Watercare Services Limited

ŌRĀKEI MAIN SEWER

FAILURE ANALYSIS REPORT

1 MARCH 2024

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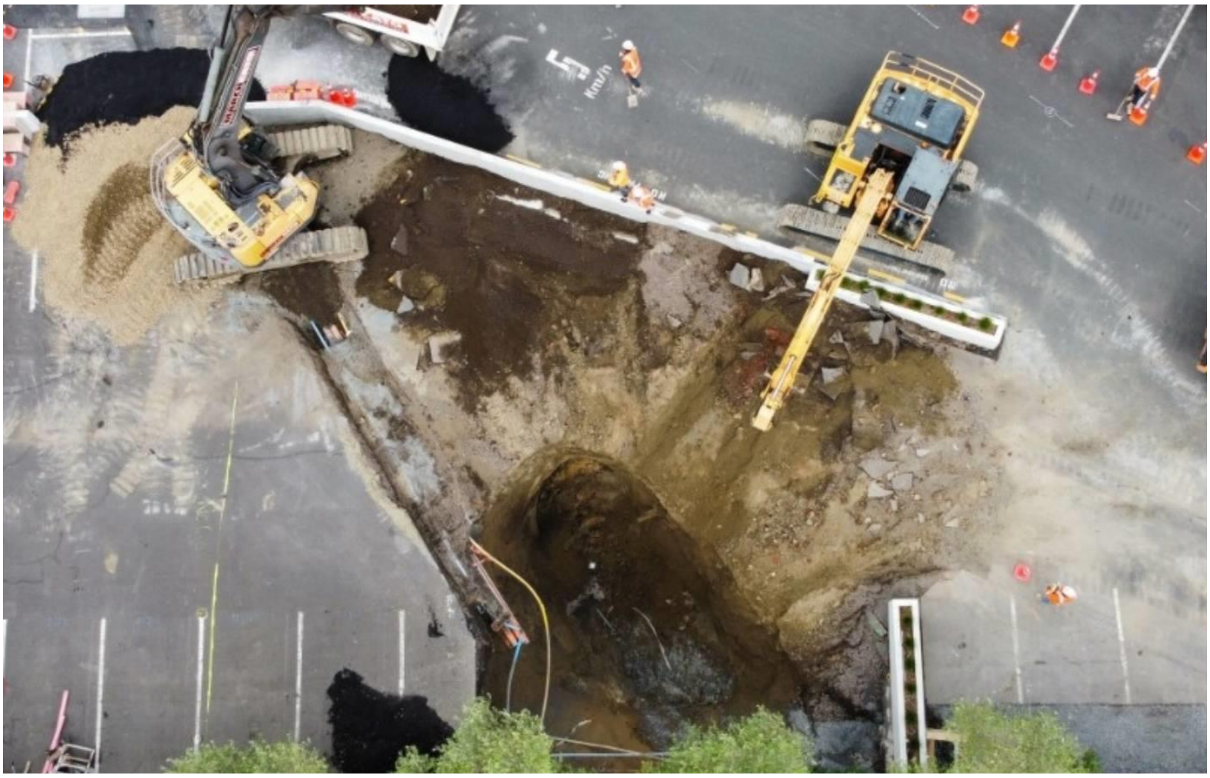

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FAILURE ANALYSIS REPORT

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EXECUTIVE SUMMARY

This report investigates the collapse of the Ōrākei Main Sewer (OMS) that occurred in the vicinity of 79 St Georges Bay Road in September 2023.

The likely cause of the failure has been identified to the extent possible from the information available. Watercare's condition assessment, maintenance and renewal practices for interceptor sewers have been reviewed and compared with best practice. Improvements to address learnings from the collapse have been identified.

Background

The OMS is 13km long. It conveys wastewater from central Auckland suburbs to the Eastern Interceptor in Ōrākei. Wastewater is then conveyed through the Eastern Interceptor to the Māngere Wastewater Treatment Plant where it is treated and discharged.

The OMS was constructed circa 1910's. It was constructed in an egg-shape. The section that collapsed is 2.3m high and 10m underground. The base of the sewer is constructed from cast in place concrete. The top is a concrete block arch with mortar between the blocks.

On 26 September 2023, a contractor who was excavating a trench for a power cable noticed a hole in the ground above the OMS and alerted Watercare. The hole grew over the next 24 hours into a large sinkhole. On 27 September the sewer collapsed, and the OMS became fully blocked.

Repair Works

To stabilise the sinkhole a steel caisson was installed over the collapsed sewer. The sides of the sinkhole were lined with concrete.

Bypass pumping was installed to convey wastewater around the collapsed section and avoid wastewater being discharged directly into the harbour during dry weather. The pumping arrangement was operational within three weeks of the collapse. Its pumping capacity is comparable to the larger pumpstations in Watercare's network which take several years to design and construct. The capacity of the temporary arrangement was limited, however, by the availability of pumps, the space required to site pumps, noise and vibration impacts and hydraulic constraints.

Installation of the bypass pumping enabled the collapsed section to be exposed and strengthening works around the collapse to be completed in December 2023.

Watercare intend to rehabilitate the section of OMS that collapsed using segmented slip-lining. This is expected to be completed in early 2024.

Likely Cause of Sinkhole and Sewer Collapse

The collapse occurred in the block arch portion of the OMS. It is likely that it was caused by the following factors acting together:

- general deterioration and weakening of the sewer over the 110 years that the OMS has been in service.
- exceptionally wet weather in 2023. The year was the wettest on record for Auckland. In the 9 months leading up to the collapse 50% more rain fell than the average for an entire year.

Significant rainfall events include the Auckland Anniversary floods, Cyclone Gabrielle and heavy rainfall on 9th May 2023.

— weak blockwork at the location that collapsed. Some of the blocks retrieved after the collapse were a lot weaker than others. They had very little strength and crumbled when disturbed. It wasn't possible to determine the extent of the weak section as most of it had collapsed into the OMS but it is unlikely that the weak section was more than 5m long.

