PROJECT HOBSON TUNNEL DESIGN

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ABSTRACT

Project Hobson is Watercare's project to replace the Hobson Bay above ground sewer pipeline which serves a large part of Auckland City but is reaching the end of its life.

The replacement sewer is a 3 km long, 3.7 m internal diameter sewer tunnel running from Parnell to a new pump station in the Orakei Domain. The sewer tunnel will provide storage to capture wet weather flows and practically eliminate overflows from Watercare's wastewater system into Hobson Bay.

The tunnel was sized based on modeling which took account of the level of service, capacity of the downstream sewer and the upstream catchment characteristics. Alternative hydraulic system options were considered to develop the optimum, sustainable solution. 30 m deep vortex drop shafts have been included in the design to deliver the flow down to the tunnel and reduce erosion, odour and noise by controlling the flow within a vortex drop pipe.

A ventilation system is provided to ensure the new sewer does not cause odour complaints. The strategy involves the operation of a fan at the furthest downstream shaft (PS64 Wet Well) pulling air down the Hobson Bay sewer. The extracted air will be processed through a bark biofilter before venting to atmosphere.

KEYWORDS

Hydraulic Modelling, Hydraulic Arrangement, Overflow, Dropshaft, Ventilation, Sulphide Generation

1 INTRODUCTION

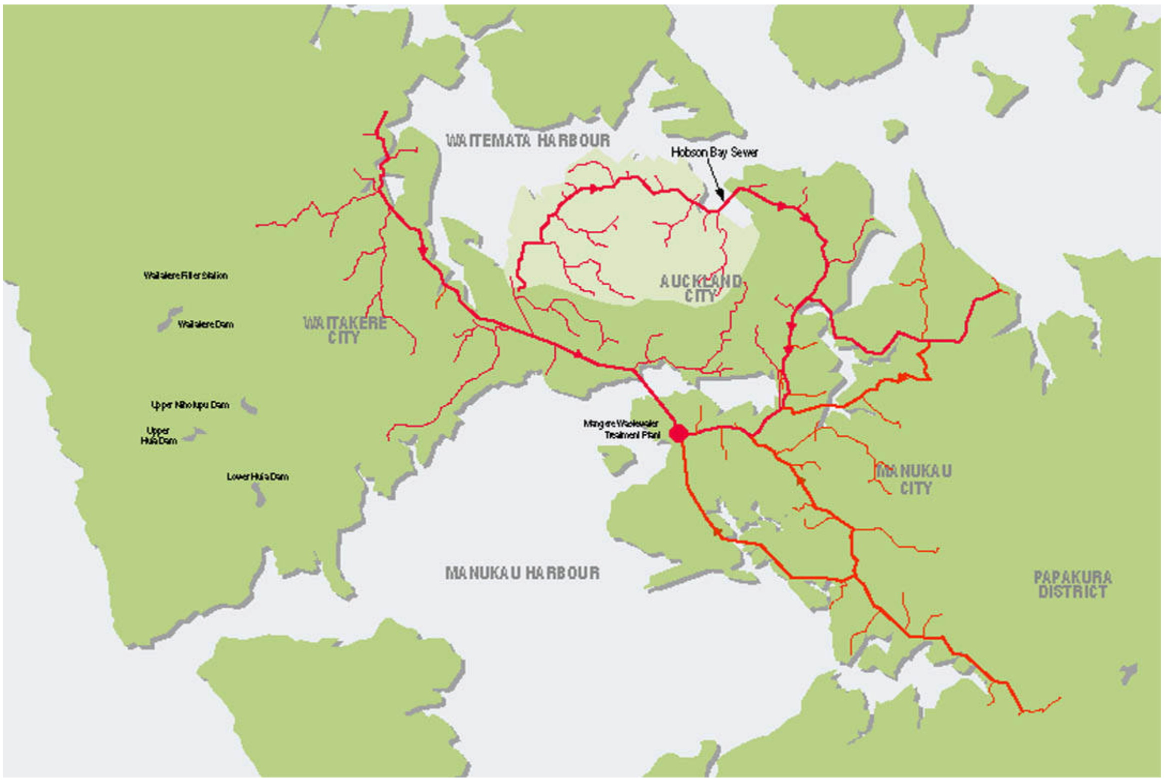
The Hobson Bay sewer is part of Watercare's Orakei main sewer; a combined wastewater and stormwater sewer, which serves a large part of the Auckland isthmus - from Avondale through Mt Albert, Three Kings, Pt Chevalier, Sandringham, Grey Lynn, Ponsonby, the central business district, Newmarket, Remuera, Ellerslie and Meadowbank. In total, the Orakei main sewer conveys about 25% of the region's wastewater en route to the Mangere Wastewater Treatment Plant.

Wastewater flows by gravity from Avondale to the Orakei pump station which lifts it to the Eastern Interceptor (EI). From here, the wastewater flows by gravity to the Mangere Wastewater Treatment Plant.

The existing Hobson Bay sewer is an above ground concrete pipeline that conveys wastewater 2 km across Hobson Bay from Logan Terrace in Parnell at the western upstream end of the sewer, to a tunnel at Ngapipi Road at the downstream eastern end. The pipeline is in poor condition and is reaching the end of its life.

Project Hobson is Watercare's project to replace the Hobson Bay sewer pipeline. Project Hobson involves the construction of a replacement sewer (tunnel) running from Logan Terrace under Hobson Bay and the Orakei ridgeline to a new pump station (PS64) in the Orakei Domain.

Figure 1: Wastewater Network and Catchments serviced by the Orakei main sewer



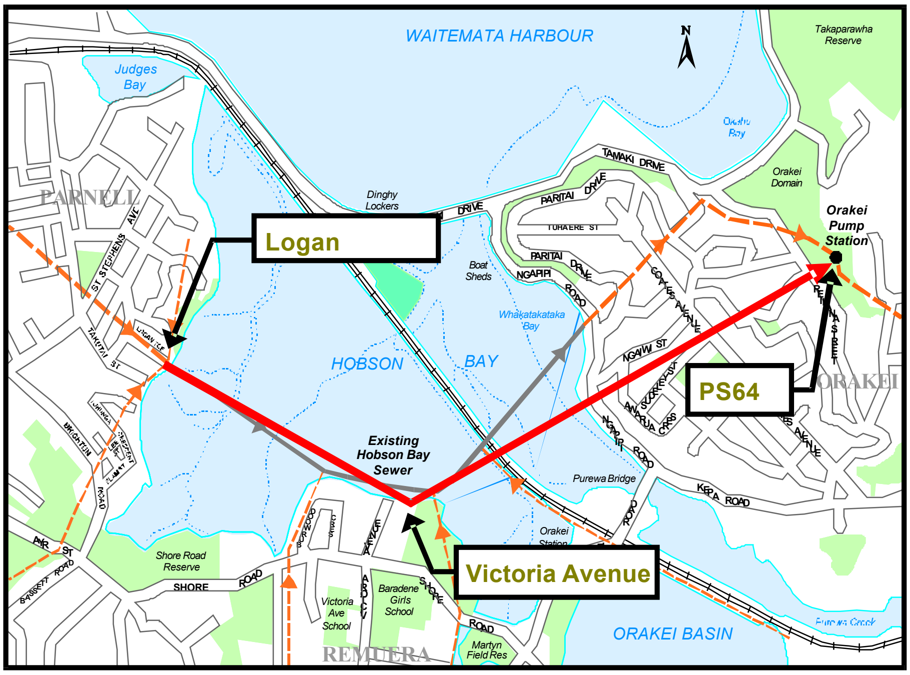
Watercare undertook an extensive consultation programme with the community to review and assess options for replacing the Hobson Bay sewer. A range of replacement options were considered, including an above ground pipeline and a buried pipeline. At the conclusion of this option analysis, Watercare determined that a tunnel offered the best mix of environmental, social and economic benefits and adopted it as the preferred replacement solution.

In addition to ensuring the continued performance of the Watercare network, the new Hobson sewer will provide storage to capture wet weather flows and practically eliminate overflows from Watercare's wastewater system that would otherwise flow into Hobson Bay. The new 3 km long, 3.7 m internal diameter sewer tunnel will connect from the Orakei main sewer at Logan Terrace to a new pumping station (PS64) in the Orakei Domain. The total cost of the project is NZ$120 million.

The tunnel is approximately 30 m below sea level and constructed by a tunnel boring machine. Flows from upstream sewers will be conveyed down to the tunnel through vortex drop shafts built in new reclamation areas. PS64 at the downstream end will deliver flows through twin rising mains to the existing trunk sewer or an overflow. The pumping station will include six vertically mounted dry well pumps with a peak design flow capacity of 6 m^3/s at 45m head.

Work started on construction in May 2007 and is due for commissioning in December 2009 followed by demolition of the existing sewer and pump station by August 2010.

Figure 2: Hobson Bay (alignment of new tunnel shown as red arrow)



This paper describes the options selection and design process for the wastewater system design, including tunnel sizing, hydraulic design, operation, ventilation design and sulphide modelling.

