part of this investigation were composited from three sub samples, the guideline value provided in the table below has not been altered. Revision of the NES guideline values were not deemed necessary as laboratory results for heavy metals would still remain below the altered guideline value.

### 11.1. Heavy Metals

The concentrations of all the heavy metals analysed for were below the NES criteria for residential land use.

### 11.2. Oranochlorine pesticides

All organochlorine pesticides were below analytical instrumentation detection limits and are therefore not referred to in Table 1 (for a full list, refer to Appendix B).

## 12.0 Conclusion

A preliminary site investigation (PSI) has been conducted for the site located at 220 and 234 State Highway 30. The PSI concluded that horticulture (olive trees) was the only potential contaminating activity that occurred on site.

To further investigate the impact of the former used of agri-chemicals on the site six soil samples were collected from the olive grove and neighbouring pasture and analysed for arsenic, lead, copper, cadmium and OCPs. All the heavy metals and OCPs analysed for revealed concentrations within the NES guideline criteria for the protection of human health for residential land use.

Based on the above, Geosciences Ltd concludes that the properties legally described as Lot 3 DP 387805 and Lot 2 DPS 41267, SH30, Whakatane, are suitable for residential land use and it is highly unlikely that there will be a risk to human health or the environment as a result of former horticultural activities that occurred on site.

Should you have any queries regarding this report please do not hesitate to contact us on 09 476 0454 or Trish on 021 531966.

Yours sincerely

Trish Meyer
NES Qualified Practitioner
Geosciences Ltd

**Statement**

This Preliminary Soil Contamination Investigation has been prepared in accordance with the Resource Management (National Environmental Standard for Assessing and Managing Contaminants in Soil to Protect Human Health) Regulations 2011. It has been undertaken by a suitably qualified and experienced practitioner (SQEP); and reported on in accordance with the current edition of the Ministry for the Environment's *Contaminated* Land *Management guidelines No.1 – Reporting on Contaminated Sites in New Zealand*.

## References

1 Ministry for the Environment (2011) - *Draft Users Guide National Environmental Standard for Assessing and Managing Contaminants in Soil to Protect Human Health*. Ministry for the Environment, Wellington, New Zealand.

2 Ministry for the Environment (2011) – *Methodology for Deriving Standards for contaminants in Soil to Protect Human Health.* Ministry for the Environment, Wellington, New Zealand.

3 Ministry for the Environment (2003) — *Contaminated Land Management Guidelines No.1: Reporting on contaminated Sites in New Zealand*. Ministry for the Environment, Wellington, New Zealand.

4 Ministry for the Environment (2003) — *Contaminated Land Management Guidelines No.5: Site Investigation and Analysis of Soils*. Ministry for the Environment, Wellington, New Zealand.

## Limitations

This letter report has been prepared by Geosciences Ltd in response to and subject to the following limitations:

1. The specific instructions received from Ross Overington Surveyors.
2. This report comprises the formal report, documentation sections, tables, figures and appendices as referred to in this report and must not be released to any third party or copied in part without all the material included in this report for any reason;
3. The report relates to the site located at Lot 3 DP 387805 and Lot 2 DPS 41267as at the date of the report as conditions may change thereafter due to natural processes and/or site activities;
4. No warranty or guarantee is made in regard to any other use than as specified in the scope of works and only applies to the depth tested and reported in this report.

## Table 1: Heavy metals in soil[1]

| Element/ Compound | Comp 1 | Comp 2 | Comp 3 | Comp 4 | Comp 5 | Comp M | NES Human health levels residential/ land use[2] |
|---|---|---|---|---|---|---|---|
| **Arsenic** | 3 | 5 | 4 | 4 | 3 | 4 | 20 |
| **Cadmium** | 0.19 | 0.18 | 0.17 | 0.20 | 0.17 | 0.16 | 3 |
| **Copper** | 6 | 10 | 13 | 21 | 16 | 7 | NL |
| **Lead** | 4.1 | 6.7 | 6.0 | 12.2 | 6.7 | 7.6 | 210 |
| | | | | | | | |

**Notes:**

1. All metal concentrations measured in mg/kg
2. Newly approved National Environmental Standards (NES) for assessing and managing contaminants in soil to protect human health – Residential 10% produce (Reference 2)
3. NV – No Value
4. ND – None Detected
5. NL – No Limit
6. Values in **red** exceed the human health levels for residential no produce and the environmental discharge level.