The reason for the weak blockwork is unclear but one possible explanation is that the location could have been used as a portal for building the tunnel for the OMS. When construction was completed, equipment might have been brought out through the opening and the arch formed from the outside. The weak blocks could have been part of the final piece to close in the arch and constructed from concrete mixed on site. This has not been able to be confirmed, but it is plausible given that when the OMS was constructed the area was a natural low point.

Based on the review of the available information the likely failure mechanism is:

1. The internal surface of the block arch slowly deteriorated through a combination of corrosion and erosion.
2. High wastewater flows in the OMS during the 2023 storms accelerated erosion of the deteriorated concrete blocks and mortar joints. It is possible that some blocks were dislodged.
3. Infiltration of groundwater into the OMS increased due to its deteriorated state and the raised groundwater and flooding above the sewer.
4. Fine soil particles around the OMS were eroded by the infiltrating groundwater causing cavities to form.
5. Fluctuating groundwater levels due to the repeated wet weather caused the cavities to grow and a sinkhole to form.
6. Excavation for the power cable disturbed the already unstable sinkhole cavity, causing it to grow further and become visible at the ground surface.
7. The block arch was not able to withstand the uneven loading caused by the sinkhole. Eventually it failed under buckling and the material above it fell in and blocked the OMS.

The weaker blocks observed near the failed section most likely contributed to the sinkhole and collapse as they were:

— more susceptible to being eroded and dislodged by the high sewer flows in January and February 2023 which would have increased flow of groundwater and soil into the OMS and hastened the formation of the sinkhole.

— less able to withstand the uneven loading applied to the arch section of the OMS due to the sinkhole.

The very wet weather experienced in 2023 also contributed to the collapse by:

— increasing wastewater flows in the OMS, causing the block arch to be fully exposed to flows which accelerated erosion of the blockwork and mortar.

— causing flooding above the OMS which increased the hydrostatic loading on the pipe. The collapse is below an overland flowpath where stormwater flows across the ground surface in large rain events. The flowpath is obstructed by a solid brick wall which causes water to pond above the OMS. This increased flow of sediment ladened groundwater into the OMS either starting the formation of the sinkhole cavity or increasing existing cavities.

— repeated rain events causing fluctuations in groundwater loading which would have hastened the formation of the sinkhole.

It should be noted however that the overland flowpath and its obstruction are potential contributory factors only and are unlikely to have been the primary cause of the sinkhole formation.

Watercare's Condition Inspection Practices

Watercare's standard practice is to inspect wastewater interceptor sewers like the OMS every five years. This is aligned to the practices of other utilities. More frequent inspections are undertaken where there are concerns about the condition of pipelines.

The OMS was inspected in 2012 and 2019 using CCTV inspection, laser scanning and sonar. CCTV provides images of the inside of the pipeline from which the general condition can be assessed while laser profiling and sonar measure internal dimensions which can be used to determine the extent and severity of corrosion. Watercare were an early adopter of the profiling technology which is now used by other large utilities including Wellington Water and Sydney Water.

The CCTV inspections undertaken in 2012 and 2019 did not provide a sufficiently clear view of the pipeline to enable all defects to be identified due to limitations with the equipment available at the time. In addition, in 2019 the profiling data was not analysed as Watercare's policy at the time was to only acquire the analysis reports if issues were observed from the CCTV inspection.

An independent assessment of the 2012 and 2019 condition inspections was undertaken during the preparation of this report. This covered 1.6km of the OMS either side of the collapsed section and included assessment of the 2019 profiling information that was not available until recently. The assessment found that significant deterioration had occurred along the OMS between 2012 and 2019. If identified at the time this level of deterioration should have triggered a more in-depth investigation of the pipeline condition and likelihood of collapse, involving activities such as walkthrough inspection, coring, and testing of pipe wall. However, if these investigations had been undertaken it is quite possible that the weak blockwork which was a significant contributor to the collapse still might not have been picked up as the weak blocks only cover about 5m a pipeline section that is 118m long between manholes.

It is a primary recommendation of this report that Watercare adopt the following to improve the identification of condition issues:

1. Profiling reports be provided with all inspections so the extent and rate of deterioration can be assessed.
2. The quality and resolution of the CCTV inspections be improved to provide a clearer view of the pipe wall and aid the identification of faults.
3. Undertake more in-depth assessments to determine the strength of blockwork at low points where tunnel portals for the construction of the OMS might have been located. This could involve laser profiling to determine the extent of erosion, followed by core sampling and testing at potentially weak areas.

Watercare's Condition Assessment and Renewal Practices

Timely renewal of interceptor sewers to avoid failures will become increasingly important as the assets age and extreme weather events like those experienced in 2023 become more prevalent due to climate change.

Whilst Watercare's investment into renewal of wastewater pipes has been low over recent years, a significant increase in renewals is planned. The latest Asset Management Plan has allocated $1.9b over the next 20 years, which will increase investment to more than double the median amount currently being spent by European Countries.

But it is important that the money available for renewals be invested into the right assets at the most appropriate time, particularly for wastewater interceptors where there is a high consequence of failure. This requires assessment of the information collected from condition inspections and other sources to determine the likelihood and consequence of failure.

Whilst Watercare's corporate level policies and asset management practices are aligned to international standards, there is limited guidance on how these corporate level documents should be applied to the management and renewal of wastewater systems.

It is a primary recommendation of this report that Watercare adopt the following to improve condition assessment and renewals practices:

1. Develop a Condition Assessment Strategy that specifies:
    - timing of inspections, which will depend on the consequence of failure, the previously observed condition of the asset and the forecast rate of deterioration.
    - condition inspection techniques to be used under various circumstances.
    - processes for deriving likelihood of failure rankings from condition inspections and other information collected.
    - trigger levels for undertaking more intensive or additional investigations, e.g. evidence of increased corrosion or major rainfall that could cause high flows.
2. Develop a Renewals Interventions Strategy that:
    - specifies triggers for undertaking repairs and renewals based on the observed defects/condition and the consequence of failure.
    - enables a prioritised list of pipelines for renewal to be developed based on the triggers set out above.