2 TUNNEL SIZING

The amount of storage required and therefore tunnel size is dependent on the level of service requirement, pumping station (PS64) maximum pumping rate, capacity of the EI and the upstream catchment characteristics.

2.1 LEVEL OF SERVICE

Watercare's requirement was that the storage provided needs to be of sufficient size to "virtually eliminate" overflows into Hobson Bay and reduce the frequency of overflows into Okahu Bay to one per year on average. The tunnel is also designed to ensure that it is practical to meet this objective to a design horizon of 2050, whilst not having significant excess storage capacity that may become redundant due to future changes to the upstream catchments.

In the case of storms with a higher than one year Average Recurrence Interval (ARI), with 2050 catchment, excess flows should be pumped from PS64 to overflow at Okahu Bay, through the existing overflow line.

During a prolonged power failure / pump breakdown the water level in the pump station wet well will rise until overflows from PS64 to Okahu Bay occur. If the water level continues to rise then overflows from Logan Terrace and Victoria Avenue to Hobson Bay may occur under wet weather flow conditions.

2.2 PS64 PUMPING RATE

The peak flow limit that can be pumped to the EI, under all flow conditions, is 3.7 m^3/s due to hydraulic constraints in the EI (see below). Under some conditions it may be possible to pump up to 4.5 m^3/s to the EI. Real time control of the pumping station may be beneficial in the future to optimize use of the storage in the tunnel.

The maximum capacity of the existing Okahu Bay overflow is calculated to be approximately 2.0 m^3/s and would normally be pumped from PS64.

The total pump capacity of PS64 has been set at 6.0 m3/s. This would allow simultaneous pumping of 4.0 m^3/s to the EI and 2.0 m^3/s to the Okahu Bay overflow or other combination of flow to a maximum total of 6.0 m^3/s.

2.3 CAPACITY OF EI

The maximum flow that can be passed to the EI in future will be dependent on the development of the EI catchment entering the EI downstream of PS64. Changes to the catchment may include the following:

- population and per capita flow increase
- catchment area increase
- infiltration reduction
- sewer augmentation increasing peak wet weather flows
- provision of storage to reduce peak wet weather flows.

Some of these factors will increase peak wet weather flows whilst other factors may reduce wet weather flows. The overall effect is impossible to predict accurately to the design horizon of year 2050 and the assumption made is that future infiltration reduction, storage and augmentation of the EI will be provided to allow the current maximum flow limit of 3.7 m^3/s to be maintained.

2.4 UPSTREAM CATCHMENT

The flows into the tunnel will depend on the development of the catchment upstream of the tunnel. The developments to the catchment are under review by Watercare and Metrowater and could result in significant changes to the wet weather flows. Therefore some assumptions on the upstream catchment were made to allow a tunnel storage volume to be finalized.

The most significant changes to the upstream catchment that may occur are the construction of another major sewer tunnel to Mangere Wastewater Treatment Plant, which would relieve pressure on the upstream end of the tunnel catchment, and partial separation of stormwater from the wastewater system. These changes would reduce peak flows through the Hobson Bay sewer countering the increase in flows coming from growth and as a result make upgrades of the sewers feeding the tunnel unlikely.

The capacity of the existing sewers just upstream of the tunnel govern the maximum flow that can reach the tunnel and in the modeling it is assumed that maximum use will be made of the upstream sewers and no restrictions are to be placed on their capacity. The tunnel size is therefore based on the existing sewer capacities without restrictions.

2.5 HYDRAULIC MODELLING

Modelling was undertaken using Watercare's existing InfoWorks model to identify an optimum size tunnel based on available emptying rates to the EI, the existing network capacity that feeds into the tunnel and typical dry periods between rainfall events.

This modelling work included:

- Identifying the time to empty the tunnel and any restrictions on tunnel size resulting from the time to empty being too great to allow it to be emptied for any following storm
- A statistical review of 10 years of rainfall records to understand the times between storms
- Time series modelling of 10 years rainfall (1995 to 2004 including 1999 which is an average year as determined by Metrowater's ICS Project) to determine the tunnel size required to limit the number of overflows from the tunnel to not more than 10 events in 10 years with the year 2050 population
- A sensitivity check to confirm the effect on the tunnel size by applying the year 2026 populations
- Modelling to determine the required pumping station overflows peak flow capacity (based on the capacity required to prevent overflows into Hobson Bay in a 10 year ARI event with the year 2026 population)

- Modelling discrete design storms (six hour, Event 39) with winter infiltration rates and year 2050 population to confirm peak flow rates.

In addition to the sizing of the tunnel, this modelling work also determined flows to be received at the tunnel drop shafts.

2.5.1 TIME TO EMPTY THE TUNNEL

Initial analysis was carried out on the time to empty the tunnel with varying diameters based on an emptying rate calculated as the difference between the pumped flow at PS64 of 3.7 m^3/s to the EI, and a typical flow after a storm. This analysis indicated that a tunnel within the consented range of 3 m to 4 m diameter should normally be emptied from full within 24 hours.

2.5.2 10 YEAR TIME SERIES MODELLING (1995 TO 2004)

The model was used to investigate the required storage volume based on 10 years of rainfall data from the Albert Park rain gauge site. This provided a real time representation of how the available storage volume was affected by subsequent storms and increased run-off from catchment wetness.

The model assumes that any flows that exceed the storage in the tunnel are pumped to the existing outfall at Okahu Bay. At this point the tunnel would be just full at Logan Terrace and surcharged by approximately 3 m at PS64.

A 4 m diameter tunnel was used in the model to provide approximately 33,000 m^3 of storage and any additional flow was effectively removed from the model. The following table provides a summary of the total storage volumes that would be required assuming no spill from the tunnel.

Table 1: Hobson Bay Model Results: 10 Year Time-series

Rank	Month	Tunnel Storage Required (m^3)	Spill Volume from Tunnel (m^3)	Total Volume[1] (m^3)	Start of Spill	End of Spill
1	Jul-98	33,000	56,322	89,322	16/07/1998 12:01	17/07/1998 09:06
2	Jul-98	33,000	19,443	52,443	14/07/1998 23:11	15/07/1998 07:56
3	Mar-03	33,000	18,391	51,391	11/03/2003 12:01	11/03/2003 21:16
4	Jun-00	33,000	15,734	48,734	29/06/2000 06:11	29/06/2000 11:01
5	Jul-98	33,000	13,170	46,170	09/07/1998 18:51	09/07/1998 23:36
6	Aug-98	33,000	11,949	44,949	11/08/1998 05:01	11/08/1998 09:36
7	Mar-95	33,000	6,859	39,859	29/03/1995 23:51	30/03/1995 02:31
8	Jun-96	33,000	2,187	35,187	23/06/1996 15:06	23/06/1996 17:56
9	Jul-00	33,000	1,995	34,995	02/07/2000 16:56	02/07/2000 19:41
10	Feb-04	29,492	0	29,492	-	-
11	Jul-96	26,279	0	26,279	-	-

Tunnel overflows within 24 hours of each other were considered to be the same event as it would effectively take 24 hours for the tunnel to drain down from the initial overflow and therefore any additional spills would be due to the increased water level in the tunnel and not necessarily subsequent rainfall events.

If it is assumed that the tunnel is not to spill more than 10 times in the last 10 years then the 11th greatest storage volume must be stored in the tunnel. The storage volume required for this storm is 26,279 m^3, which with an

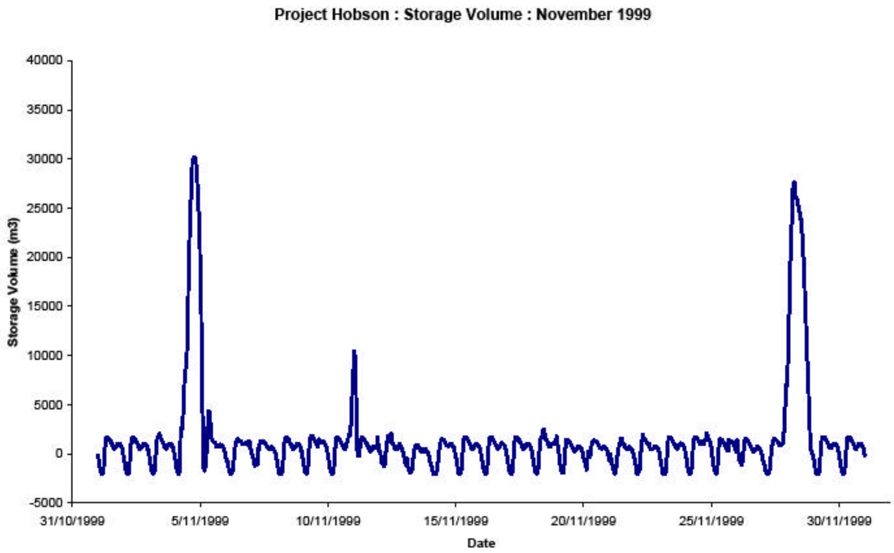
[1] Storage volumes in addition to the average dry weather flow of approximately 2,220 l/s that occupies approximately 5,000 m^3 of storage in the tunnel.

average dry weather flow volume of 5,000 m^3 already taken up in the tunnel as pass through flow, would equate to a tunnel diameter of 3.63 m. A tunnel diameter of 3.7 m was therefore selected.