# FIGURES


Figure 1: Locality

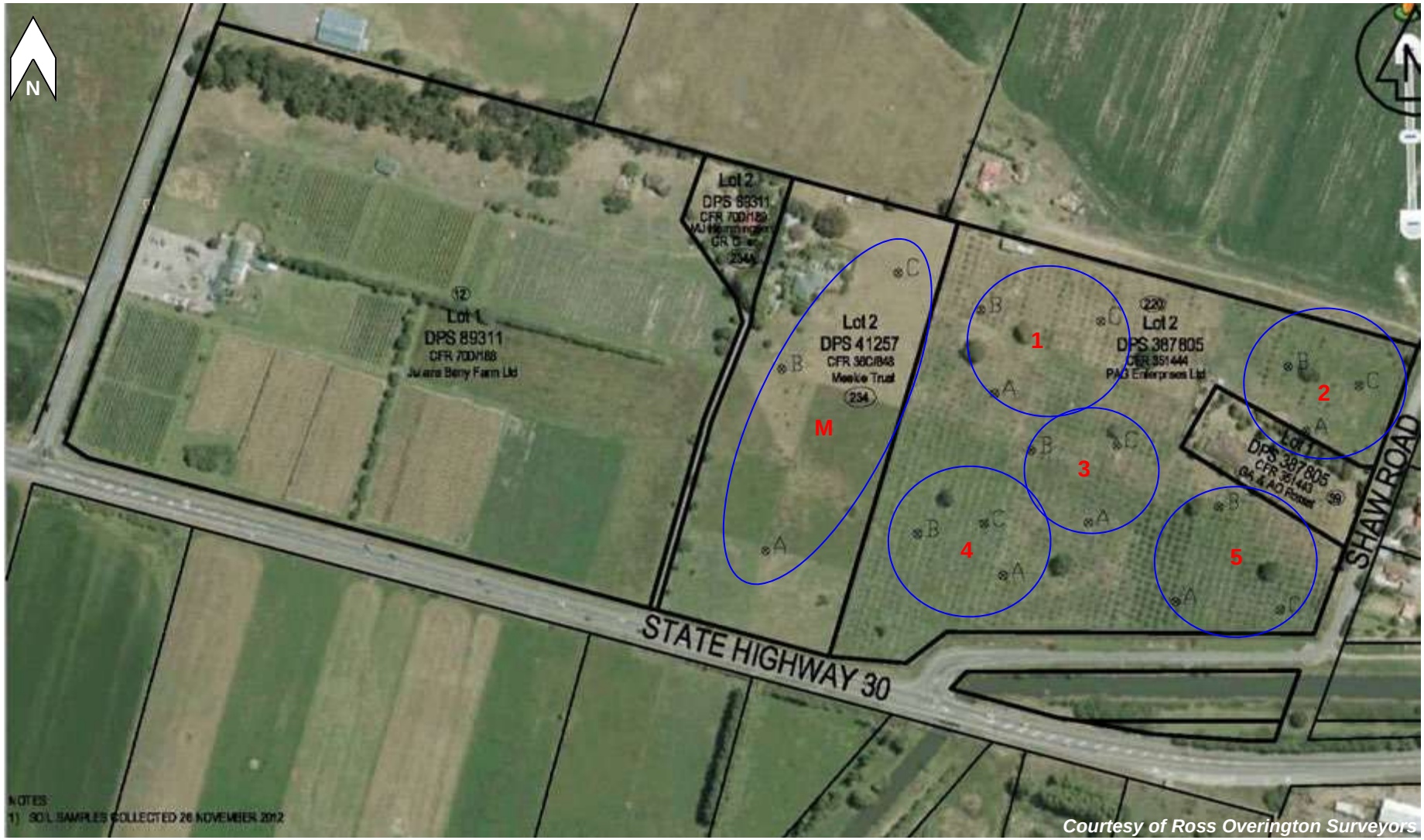