It is understood that preparation of these documents is underway and is due to be completed by June 2024.

RECOMMENDATIONS

Condition Inspection Practices

It is recommended that Watercare continue to inspect transmission sewers every 5 years using CCTV and laser and sonar profiling with inspections being undertaken on tighter frequencies on pipelines where there are concerns about condition. However, to improve identification of condition issues it is recommended that the following be adopted:

- Timing of inspections
  - Undertake condition inspections after events that could trigger rapid decline in condition, e.g. after large storms.
- Inspection Practices
  - Improve the quality and resolution of the CCTV inspections to provide a clearer view of the pipe wall and aid the identification of faults. Either use a CCTV camera that can pan towards the pipe wall and zoom into issues or use the latest CCTV and laser & sonar profiling units which have multiple cameras that look both straight ahead and towards the pipe wall
  - Reinstate cleaning of the OMS using the plough, subject to addressing health and safety concerns. This will improve the identification of defects as the personnel who are required to travel down the sewer with the plough will also be able to observe the condition of the sewer and report on issues. Watercare have already started investigating reinstating the use of the plough. It is also worth investigating whether suitable camera equipment could be installed to allow unmanned inspections while cleaning the line with the plough
  - Produce detailed CCTV logsheets to record inspections. Assign Structural Condition Grades to provide an initial grading of condition and to identify the need for more intensive condition assessment and likelihood of failure analysis. Use a logging and grading system better suited to assessing defects in brick pipelines, e.g. the 4th Edition of the New Zealand Pipe Inspection Manual or the Conduit Inspection Reporting Code of Australia, both of which were published since the last inspections were undertaken on the OMS.
- Analysis of Condition Assessment
  - Change standard practice so that laser and sonar profiling inspections are analysed as a matter of course rather than only if issues are identified from the CCTV inspection. This could identify faults not picked up from the CCTV inspection
  - Compare laser profiling against previous inspections to determine the extent and severity of corrosion that could signal a trigger for renewal.

Condition Assessment and Renewal Practices

It is recommended that Watercare continue with a risk-based approach to the management of assets. This approach prioritises condition assessment and renewal of assets with a high consequence of failure like transmission sewers.

However, to help ensure that the condition assessment and renewal works are undertaken on the the right assets at the most appropriate time, it is recommended that Watercare develop the guidance documents and practices outlined below for the management of interceptor sewers.

Documentation of procedures and the reasoning behind decisions will also help auditing of decision quality and give a starting point for refining practices over time to facilitate continuous improvement.

- Consequence of Failure
  - Update processes for determining the criticality of pipelines to include all factors that could impact the consequence of failure or ease of repair, e.g. pipes under buildings, deep pipes, pipes near sensitive receiving environments. This will highlight transmission mains that might warrant more frequent condition assessment or earlier renewal.
- Likelihood of Failure
  - Develop a Condition Assessment Strategy that specifies:
    - Condition inspections – the techniques to be used under various circumstances
    - Timing of inspections, which depend on the consequence of failure, the previously observed condition of the asset and the forecast rate of deterioration
    - Other information be collected, e.g. past issues in the vicinity of the sewer
    - Processes for deriving likelihood of failure ranking from the condition inspections and other information collected
    - Circumstances that may necessitate the need for additional inspections, e.g. after high flows that could cause rapid deterioration
    - Trigger levels for undertaking more intensive investigations.
- Renewals Processes
  - Renewals interventions strategy be documented that specifies the repairs and renewals to be undertaken and the urgency for undertaking these works based on the observed defects/condition and the consequence of failure. This will improve transparency of renewals decisions
  - Consider undertaking structural analysis of block and brick-built sewers using finite element analysis to determine the extent of pipe wall deterioration that can be tolerated under various loading conditions. Sensitivity to uncertainties such as block strength should be considered. This will improve assessment of the likelihood of failure and enable the setting of trigger levels for intervention
  - Develop a prioritised list of pipelines for renewal based on observed condition and the triggers set out in the Renewals Intervention Strategy.

It is understood that Watercare are in the process of improving the documentation of their processes in line with the recommendations above. They expect to be complete by July 2024.

ABBREVIATIONS

AMP	Asset Management Plan
ARI	Average Reoccurrence Interval
CIPP	Cured In Place Pipe
EOP	Engineered Overflow Points
FEA	Finite Element Analysis
GRP	Glass Reinforced Plastic
IIMM	International Infrastructure Management Manual
MBGL	Metres Below Ground Level
MH	Manhole
OLFP	Overland Flow Paths
OMS	Ōrākei Main Sewer
RCM	Reliability Centred Maintenance
WSAA	Water Services Association of Australia
WSL	Watercare Services Limited
WWTP	Wastewater Treatment

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1 INTRODUCTION

1.1 PURPOSE OF REPORT

This report considers the collapse of the Ōrākei Main Sewer (OMS) that occurred in the vicinity of 79 St Georges Bay Road in September 2023. Watercare Services Limited ('Watercare' or WSL) engaged WSP to conduct a failure analysis of the incident. As per the Terms of Reference (Appendix A), this assessment is split into three parts:

— Investigation of the incident and likely cause of failure.

— Review of Watercare's maintenance and condition assessment practices.

— Review of Watercare's rehabilitation and renewal planning.

WSP has compared Watercare's practices for interceptor sewers with best practice and provided recommendations for improvements.

Information made available for this work was believed to be complete and correct at the time it was made available, but it is possible that additional or better information may become available over time, which could influence the findings of the report. Therefore, before undertaking any physical works, further investigations should be carried out to confirm the underlying assumptions.