Figure 3 below shows how the tunnel will fill and empty during a typical period in 1999.

Figure 3: Storage Volumes in Tunnel with November 1999 Rainfall at year 2050

(Winter Infiltration Rate assumed)



3 TUNNEL OVERFLOW DESIGN

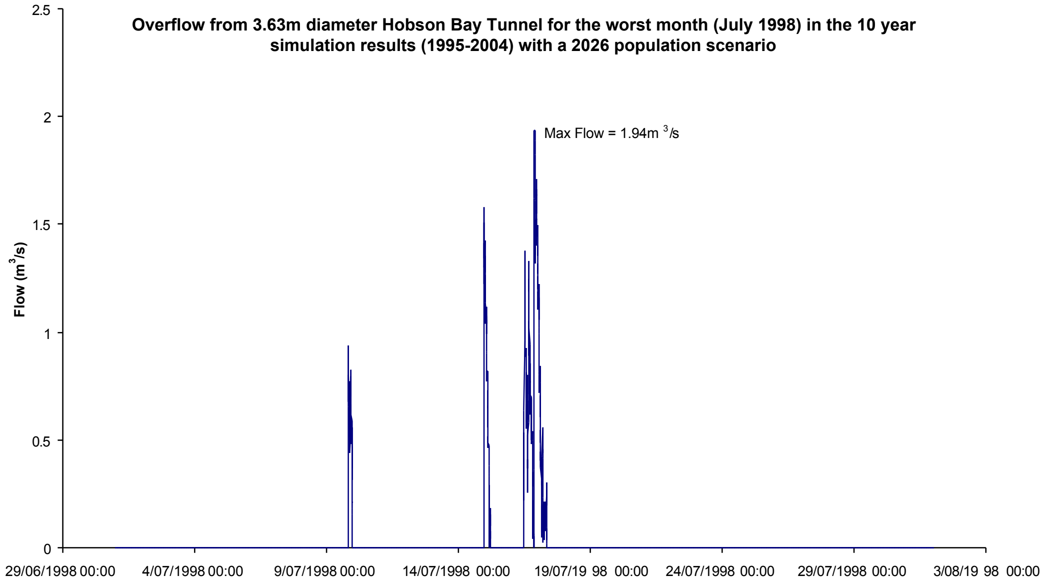
Overflows from the tunnel are provided at PS64, Victoria Avenue and Logan Terrace. Overflow levels are set so that the PS64 overflow (to Okahu Bay) will operate before the Victoria Avenue and Logan Terrace overflows.

In normal operation flows entering PS64 in excess of the capacity of the EI, and once the tunnel storage is full, are pumped from the wet well to the overflow. In the case of prolonged pump station / power failure the water level in the wet well rises until a weir in the wet well is overtopped and flows to the overflow.

The overflow weir from PS64 discharges into the existing overflow sewer to Okahu Bay. The existing overflow sewer has limited capacity of approximately 2 m^3/s and would need to be upgraded to pass the peak overflow in 2050. However a review of the modelling carried for the 2026 population showed that the existing overflow sewer would have sufficient capacity until 2026.

The results are given in Figure 4 below.

Figure 4: Overflows to Okahu Bay Overflow



The acceptance of a shorter time horizon for the overflow sewer is acceptable because:-

- Watercare's "Central Interceptor sewer" is likely to be constructed by 2026 and may reduce the overflow capacity required to Okahu Bay
- if the "Central Interceptor sewer" is not in place by 2026 then the requirement to construct a new overflow line from PS64 to Okahu Bay can be reassessed.

The overflow weir level at PS64 is set above the hydraulic level required to discharge pumped overflows with a 10 year ARI (July 1998 storm at 2026 population) to Okahu Bay and hence there will be no overflow at Victoria Avenue or Logan Terrace under this condition.

Under prolonged pump failure conditions and high flow conditions the PS64 overflow has insufficient capacity to take all flows entering the tunnel. The operation of the overflows under a prolonged pump failure with the 100 year ARI six hour storm peak flows from Project Hobson 2003 Model (with 2026 population) was modelled. The modelling demonstrated that in the first instance overflows will occur at Okahu Bay and that if the event continues controlled discharged will occur from the Logan Terrace and Victoria Avenue overflows.

4 TUNNEL HYDRAULIC ARRANGEMENT

In parallel with the selection of the tunnel vertical and horizontal alignment (which is not covered in this paper) the following alternative hydraulic system options were considered:

A. Single carrier - conventional sewer tunnel at a constant diameter and grade.

B. Tunnel Storage at the downstream end – Construction of a larger storage tunnel under the orakei ridge where tunnelling conditions are likely to be better.

C. Provision of separate dry weather flow (DWF) pipe in the tunnel in the form of as a siphon.

D. Provision of separate dry weather flow (DWF) pipe in the tunnel in the form of a long pump suction.

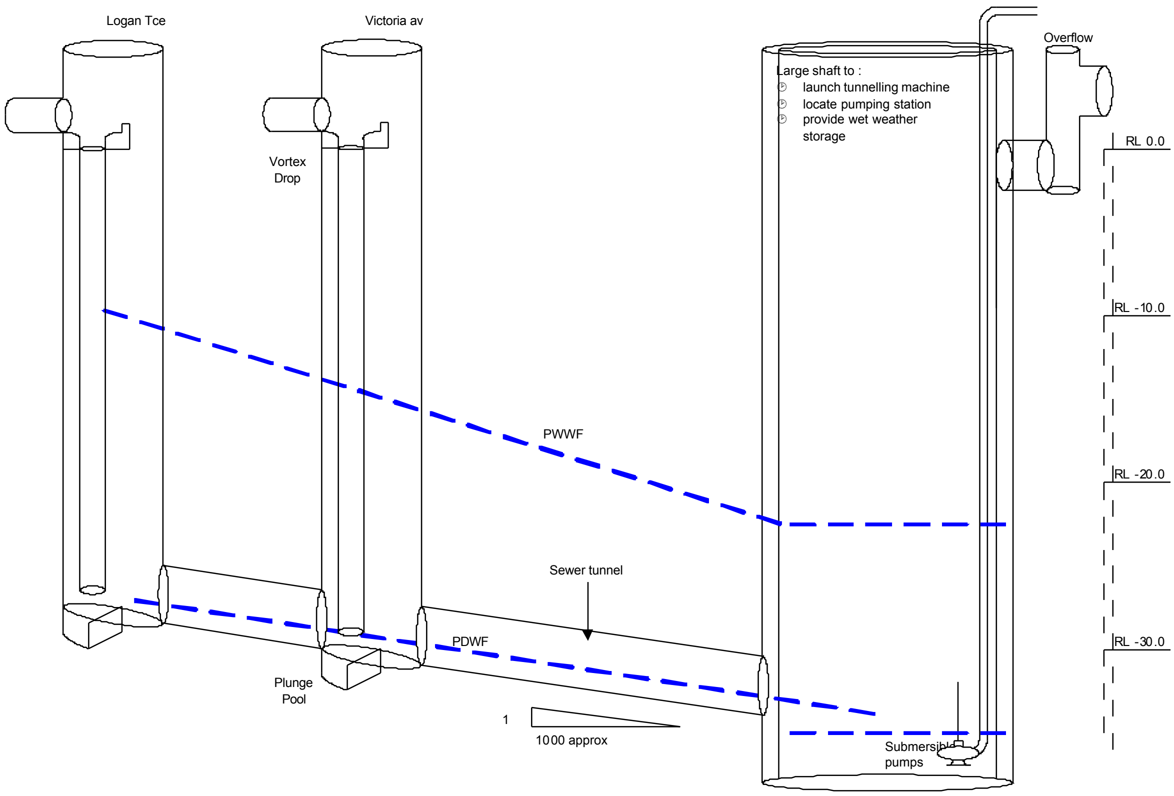
E. Provision of separate dry weather flow (DWF) pipe in the tunnel in the form of a pumping main – Pumping from the upstream end.

4.1 SYSTEM OPTION A - SINGLE CARRIER

With the single carrier option flows drop into the tunnel at Logan Terrace, Victoria Avenue and PS64 through vortex drop shafts. A pumping station is provided at the downstream end to raise the flow from dual sumps at PS64.

The tunnel diameter would be constant 3.7 m diameter. The gradient of the tunnel would meet the requirements to promote grit transport and to reduce the quantity of sulphate reducing slimes on sewer walls.

Figure 5: Single Carrier Arrangement



4.2 SYSTEM OPTION B - TUNNEL STORAGE AT THE DOWNSTREAM END

This hydraulic system option is the same as the single carrier except that the tunnel under Hobson Bay is reduced in size to about 2.4 m diameter to allow it to be constructed as a pipejack. The actual size chosen would be the minimum diameter required to achieve the hydraulic flow requirements under peak flow overflow conditions. The loss of tunnel volume for storage of wet weather flows resulting from the reduction in tunnel size would be made up by increasing the tunnel size at the downstream end under the Orakei ridge to about 4.2 m diameter. Vents would be required where air could be trapped above rising water levels.