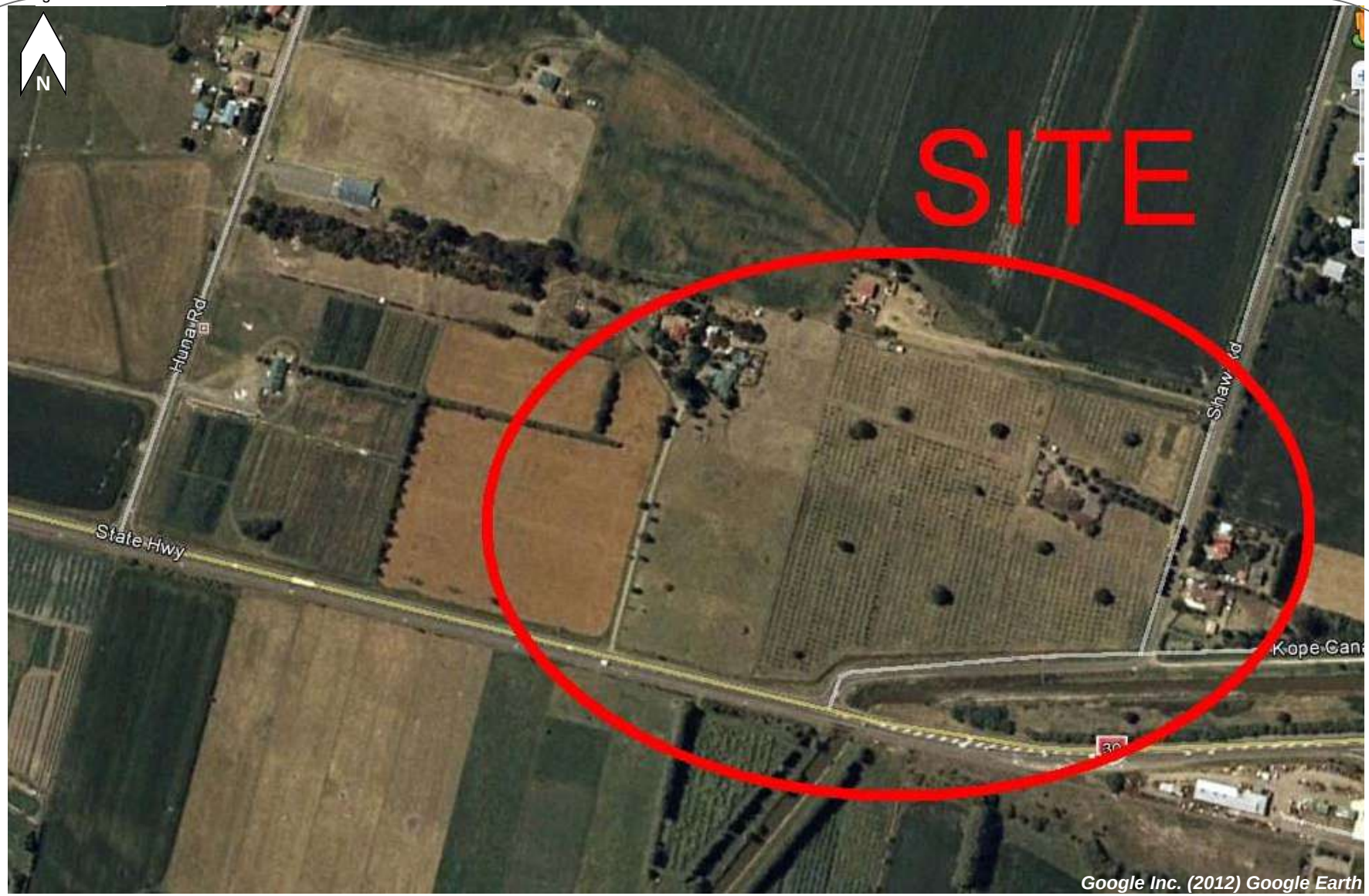