1.2 OVERVIEW OF INCIDENT

On Tuesday 26 September 2023, Watercare was alerted to the formation of a sinkhole over the OMS at 79 St Georges Bay Road in Parnell.

The sinkhole grew over the next 24 hours. On 27 September 2023 the sewer collapsed, and the OMS became fully blocked. This resulted in sewer flows being discharged into the Waitematā Harbour via two engineered overflow points (EOPs) – one at Mechanics Bay and another at Daldy Street. The situation remained until 17 October 2023 when temporary bypass pumping was installed.

A more complete timeline is included in Appendix B.

Further site investigation showed that the blockage spanned 25 metres. It comprised mainly of large rocks and clay. Indications were that a section of the roof of the OMS, approximately 3.5m long, had collapsed.

2 BACKGROUND

2.1 OVERVIEW ŌRĀKEI MAIN SEWER

The OMS is a transmission main sewer, i.e. a large pipe that collects wastewater from local wastewater pipes to convey the flow for downstream pumping or treatment (also called an 'interceptor'). The OMS is 13km long. It runs from the Oakley Creek Esplanade Reserve to Ōrākei Domain, conveying wastewater from central Auckland suburbs to the Eastern Interceptor, refer Figure 2:1.

When the OMS was constructed in the 1910's, it discharged wastewater into the Waitematā Harbour through an outfall at Ōrākei Bay. The OMS was connected to the Eastern Interceptor in 1960's (Smith, Rogers, Graham, & Brown, 2016). Wastewater is now conveyed through the Eastern Interceptor to the Mangere Wastewater Treatment Plant where it is treated and discharged.

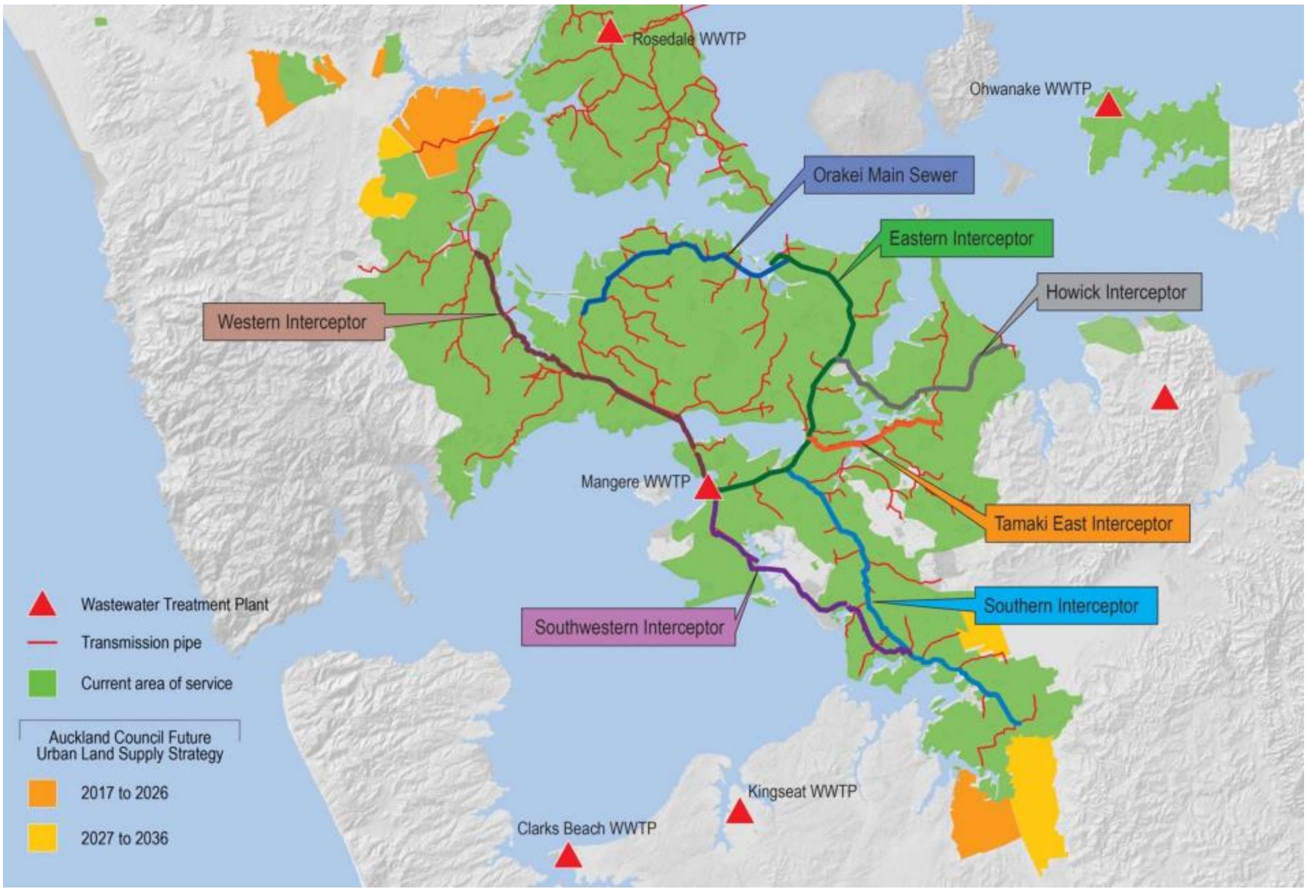

Figure 2:1 Map of wastewater transmission pipes in Auckland (Auckland Council, 2021)

2.2 LOCATION OF THE FAILURE

The sinkhole and collapse occurred at 79 St Georges Bay Rd. Figure 2:2 shows an aerial photograph of the collapse after initial works to stabilise the area had begun.