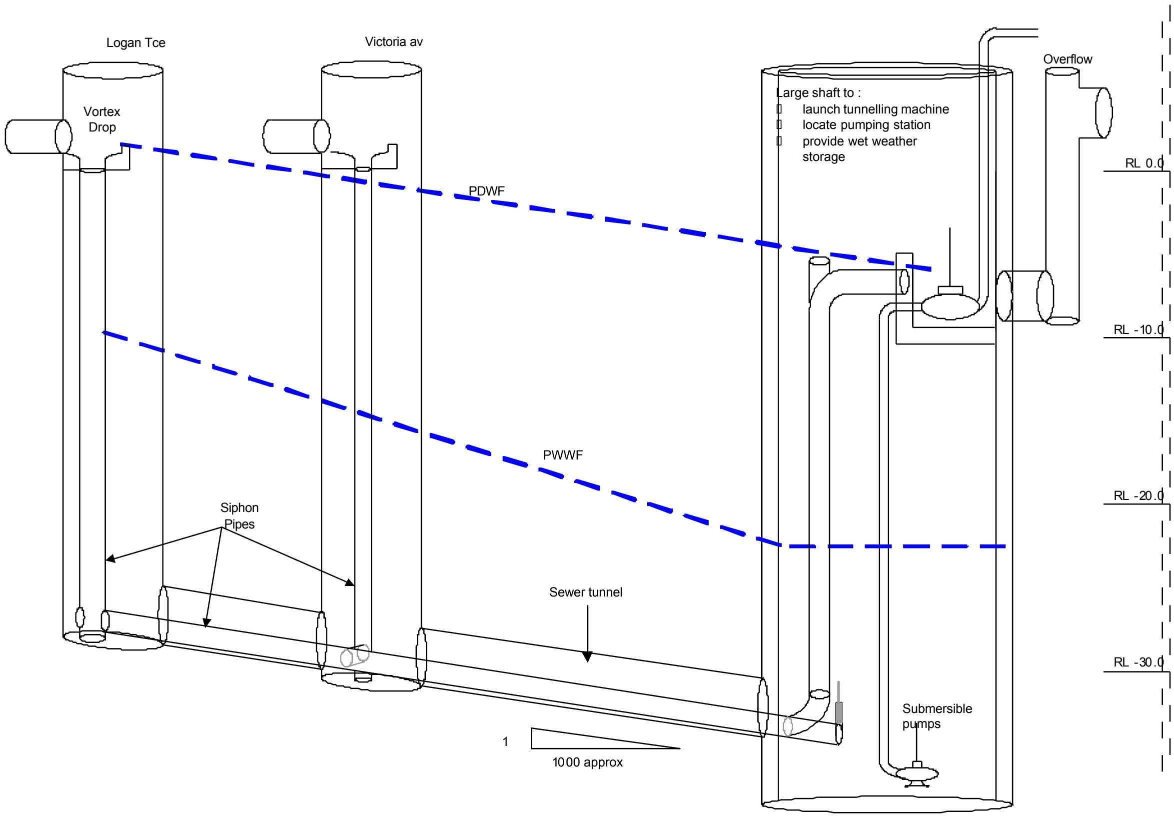
4.3 SYSTEM OPTION C - SEPARATE DWF PIPE AS A SIPHON

A pipe of approximately 1.2 m diameter would be installed in the tunnel to carry dry weather flows. The pipe would work as an inverted siphon receiving flows at a high level at Logan Terrace and Victoria Avenue and discharging to an intermediate level pumping station wet well at PS64.

Flows in excess of the capacity of the siphon would overflow at the Logan Terrace and Victoria Avenue shafts into the main tunnel and would be conveyed along the tunnel to a low level pumping station at the bottom of the PS64 shaft.

The siphon pipe size would be set to maintain a minimum velocity of approximately 0.5 m/s to minimise the build up of any grit at low flows. A valve would be provided on the lowest point of the siphon pipe to allow periodic flushing. Air valves may be required to remove entrapped air.

Figure 6: Siphon Arrangement



The main tunnel would be approximately 3.9 m diameter to provide the require storage and would be constructed at a sufficient gradient to ensure that grit or sediment is scoured out after any build up during storm events.

4.4 SYSTEM OPTION D - SEPARATE DWF PIPE AS A LONG SUCTION

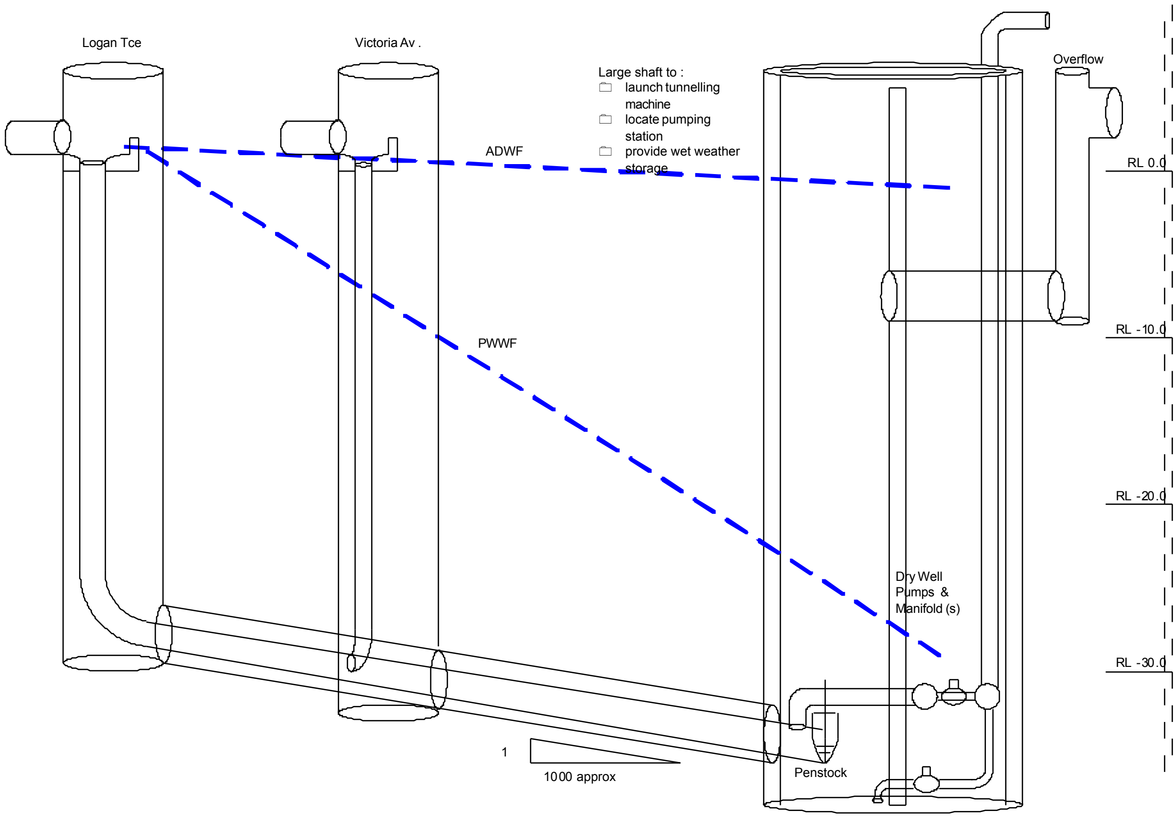
A pipe of approximately 1.2 m diameter would be installed in the tunnel to carry all dry weather flows. The pipe would receive flows at a high level at Logan Terrace and Victoria Avenue and would operate as a pump suction. Dry well pumps installed at tunnel level in a separate shaft or chamber would draw the flow through the pipe and discharge it into the rising main at PS64. A schematic of the arrangement is shown in the figure below.

Flows in excess of the capacity of the pumped suction would overflow at the Logan Terrace and Victoria Avenue Shafts into the main tunnel and would be conveyed along the tunnel to separate low level pumping station at the bottom of the DPS64 shaft.

The suction pipe size would be set to maintain a minimum velocity of about 0.5 m/s to minimise the build up of any grit at low flows. A valve would be provided on lowest point of the suction pipe to allow any build up of grit to be flushed out regularly. Air valves may be required to remove entrapped air.

The dry well suction pumps would either be controlled on pressure in the suction main close to the pumps or on water level at the upstream end.

Figure 7: Long Suction Main Arrangement



The main tunnel would be approximately 3.9 m diameter to provide the required storage and would be constructed at a sufficient gradient to ensure that grit or sediment is scoured out after any build up during storm events.