Figure 2: Sample locations

# APPENDIX A: HISTORICAL AERIAL PHOTOGRAPHS

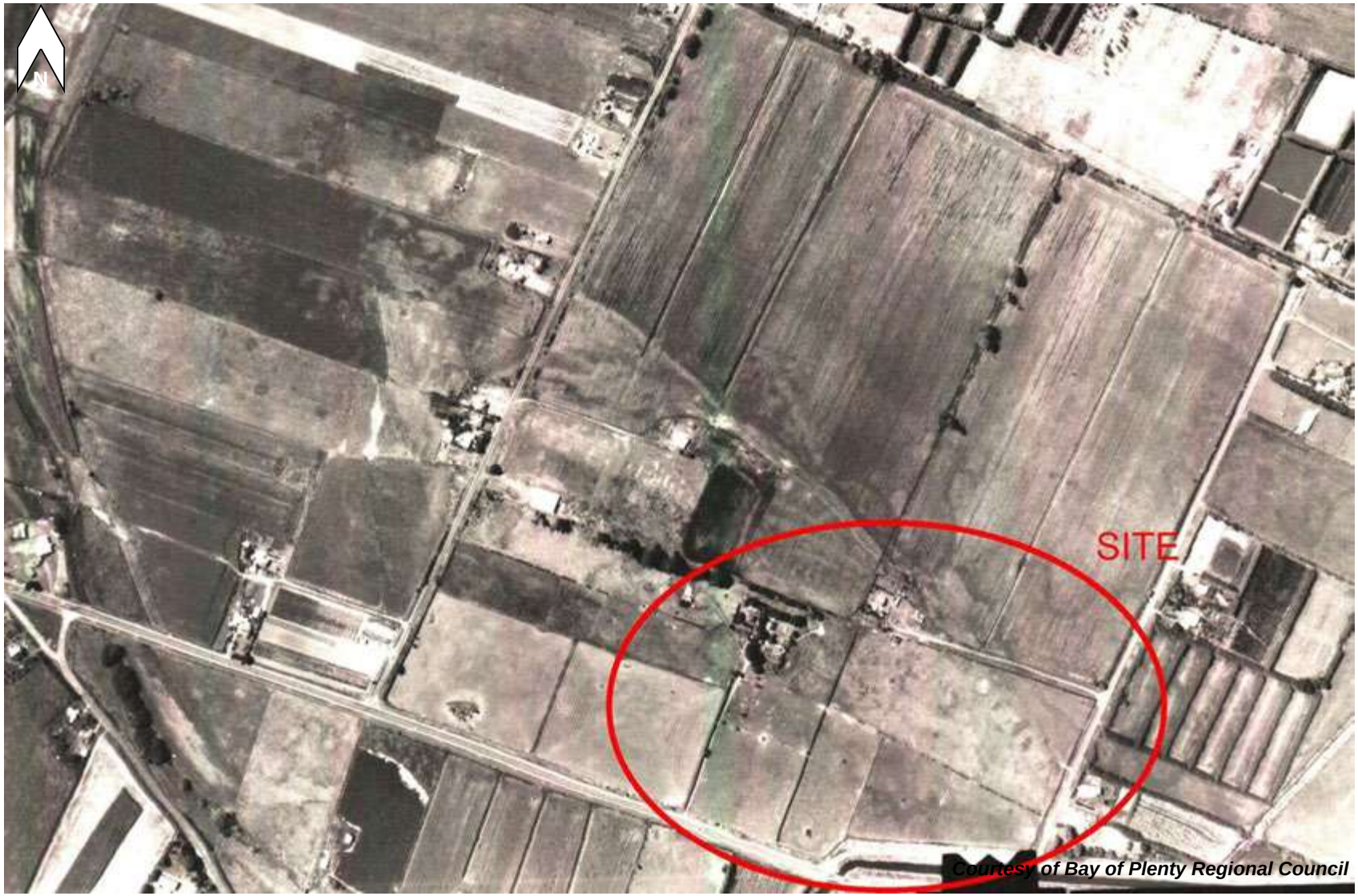

2007 Aerial Photograph

geosciences ltd

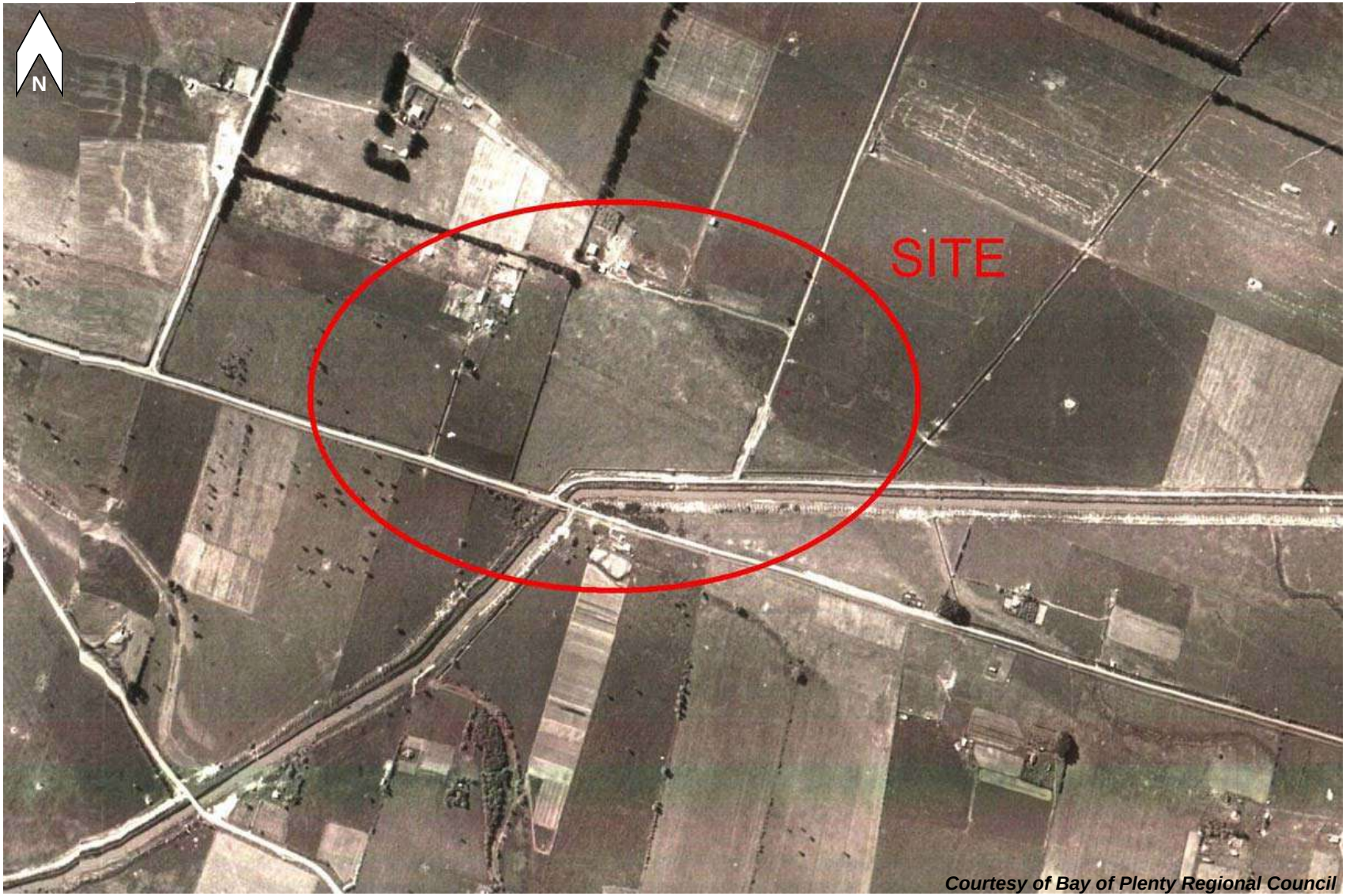



2003 Aerial Photograph


2002 Aerial Photograph




1987 Aerial Photograph




1944 Aerial Photograph

# APPENDIX B: LABORATORY TRANSCRIPTS

Hill Laboratories
BETTER TESTING BETTER RESULTS

R J Hill Laboratories Limited
1 Clyde Street
Private Bag 3205
Hamilton 3240, New Zealand

Tel +64 7 858 2000
Fax +64 7 858 2001
Email mail@hill-labs.co.nz
Web www.hill-labs.co.nz

# ANALYSIS REPORT



| | | | | |
|---|---|---|---|---|
| **Client:** | Geosciences Ltd | **Lab No:** | 1075192 | SPv1 |
| **Contact:** | Trish Meyer | **Date Registered:** | 29-Nov-2012 | |
| | C/- Geosciences Ltd | **Date Reported:** | 11-Dec-2012 | |
| | PO Box 35366 | **Quote No:** | | |
| | Browns Bay | **Order No:** | 0230 | |
| | AUCKLAND 0753 | **Client Reference:** | | |
| | | **Submitted By:** | Trish Meyer | |