The collapse is located approximately midway between Manholes 15 and 16, shown on Figure 2:3 i.e. 65m downstream of Manhole 16, 54m upstream of manhole 15. At this location, the OMS has a nominal diameter of 2.1m and is about 10m underground (WSL, 2023a).

Figure 2:3 also shows the location relative to buildings and other services. A 1.35m diameter stormwater pipe crosses above the pipe near the collapse, implications of this are discussed in Section 3.3.

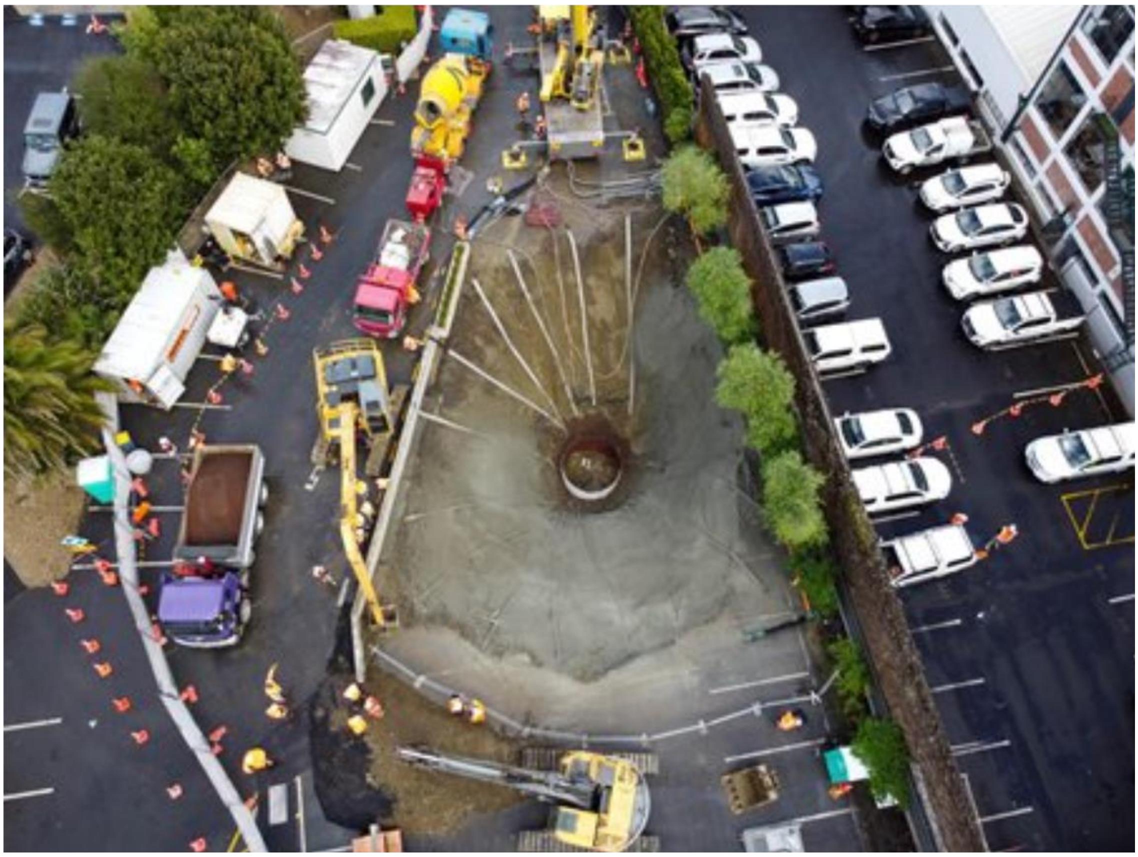

Figure 2:2 Drone aerial image of the sinkhole on 29/09/2023 (photo supplied by WSL)

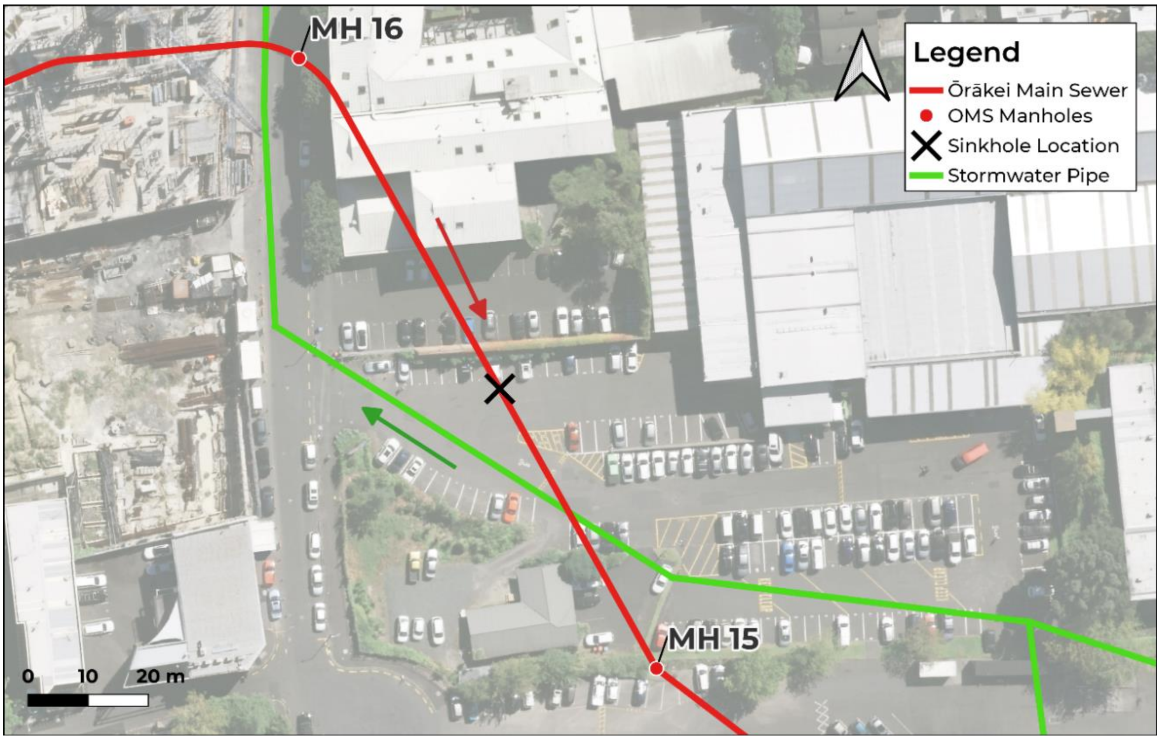

Figure 2:3 Map of sinkhole location, including arrows showing pipe flow directions