4.5 SYSTEM OPTION E - SEPARATE DWF PIPE AS A PUMPING MAIN

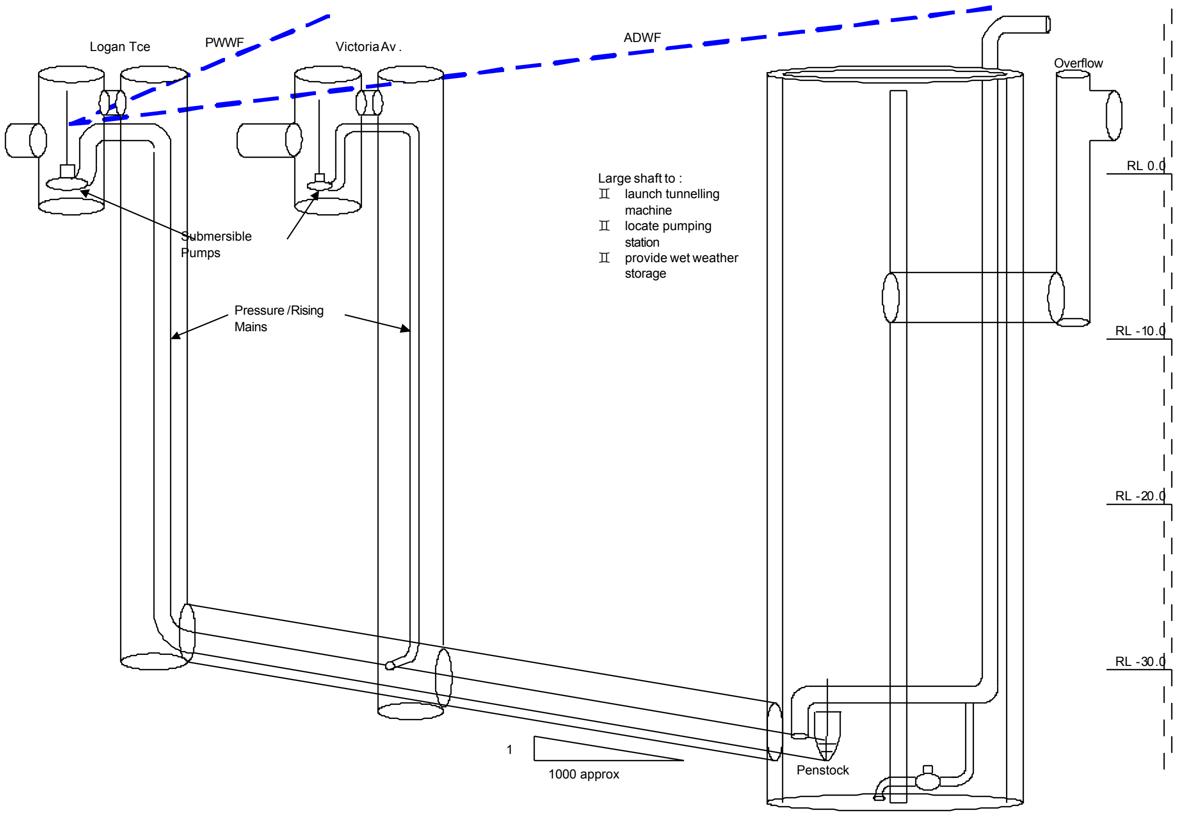
A pipe of approximately 1 m diameter would be installed in the tunnel to carry all dry weather flows. The pipe would receive flows from pumping stations at a high level at Logan Terrace and Victoria Avenue. The rising main would discharge the flow into the PS64 rising main. A schematic of the arrangement is shown in below.

Flows in excess of the capacity of the Logan Terrace and Victoria Avenue pumping stations suction would overflow into the tunnel shafts and along the main tunnel to low level pumping station at the bottom of the PS64 shaft.

The pumping main pipe size would be set to maintain a minimum velocity of about 0.5 m/s to minimise the build up of any grit at low flows. A valve would be provided on lowest point of the pipe to allow any build up of grit to be flushed out.

The main tunnel would be approximately 3.9 m diameter to provide the require storage and would be constructed at a gradient of about 1:1000 such that grit or sediment is scoured out after any build up during storm events.

Figure 8: Pressure Main Arrangement



4.6 ASSESSMENT OF HYDRAULIC SYSTEM OPTIONS

4.6.1 SYSTEM DESIGN REQUIREMENTS

The design of the system must accommodate two primary objectives:

- Satisfactory dry weather performance (especially in respect of solids transport, sulphide generation, odour and corrosion).
- Satisfactory wet weather performance (especially in respect of the transfer and storage of the maximum volume of wet weather flows).

In dry weather, maximising solids transport and minimising sulphide generation are best achieved in part-full flow conditions. The hydraulic design of sewers for part-full, dry weather flows mainly relates to the selection of an optimum gradient. As the Orakei main sewer serves a combined system with a potentially significant grit load, the gradient should be relatively steep to reduce the potential for sedimentation.

Dry weather odour and corrosion are reduced through (amongst other things) the control of turbulence. Under part-full flow conditions, the detailed design of drops, junctions and changes of cross-section is appropriate, but unlikely to change fundamentals such as diameter and gradient.

An assessment of the hydraulic performance of the alternative options is given in the table below.

Table 2: Assessment of Hydraulic Performance

Option	Hydraulic Performance
Single Carrier	Similar to a traditional sewer as follows: • part-full flow, approximately 20% full in dry weather • average velocity approximately 1.4m/s • Froude No approximately 0.8 (ie. less than 1.0) • wall shear exceeding 3.35Pa
Tunnel Storage at Downstream end	**Similar to the above but, due to increased diameter and reduced hydraulic efficiency, the storage section of the tunnel should be steepened to improve solids transport.**
Separate DWF Pipe as a siphon	Siphon entry will operate as a free overfall and generate turbulence and odour at flow rates other than optimum (ie. most of the time). Siphon exit will also generate odour. Flows slightly in excess of siphon capacity will trickle into the tunnel. Overnight, low flows will slow to approximately 0.5 metres/sec. Flushing of the siphon will be required from time to time.
Separate DWF Pipe as a long suction main	If upstream level control was implemented, turbulence at one of the two main pipe entries could be controlled, however turbulence and odour would occur at the other entry. A smaller dry weather flow pipe flowing at higher velocity (than the siphon option) could be utilised but the saving in head is reduced.
Separate DWF Pipe as a pumping main	Operates at higher head than siphon or long suction options. A smaller dry weather flow pipe flowing at higher velocity (than the siphon option) could be utilised but the saving in head is reduced.

4.6.2 HYDRAULIC SYSTEM OPTION COMPARISON AND SELECTION

A comparison of the alternatives is given in the table below:

Table 3: Comparison of alternative system options

Option	Advantages	Disadvantages
Single Conduit	▪ accepted practice, well understood ▪ simple configuration ▪ minimum reduction of sulphates to sulphides ▪ low cost	▪ head loss results in additional long term power cost ▪ no dry access to the tunnel
Single conduit with tunnel Storage at Downstream end	▪ as above + ▪ more flexible storage design ▪ lowest cost	▪ as above + ▪ additional ventilation required ▪ Reduced hydraulic capacity to minimise overflows at LT and VA
Separate DWF Pipe as a siphon	▪ dry access possible ▪ traditional siphon ▪ power cost saving (NPV of saving approximately $3 million for deep option)	▪ increase risk of odour at inlets ▪ increase in sulphides downstream ▪ Additional cost of increase in the diameter of the tunnel ▪ Additional cost for the pipe in tunnel ▪ Additional cost for separate pumping systems for DWF and the tunnel

Option	Advantages	Disadvantages
Separate DWF Pipe as a long suction main	▪ dry access possible ▪ possible upstream level control with no drop at inlet (cannot apply at both Victoria & Logan). ▪ increased DWF capacity (cf siphon) ▪ Power cost saving (NPV of saving approximately $3 million for deep option)	▪ increased sulphide ▪ increase in the diameter of the excavation ▪ Additional cost for the pipe in tunnel ▪ Additional cost for separate pumping systems for DWF and the tunnel ▪ Additional complexity of control of pumps from a remote point upstream ▪ Innovative but higher risk design
Separate DWF Pipe as a pumping main	▪ dry access possible ▪ possible upstream level control with no drop at inlet ▪ increased DWF capacity (cf siphon)	▪ as above ▪ upstream pumping stations (not consented ▪ upstream pumping stations – impact on local residents

After extensive assessment of whole life costs of the alternatives the single carrier option (Option A) was selected along with the shallower tunnel alignment to minimize the depth of the pumping station and pumping costs.

4.7 SEWER AND TUNNEL GRADIENT

The gradient of the tunnel and branch sewer connections was selected using the following design criteria to ensure sediment is scoured out and that excess velocities and turbulence are avoided:

- gradient of the sewer should be steeper than 1 in D (where D is the diameter in millimetres)
- gradient of the sewer should be flatter than 1 in 0.25D
- froude number should be less than 1 during dry weather flows
- average wall shear should equal or exceed 3.35pa at PDWF
- maximum velocity should not exceed 3.0 m/s.

On the basis of these parameters the tunnel gradient selected is 1:1000.