**Sample Type: Soil**

| | | Sample Name: | Comp 1 | Comp 2 | Comp 3 | Comp 4 | Comp 5 |
|---|---|---|---|---|---|---|---|
| | | **Lab Number:** | 1075192.1 | 1075192.2 | 1075192.3 | 1075192.4 | 1075192.5 |
| Individual Tests | | | | | | | |
| Total Recoverable Arsenic | mg/kg dry wt | | 3 | 5 | 4 | 4 | 3 |
| Total Recoverable Cadmium | mg/kg dry wt | | 0.19 | 0.18 | 0.17 | 0.20 | 0.17 |
| Total Recoverable Copper | mg/kg dry wt | | 6 | 10 | 13 | 21 | 16 |
| Total Recoverable Lead | mg/kg dry wt | | 4.1 | 6.7 | 6.0 | 12.2 | 6.7 |
| Organochlorine Pesticides Screening in Soil | | | | | | | |
| Aldrin | mg/kg dry wt | | < 0.010 | < 0.011 | < 0.010 | < 0.011 | < 0.010 |
| alpha-BHC | mg/kg dry wt | | < 0.010 | < 0.011 | < 0.010 | < 0.011 | < 0.010 |
| beta-BHC | mg/kg dry wt | | < 0.010 | < 0.011 | < 0.010 | < 0.011 | < 0.010 |
| delta-BHC | mg/kg dry wt | | < 0.010 | < 0.011 | < 0.010 | < 0.011 | < 0.010 |
| gamma-BHC (Lindane) | mg/kg dry wt | | < 0.010 | < 0.011 | < 0.010 | < 0.011 | < 0.010 |
| cis-Chlordane | mg/kg dry wt | | < 0.010 | < 0.011 | < 0.010 | < 0.011 | < 0.010 |
| trans-Chlordane | mg/kg dry wt | | < 0.010 | < 0.011 | < 0.010 | < 0.011 | < 0.010 |
| Total Chlordane [(cis+trans)* 100/42] | mg/kg dry wt | | < 0.04 | < 0.04 | < 0.04 | < 0.04 | < 0.04 |
| 2,4'-DDD | mg/kg dry wt | | < 0.010 | < 0.011 | < 0.010 | < 0.011 | < 0.010 |
| 4,4'-DDD | mg/kg dry wt | | < 0.010 | < 0.011 | < 0.010 | < 0.011 | < 0.010 |
| 2,4'-DDE | mg/kg dry wt | | < 0.010 | < 0.011 | < 0.010 | < 0.011 | < 0.010 |
| 4,4'-DDE | mg/kg dry wt | | < 0.010 | < 0.011 | < 0.010 | < 0.011 | < 0.010 |
| 2,4'-DDT | mg/kg dry wt | | < 0.010 | < 0.011 | < 0.010 | < 0.011 | < 0.010 |
| 4,4'-DDT | mg/kg dry wt | | < 0.010 | < 0.011 | < 0.010 | < 0.011 | < 0.010 |
| Dieldrin | mg/kg dry wt | | < 0.010 | < 0.011 | < 0.010 | < 0.011 | < 0.010 |
| Endosulfan I | mg/kg dry wt | | < 0.010 | < 0.011 | < 0.010 | < 0.011 | < 0.010 |
| Endosulfan II | mg/kg dry wt | | < 0.010 | < 0.011 | < 0.010 | < 0.011 | < 0.010 |
| Endosulfan sulphate | mg/kg dry wt | | < 0.010 | < 0.011 | < 0.010 | < 0.011 | < 0.010 |
| Endrin | mg/kg dry wt | | < 0.010 | < 0.011 | < 0.010 | < 0.011 | < 0.010 |
| Endrin Aldehyde | mg/kg dry wt | | < 0.010 | < 0.011 | < 0.010 | < 0.011 | < 0.010 |
| Endrin ketone | mg/kg dry wt | | < 0.010 | < 0.011 | < 0.010 | < 0.011 | < 0.010 |
| Heptachlor | mg/kg dry wt | | < 0.010 | < 0.011 | < 0.010 | < 0.011 | < 0.010 |
| Heptachlor epoxide | mg/kg dry wt | | < 0.010 | < 0.011 | < 0.010 | < 0.011 | < 0.010 |
| Hexachlorobenzene | mg/kg dry wt | | < 0.010 | < 0.011 | < 0.010 | < 0.011 | < 0.010 |
| Methoxychlor | mg/kg dry wt | | < 0.010 | < 0.011 | < 0.010 | < 0.011 | < 0.010 |

| | | Sample Name: | Comp M | | | | |
|---|---|---|---|---|---|---|---|
| | | **Lab Number:** | 1075192.6 | | | | |
| Individual Tests | | | | | | | |
| Total Recoverable Arsenic | mg/kg dry wt | | 4 | - | - | - | - |
| Total Recoverable Cadmium | mg/kg dry wt | | 0.16 | - | - | - | - |
| Total Recoverable Copper | mg/kg dry wt | | 7 | - | - | - | - |