2.3 EVENTS LEADING UP TO INCIDENT

Two notable activities happened prior to the incident:

Very Wet Weather

- 1,690mm of rain fell in the 9 months of 2023 preceding the incident. This is 50% more than the average for an entire year
- There were three significant rainfall events in the first half of 2023. These events occurred in January (Auckland Anniversary Floods, >100yr ARI), February (Cyclone Gabrielle, ~20yr ARI) and 9th May (2-5yr ARI)[1].

Transformer installation

- A landowner was installing an electrical transformer adjacent the collapsed section
- A 900mm deep trench for a power cable was being excavated across the location where the sinkhole was discovered
- At the time, the asphalt surface had been cut for the trench and the ground below was exposed to weather
- Watercare had issued a works over permit giving permission for the works to be undertaken near the OMS. There is no indication that the contractor carrying out the works damaged the OMS in any way.

Refer to Appendix B for an extended timeline of events relating to the OMS and its 2023 failure.

2.4 IMMEDIATE REPAIR WORKS

Immediately after the collapse the following works were undertaken, refer Figure 2:4:

1. Stabilisation of sinkhole
   - 3m diameter steel caisson (protective casing around the hole in the OMS) was installed over the collapsed sewer
   - Shotcrete was sprayed on the sides of the sinkhole.
2. Unblocking of sewer
   - Hydro-excavation (jetting water) was used to clear the blockage. This was only partially successful due to the extent of the blockage and the size of the debris.
3. Temporary bypass pumping installed (refer Figure 2:5)
   - A stop log (steel door) was installed at manhole 16 to restrict wastewater flows
   - Six bypass pumps (nominal diameter 200mm) were installed at manhole 16

[1] ARI (Average Recurrence Interval) is the frequency of an event expressed in years. This refers to the average time between rainfall events that exceed a certain magnitude.

— A bypass pipeline was installed to convey wastewater around the collapse, discharging back into the OMS at MH14.

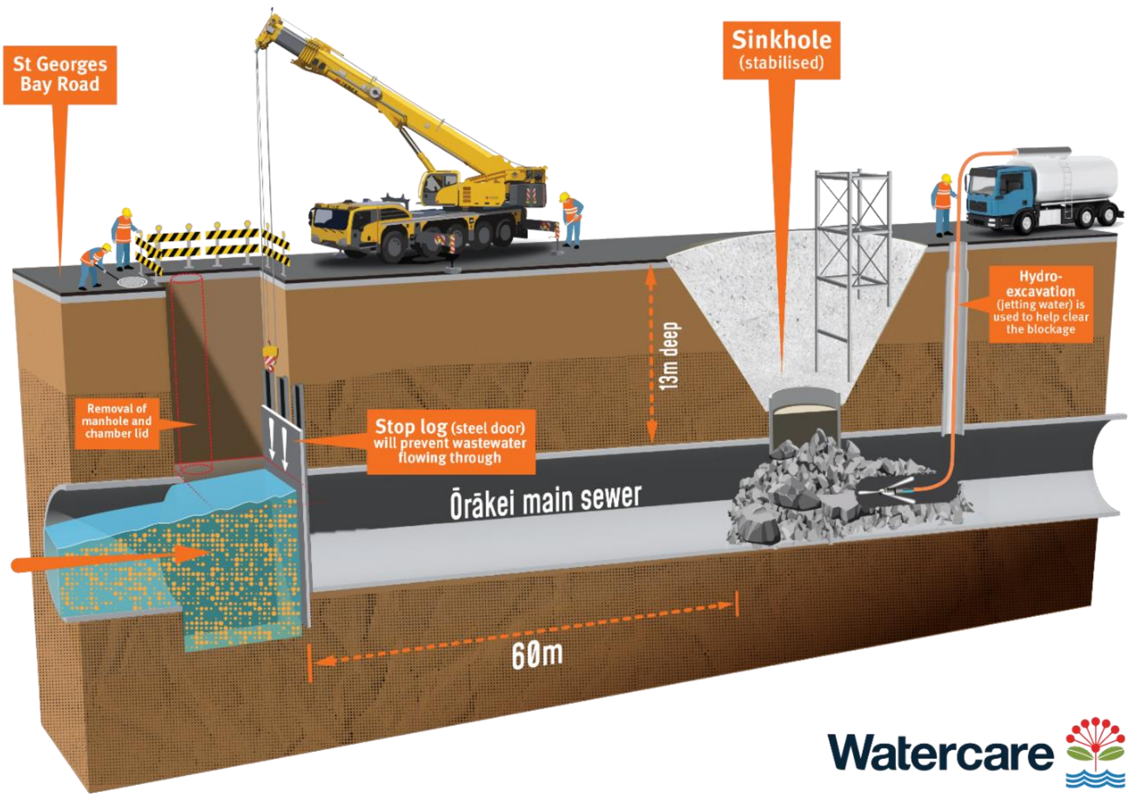

Figure 2:4 Graphic of OMS failure and initial repairs (supplied by WSL)

The bypass pumping conveyed about 600L/s which is approximately the average dry weather flow of the OMS. The pumping capacity is comparable to some the largest pumpstations in the Watercare network.