5 VORTEX DROPSHAFTS

The flow from the network connections to the tunnel at Logan Terrace, Victoria Avenue and at PS64 drops almost 30 m down into the tunnel. Vortex drop shafts have been included in the design to reduce erosion, odour and noise by controlling the flow within a vortex drop pipe. The design of the vortex drop shafts is based on the Archimedean Spiral design which has been applied in Melbourne by Melbourne Water for many years. These designs have been the subject of a program of model testing including recent modelling of very similar flows and drop heights for the Northern Sewerage Project for SKM.

The vortex drop shafts at Logan Terrace, Victoria Avenue and PS64 are sized to ensure that at minimum flows a vortex will be formed (as far as practically possible) and that at high flows the drop shafts do not become a restriction in the system and do not lead to surcharging in the upstream sewers and overflows before the tunnel storage is full. It is practical to meet this by designing the vortex drop shaft to pass flows up to a one year ARI in 2050 without surcharging the upstream sewers. Maximum and minimum flows are given below for each of the drop shafts

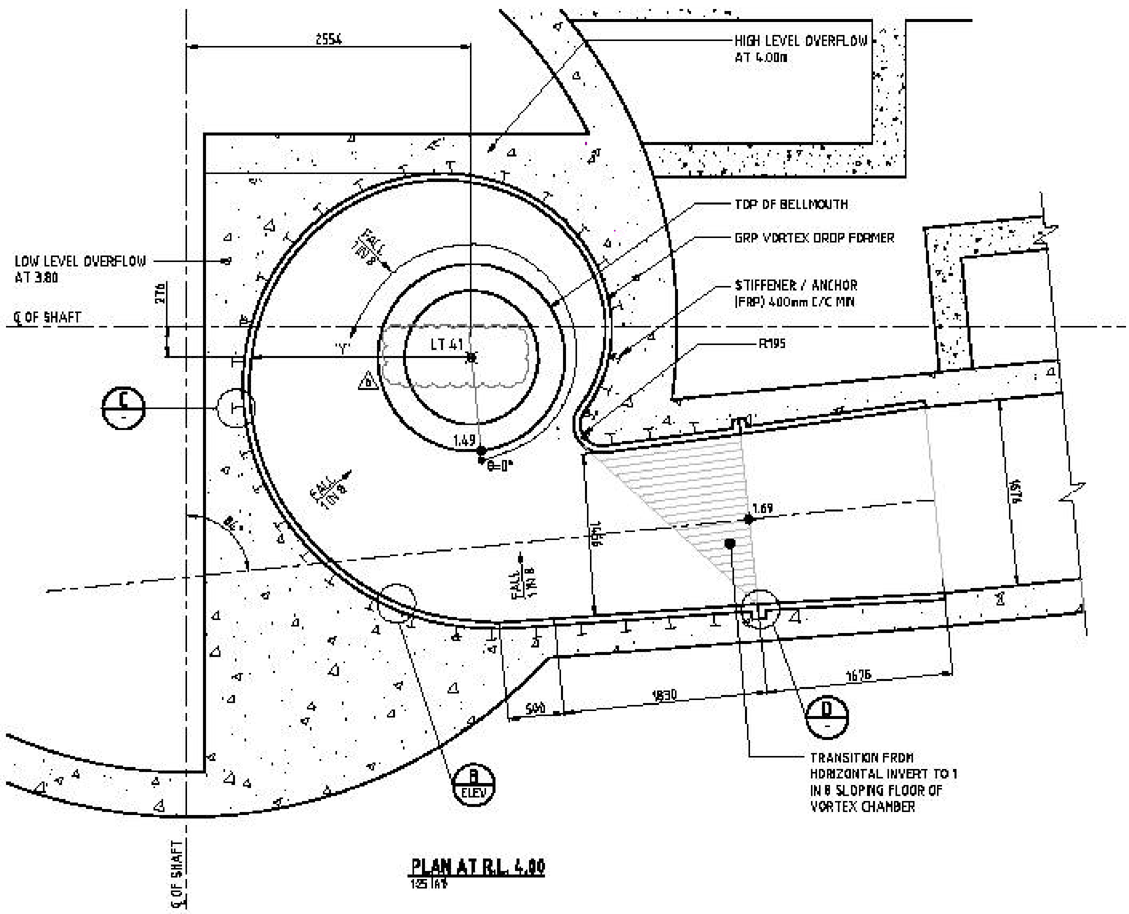
Table 4: Drop Shaft Flows (Year 2050 Population)

	Logan Terrace (m3/s)	Victoria Avenue (m3/s)	PS64[2] (m3/s)
Peak Flow	5.03	1.47	5.87
Minimum Flow	0.20	0.03	0.23

The drop shafts at Logan Terrace and Victoria Avenue will also be provided with an overflow to the access shafts to allow flows in excess of this flow to enter the tunnel. The overflows would also operate if the vortex became blocked. The capacity of the drop shaft overflows to the tunnel has been design for the 100 year ARI peak flows from Project Hobson 2003 Model (with 2050 population).

Details of the vortex arrangement selected are given in Figure 9 below.

Figure 9: Logan Terrace Vortex Shaft Design



6 TUNNEL OPERATION

The pumping station is a dry well single stage pump station pumping directly to the EI. The pump station has two sets of three pumps each pumping through separate manifolds and rising mains to the EI i.e. there are twin

[2] Peak flows from the branch sewers discharging to the tunnel may not coincide and therefore the peak flow in the tunnel may not necessarily be the sum of the other inflows.

parallel manifolds and rising mains from the pump station to the EI connection chamber. Each set of pumps includes one with a variable speed drive that allows the flow to the EI to be varied to suit its capacity. Overflows are pumped to the existing overflow pipeline from the manifolds. Overflows will be controlled by the operation of a control valve on the manifold.

The total capacity of the new pumping station is 6.0 m^3/s. Anticipated flow conditions are summarised in the table below:

Table 5: PS64 Flow Summary

Description	Flow
Absolute Minimum Flow	0.25m3/s to EI (@45m head)
Normal Minimum Flow	0.5m3/s to EI (@45m head)
Peak Wet Weather Flow - no overflow	3.7 to 4.5m3/s to EI (@44 to 48m head)
Peak Wet Weather Flow – with overflow	3.7 to 4.5m3/s to EI (@44 to 48m head) + up to 2m3/s to Overflow (@44 to 48m head)

A simplified flow schematic diagram and section are given in Figure 10 and Figure 11.

Figure 10: PS64 Flow Schematic Diagram

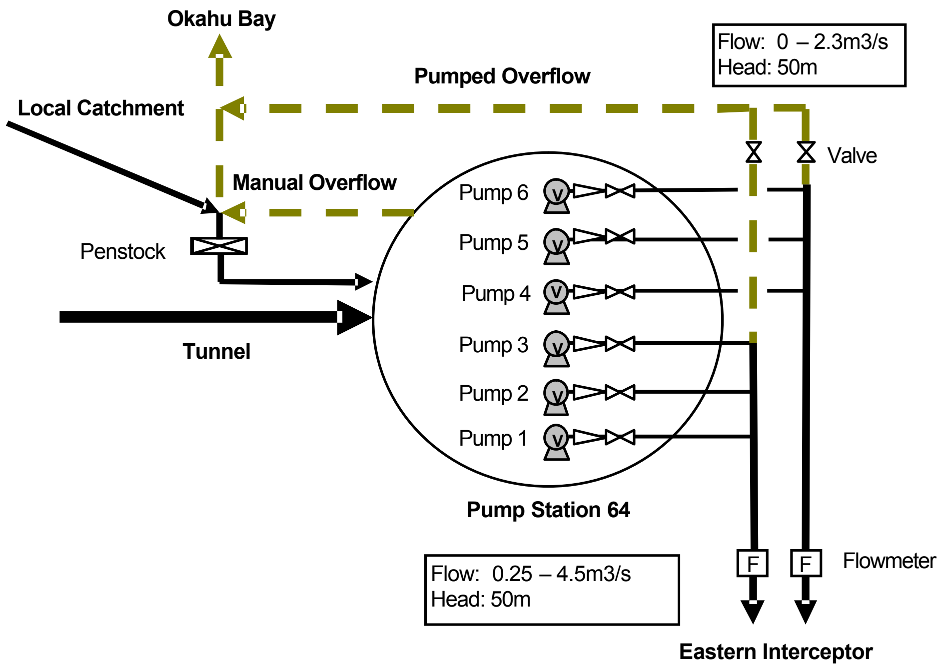
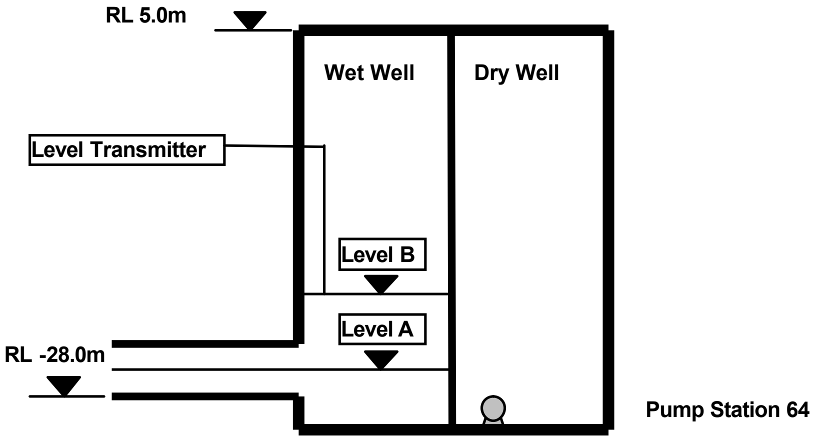