| Sample Type: Soil | | | | | | |
|---|---|---|---|---|---|---|
| **Sample Name:** | | Comp M | | | | |
| **Lab Number:** | | 1075192.6 | | | | |
| Individual Tests | | | | | | |
| Total Recoverable Lead | mg/kg dry wt | 7.6 | - | - | - | - |
| Organochlorine Pesticides Screening in Soil | | | | | | |
| Aldrin | mg/kg dry wt | < 0.010 | - | - | - | - |
| alpha-BHC | mg/kg dry wt | < 0.010 | - | - | - | - |
| beta-BHC | mg/kg dry wt | < 0.010 | - | - | - | - |
| delta-BHC | mg/kg dry wt | < 0.010 | - | - | - | - |
| gamma-BHC (Lindane) | mg/kg dry wt | < 0.010 | - | - | - | - |
| cis-Chlordane | mg/kg dry wt | < 0.010 | - | - | - | - |
| trans-Chlordane | mg/kg dry wt | < 0.010 | - | - | - | - |
| Total Chlordane [(cis+trans)* 100/42] | mg/kg dry wt | < 0.04 | - | - | - | - |
| 2,4'-DDD | mg/kg dry wt | < 0.010 | - | - | - | - |
| 4,4'-DDD | mg/kg dry wt | < 0.010 | - | - | - | - |
| 2,4'-DDE | mg/kg dry wt | < 0.010 | - | - | - | - |
| 4,4'-DDE | mg/kg dry wt | < 0.010 | - | - | - | - |
| 2,4'-DDT | mg/kg dry wt | < 0.010 | - | - | - | - |
| 4,4'-DDT | mg/kg dry wt | < 0.010 | - | - | - | - |
| Dieldrin | mg/kg dry wt | < 0.010 | - | - | - | - |
| Endosulfan I | mg/kg dry wt | < 0.010 | - | - | - | - |
| Endosulfan II | mg/kg dry wt | < 0.010 | - | - | - | - |
| Endosulfan sulphate | mg/kg dry wt | < 0.010 | - | - | - | - |
| Endrin | mg/kg dry wt | < 0.010 | - | - | - | - |
| Endrin Aldehyde | mg/kg dry wt | < 0.010 | - | - | - | - |
| Endrin ketone | mg/kg dry wt | < 0.010 | - | - | - | - |
| Heptachlor | mg/kg dry wt | < 0.010 | - | - | - | - |
| Heptachlor epoxide | mg/kg dry wt | < 0.010 | - | - | - | - |
| Hexachlorobenzene | mg/kg dry wt | < 0.010 | - | - | - | - |
| Methoxychlor | mg/kg dry wt | < 0.010 | - | - | - | - |

# SUMMARY OF METHODS

The following table(s) gives a brief description of the methods used to conduct the analyses for this job. The detection limits given below are those attainable in a relatively clean matrix. Detection limits may be higher for individual samples should insufficient sample be available, or if the matrix requires that dilutions be performed during analysis.

| Sample Type: Soil | | | |
|---|---|---|---|
| **Test** | **Method Description** | **Default Detection Limit** | **Samples** |
| Environmental Solids Sample Preparation | Air dried at 35°C and sieved, <2mm fraction.<br>Used for sample preparation.<br>May contain a residual moisture content of 2-5%. | - | 1-6 |
| Organochlorine Pesticides Screening in Soil | Sonication extraction, SPE cleanup, dual column GC-ECD analysis (modified US EPA 8082).. Tested on dried sample | - | 1-6 |
| Total Recoverable digestion | Nitric / hydrochloric acid digestion. US EPA 200.2. | - | 1-6 |
| Total Recoverable Arsenic | Dried sample, sieved as specified (if required).<br>Nitric/Hydrochloric acid digestion, ICP-MS, screen level. US EPA 200.2. | 2 mg/kg dry wt | 1-6 |
| Total Recoverable Cadmium | Dried sample, sieved as specified (if required).<br>Nitric/Hydrochloric acid digestion, ICP-MS, screen level. US EPA 200.2. | 0.10 mg/kg dry wt | 1-6 |
| Total Recoverable Copper | Dried sample, sieved as specified (if required).<br>Nitric/Hydrochloric acid digestion, ICP-MS, screen level. US EPA 200.2. | 2 mg/kg dry wt | 1-6 |
| Total Recoverable Lead | Dried sample, sieved as specified (if required).<br>Nitric/Hydrochloric acid digestion, ICP-MS, screen level. US EPA 200.2. | 0.4 mg/kg dry wt | 1-6 |

These samples were collected by yourselves (or your agent) and analysed as received at the laboratory.

Samples are held at the laboratory after reporting for a length of time depending on the preservation used and the stability of the analytes being tested. Once the storage period is completed the samples are discarded unless otherwise advised by the client.



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