The capacity of the temporary bypass pumping was limited by:

— Availability of pumps at short notice

— Space available to site pumps

— Noise and vibration impacts

— Pump suction head limitations which meant that the pumpstation site had to be excavated and the pumps located below ground level. This added to the space constraints

— Hydraulic constraints – in a permanent pumpstation, the pumps would normally draw water from a wet well (tank). The wet well would be sized so that wastewater was efficiently drawn out when multiple pumps were operating. But it wasn't possible to construct a wet well for the temporary pumpstation due to space and time constraints. The pumps suck wastewater directly out of the OMS itself which isn't hydraulically efficient. As more pumps are added, they begin to compete against each other, and pumping capacity reaches a threshold.

Bypass pumping was operational from 17 October 2023. This avoided wastewater being discharged directly into the harbour during dry weather. The installation of the bypass pumping also enabled unblocking and repair works to be undertaken in a dry setting. Design and construction of a permanent pumpstation and rising main of this size would normally take several years to complete.

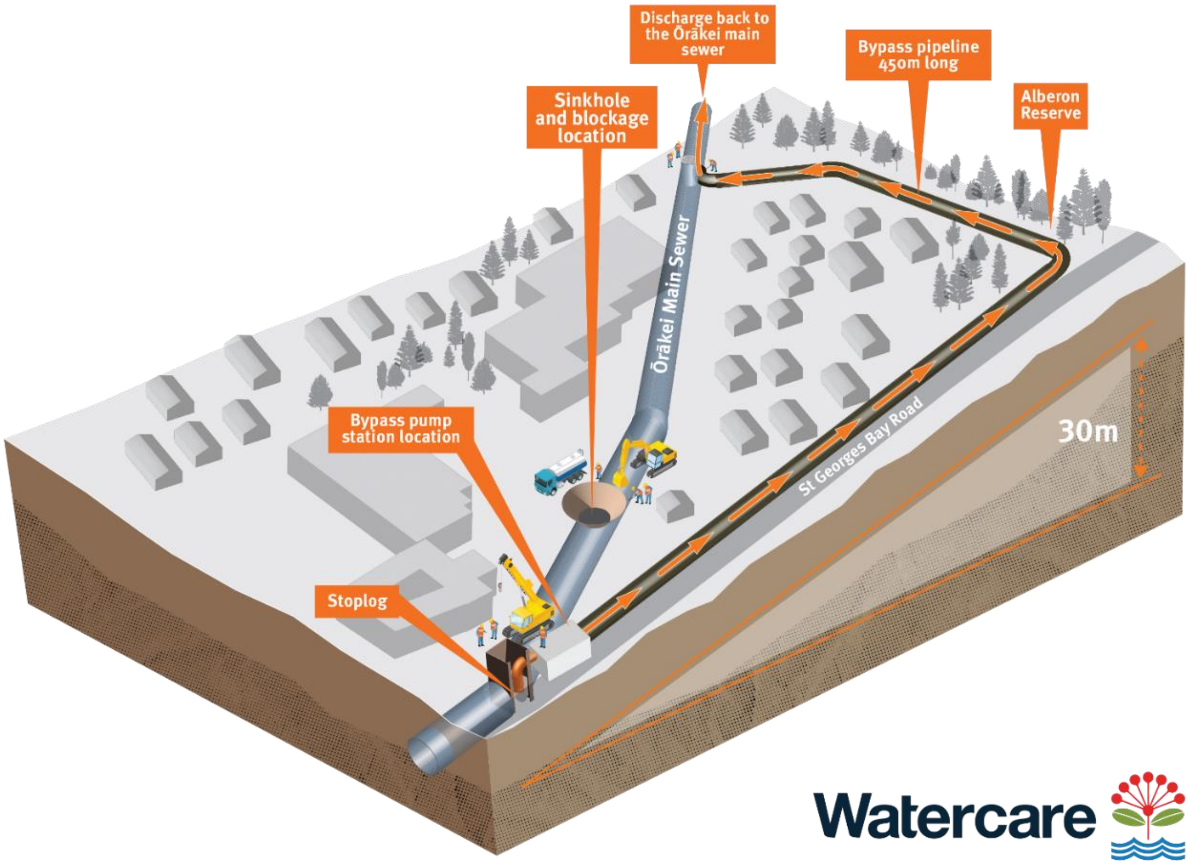

Figure 2:5 Graphic showing sinkhole location and temporary bypass pipeline (supplied by WSL)

The following steps are being undertaken to repair the OMS:

1 Expose the collapsed section (completed November 2023)
2 Strengthen the pipe around the collapse (completed December 2023)
3 Rehabilitate the OMS from manhole 16 to manhole 15 using segmented slip-lining (refer to Section 8.5.3 for details of this rehabilitation technique)

2.5 CONSTRUCTION METHOD

The OMS was constructed in 1911. It is oviform (egg shaped), refer Figure 2:6. Appendix C contains some of the original OMS drawings.

The section in St Georges Bay Rd that collapsed is 2.29m high by 1.52m wide. The base of the sewer is constructed from cast in place concrete. The top is a concrete block arch which appears to have been constructed from rectangular blocks. The depth to the invert is approximately 10.4m.

Figure 2:7 shows a photograph from the construction of a similar sewer. However, the arch section of the OMS is formed from concrete blocks rather than from bricks as shown in the photograph.