Figure 11: Pumping Station 64 Section



To deliver the total required flow, the capacity for each pump is 1 m^3/s with a head of 44 to 48 m. The control philosophy for a typical operating cycle is summarised below:

Table 6: PS64 Flow and Storage Summary

Flow & Storage Status		Basis of Pump Control
Flow into tunnel	**Storage status**	
Less than 3.7* m^3/s	Full storage available	Water level in Pumping Station 64 wet well maintained at Level A. Control valve to overflow is shut. Isolation valve from local catchment is open.
Greater than 3.7* m^3/s	Storage starts to fill	Pumping Station 64 pumping at maximum flow to EI. Control valve to overflow is shut. Isolation valve from local catchment is shut before Level B is reached to store local flows in the overflow line***.
Greater than 3.7* m^3/s	Storage full to Level B (overflow occurs from PS64)	Pumping Station 64 pumping at maximum flow to EI and at minimum flow to Overflow to maintain Level B in wet well. Control valve to overflow is open (modulating). *** Isolation valve from local catchment is shut.
Greater than 6.0** m^3/s	Storage filling above Level B (overflow occurs from PS64 and may occur from upstream at the tunnel dropshafts)	Pumping Station 64 pumping at maximum flow to EI and at maximum flow to Overflow. Control valve to overflow is fully open Isolation valve from local catchment is shut.
Less than 6.0** m^3/s	Storage emptying to Level B	Pumping Station 64 pumping at maximum flow to EI and at maximum flow to Overflow. Control valve to overflow is fully open. Isolation valve from local catchment is shut.
Less than 6.0* *m^3/s	Storage emptying below Level B (overflow occurs from PS64)	Pumping Station 64 pumping at maximum flow to EI and at minimum flow to Overflow to achieve Level A in wet well. Control valve to overflow is open (modulating). Isolation valve from local catchment is shut.

- *Maximum flow to the EI varies between 3.7 and 4.5 m^3/s
- **Maximum flow to the EI and overflow taken to be 6.0 m^3/s

- *** Storage of the Orakei and Branch 1a / 1d inflows in the inlet pipe before opening the control valve to the overflow means that the overflow valve will open against a depth of sewage and avoid cavitation in the pipe.

For normal flows to the EI four / five pumps will be required and there will therefore be one / two standby pumps. For extreme flows when pumping to the overflow is needed all six pumps may need to operate simultaneously.

In the case that one pump is out of service during an extreme event, it is intended that the remaining pumps can be operated slightly overspeed using the VSD's.

6.1 VENTILATION SYSTEM

The primary role of the ventilation system is to ensure the new sewer is not the cause of odour complaints. Ventilation is not intended as the primary means of controlling corrosion.

The strategy involves the operation of a fan at the furthest downstream shaft (Pump Station PS64 Wet Well) pulling air down the Hobson Bay sewer. The extracted air is likely to be odorous and will be processed through a bark biofilter before venting to atmosphere.

6.1.1 DESIGN CRITERIA

The design criteria's for the ventilation system are:

- Establish a negative pressure of 60Pa[3] below atmospheric at the most upstream sewer shaft (i.e. Logan Terrace)
- Extraction rate that is greater than the volume of displaced air due to diurnal variation during normal dry weather conditions (excluding wet weather events where odour is not an issue)
- Air velocities greater to or equal to that generated by sewage drag, so the fan is capable of extracting out the air volume that is naturally dragged down the sewer.

When the tunnel fills during storage events there is no air path through the tunnel and the ventilation system at PS64 cannot draw air though the tunnel. Under this situation negative pressure will not be maintained and at the upstream shafts air may escape from the sewer. The risk of unacceptable odour emissions is however very low because the tunnel will only normally fill in wet weather when the sewage will be dilute and have very low sulphide levels. However, the provision for future installations of local odour treatment facilities at Logan Terrace and Victoria Avenue has been incorporated into the design of the shafts sites. Odour control, if required, at the shafts will be compact activated carbon or chemical scrubbing units which can be turned on and be effective immediately during a short term tunnel full event.

6.1.2 AIR FLOW MODELLING BASIS

An airflow "Ventilation" model was developed for the Hobson Bay sewer tunnel and the connecting branch sewers. This helped to predict where potential air may enter the system in the future and the vacuum that may be created in the sewer by using different air extraction rates at the odour control facility at PS64.

This model was developed based on assumptions of frictional losses due to bends and openings. There is the added complexity with the overflows at Logan Terrace and Victoria Avenue, which are expected to be the major source of air input into the system. These overflows have a flap gate which opens in one direction to

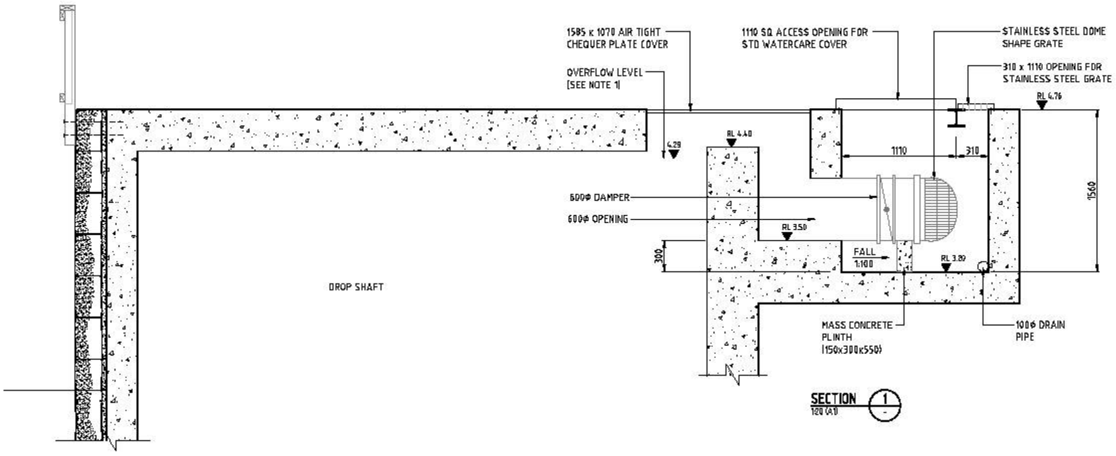
[3] The ventilation design is based on steady state conditions. It does not account for pressure fluctuations that will arise due to variation in atmospheric pressure over the distances of the project, air buoyancy created at times when the sewer air is warmer and more moist than surrounding atmosphere and any suction effect arising from wind blowing across vent stacks and diurnal rise and fall in the sewage levels. These effects are highly variable and difficult to calculate, however, our best estimate is that they could cause around 50 Pa opposing the direction we are trying to ventilate. Hence, the design vacuum for the system needs to be comfortably above this to prevent odorous air escaping under most likely environmental conditions.

allow water to flow out. There is no measured information on how air tight these gates will be, so assumptions have been made to estimate the amount of air that will "leak" into system.

During times of high tides, water levels will cover the mouth of the overflow pipe, effectively creating an air seal. Under this operating condition, insufficient air will be entering the system to create effective ventilation.

Therefore an engineering design solution is to include *"air induct"* points at Logan Terrace shaft and Victoria Avenue shaft, in order to introduce enough fresh air into the system. **Error! Reference source not found.** Figure 12 shows the configuration of these air inducts in relation to the over flow, such that there is air inflow regardless of high or low tide. The air inducts are provided with a damper so that air inflows can be adjusted to generate the required negative pressure at the dropshafts.

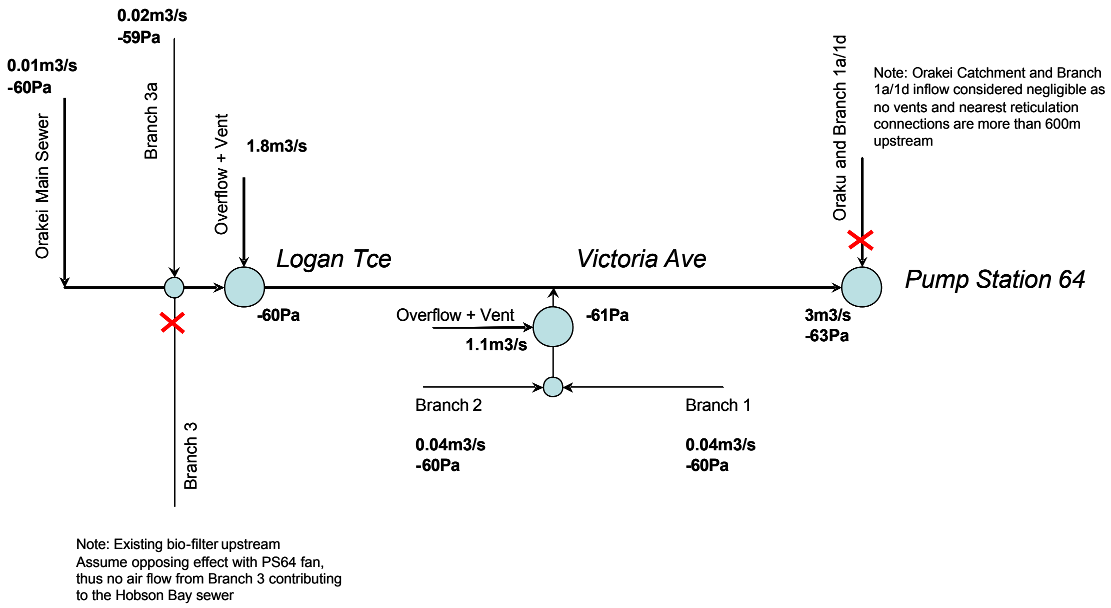
Figure 12: Air induct for Logan Terrace Dropshaft



6.1.3 DESIGN VENTILATION SCENARIO DURING NORMAL OPERATION

Figure 13**Error! Reference source not found.** describes the design air flow rates and pressure in the sewer arrived at through the "Ventilation" model. This requires a fan extraction rate of *3 m^3/s at PS64*, in order to satisfy the design criteria.

Figure 13: Design Ventilation Flow in Hobson Bay Sewer



The model suggests very little pressure loss along the main sewer, primarily because it is large at 3.7 m diameter with a large air space during dry weather flows.

6.2 VENTILATION DESIGN CONCLUSION

The flexible approach to the ventilation design has avoided the need for odour control treatment at the upstream shaft sites which would have been difficult to access for maintenance and cause a potential nuisance to local residents. The odour treatment and ventilation fans are located at PS64 which is a less sensitive site and can be easily maintained.

7 SULPHIDE GENERATION AND CORROSION

Sulphide generation occurs in sewers where there are anaerobic conditions and required nutrients are present. Hydrogen sulphide and other odourous compounds will be released into the gas phase and are the cause of most odour complaints relating to sewerage assets. In addition, hydrogen sulphide can be absorbed onto wet surfaces leading to the formation of sulphuric acid which will corrode concrete and most metals. The degree of sulphide generation is dependant on a variety of parameters including flow rate, sewage velocity, temperature, BOD, pH, sewer diameter etc.

7.1 LIQUID SULPHIDE CONCENTRATION

Modelling was carried out on the sewer, to estimate the generation of sulphides in the liquid phase and expected gas sulphide concentration in the sewer atmosphere. The sulphide model was calibrated to sets of data collected in winter and summer.

Greater sulphide generation will probably occur in the earlier years after commissioning as lower flows will mean longer sewage detention time. The maximum predicted liquid sulphide level is 1.3 mg/l and, overall, the liquid sulphide generation is expected to be less than that generated in the existing Orakei main sewer. The reason for this is primarily the effect of shorter detention times, due to higher flow velocities, leading to less sulphide generation. The predicted liquid sulphide concentrations account for the effects of sulphide generation in the sewage as it travels down the system, and also estimates the amount of sulphide stripped out into the gas phase.

If future flows decrease because of reduced infiltration, the sewage will become more concentrated in sulphates (the precursor to sulphides) and BOD/COD, then greater sulphide generation is anticipated.

7.2 CORROSION RATES

From correlations between corrosion rates and liquid sulphide levels based on historical data, anticipated corrosion rates of between 0.27 mm/yr to 0.48 mm/yr over the life time of the sewer were estimated caused by the following mechanisms:

- Leaching
- H_2S gaseous attack
- Biological attack on concrete caused by generation of acid in presence of bacteria
- Sulphate attack on concrete caused by the availability of sulphate ions in its pore solution.

These are the main mechanisms for deterioration of concrete above the water line, exposed to water condensation / high humidity and H_2S gas. Deterioration of concrete below the water line is of course possible and has been observed. The extent of concrete deterioration below the water line is dependent on the make up of the wastewater, particularly the organic acid content. In normal circumstances, for domestic wastewater, the deterioration of concrete below the water line is much less than above the water line because biological attack on concrete cannot occur when submerged since O_2 in the solution is limited.

The corrosion rates presented here apply for concrete sewers where the walls are damp for the majority of the time (as is expected to be the case for the new Hobson Bay sewer).

However this correlation cannot be applied to sewers with vortex drops, where excessive sulphide stripping may enhance the corrosion rate locally at the shaft. Stagnant pockets of air thus resulting in the accumulation of sulphide gas may be an issue particularly if there is insufficient ventilation bringing in fresh air.

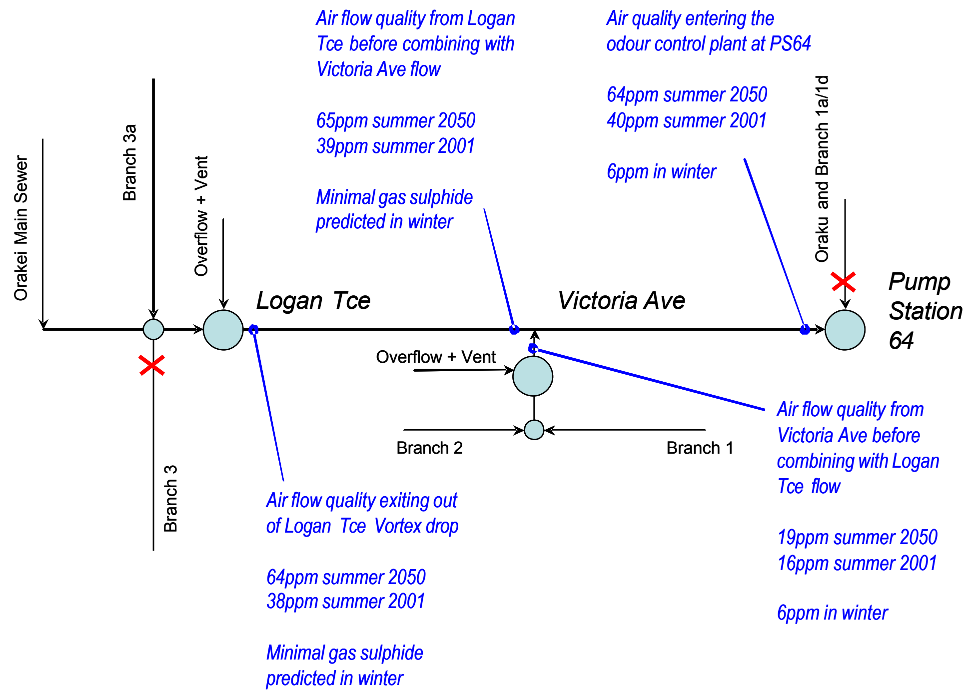
7.3 GAS SULPHIDE CONCENTRATIONS

With the design ventilation rates, gas sulphide predictions can be determined. The larger the air flow the more the gas concentration can be diluted, and vice versa is true. The model is based on theoretical equations and some empirical correlations for mass transfer of hydrogen sulphide from sewage. Information on stripping effects due to vortex drops are limited, and information from SKM field measurements in existing vortex drops has been used in this instance.

The additional turbulence will strip more sulphide in the gas phase and may cause local (or more extensive) corrosion. Though we can not confidently quantify the resulting increase in corrosion rate as there is limited data and research into this area.

Although liquid sulphide is slightly lower in 2050, the gas phase is higher as flows over the vortex drop are greater thus enhancing the amount of sulphide that is stripped. Gas concentrations are particularly sensitive to temperature, so there is a significant drop in gas sulphide levels from summer to winter months.

Figure 14: Predicted Gas Sulphide Concentrations in Hobson Bay Sewer air space



Furthermore, from the sewer arrangement, the Victoria Avenue drop shaft is connected to the main sewer via a side tunnel. Effective ventilation of this shaft will be required to prevent hydrogen sulphide gas build up which could lead to significant local corrosion in the shaft interior.

This has been done by providing an air induct to ensure the design air flow from Victoria Avenue drop to the main tunnel is obtained. The air induct is provided with a damper to tune the quantity of air flow through the air vent after the biofilter at PS64 is commissioned.

Achieving good ventilation air flows into both the Logan Terrace and Victoria Avenue drop structure shafts will act to reduce concrete corrosion by:

- Diluting the hydrogen sulphide concentrations
- Drying concrete walls in the vicinity of the shafts.

However, it is still possible that corrosion rates higher than predicted could occur at and just downstream of the vortex drop shafts as it is unlikely that the walls can be kept fully dry. Hence a corrosion resistant coating to the concrete has been included in the vicinity of the vortex drops.

7.4 SULPHIDE ASSESSMENTS DESIGN CONCLUSION

By a thorough approach to sulphide assessment and modeling significant savings can be achieved in sewer system design by a targeted approach to corrosion protection and avoidance of very expensive corrosion resistant tunnel linings.