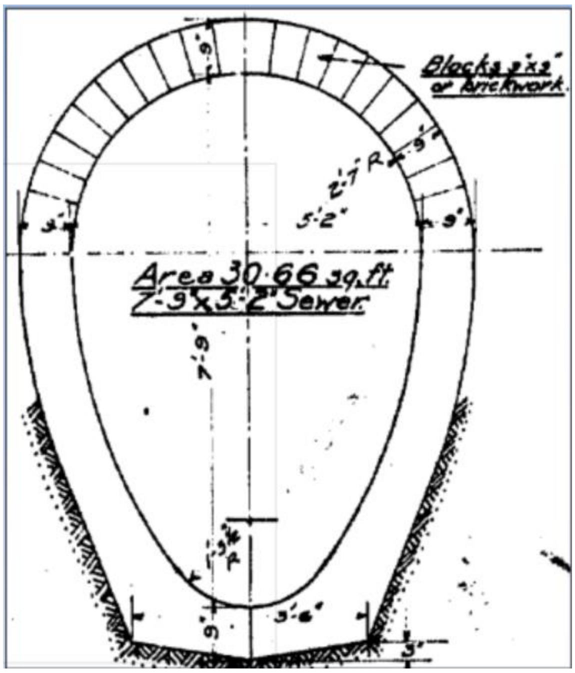

Figure 2:6 Profile of OMS (Extract from drawings – Appendix C)

Figure 2:7 Photo of Hobson Bay Sewer Construction in 1910 (WSL, 2018)

The OMS was either constructed within a hand dug tunnel or a trench depending on the depth of the pipeline. It is uncertain which method was used for construction of section in St George Bay Rd that collapsed.

The sequence of construction for the OMS was:

1 Dig hand dug tunnel or trenched
2 Cast concrete base in place
3 Install block arch
4 Backfill tunnel or trench above pipe.

Where the pipe was installed in a tunnel, it is quite possible that either a void remained between the top of the arch and tunnel or the gap was filled with poorly compacted material (refer to Section 5.2.1).

3 INVESTIGATIONS AND FURTHER INFORMATION

3.1 SEWER FLOWS

The OMS was originally constructed as a combined sewerage system conveying both wastewater and stormwater flows, which was common practice at the time. Modern practice is to install separated systems for conveyance of wastewater and stormwater.

The catchments served by the OMS (shown in Figure 2:1) have been extended and modified over time. The OMS now serves catchments that were originally constructed as:

— combined systems

— separated systems

— combined systems but have since been largely separated.

As a result, there is significant variation between dry weather and wet weather flows, refer Figure 3:1.

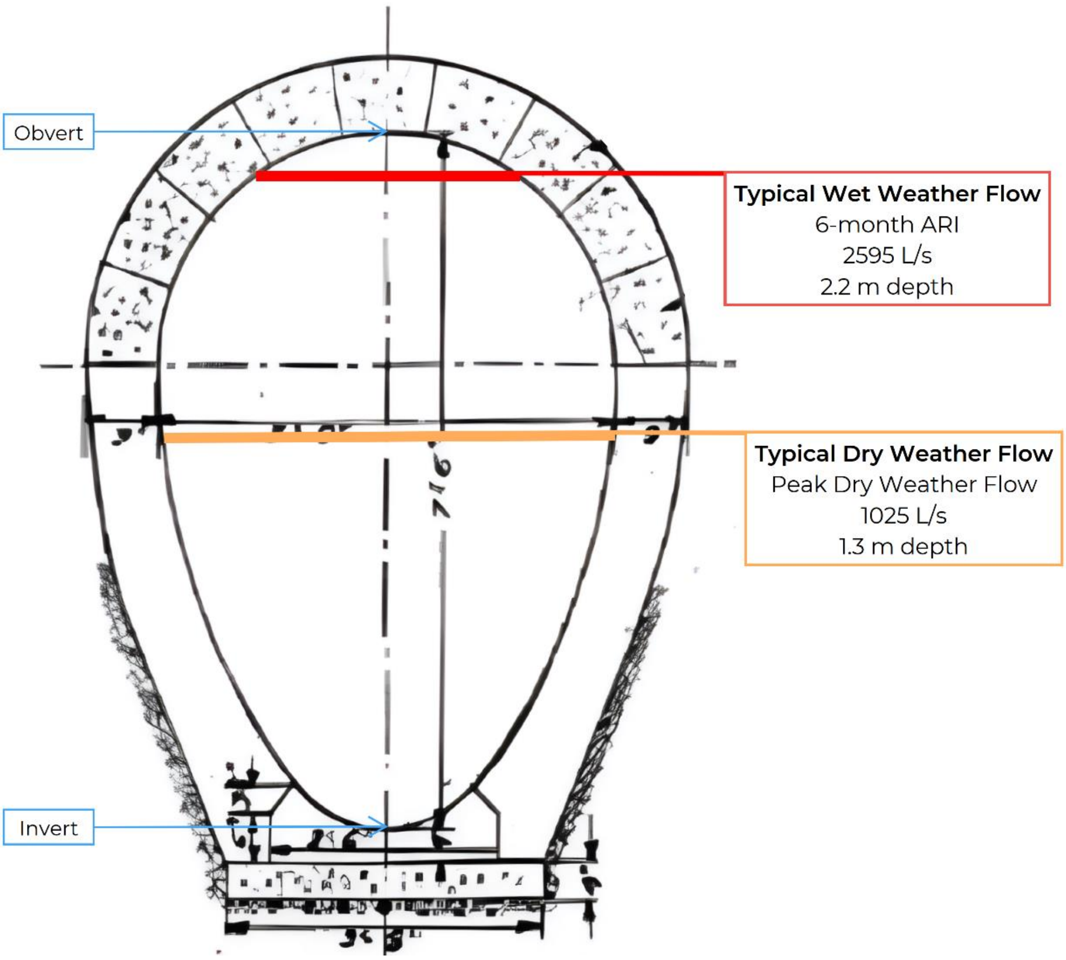

Figure 3:1 Depth of typical flow rates relative to the OMS cross-section (supplied by WSL)

During dry weather, the OMS largely conveys wastewater from households and industry, i.e. water that goes down sinks, washing machines, showers, baths and toilets. Wastewater flows will vary throughout the day based on usage patterns. Peak daily dry weather flow is estimated by Watercare to be 1,025L/s. The flow depth is 1.3m. That is, under dry weather conditions wastewater flows will be contained within the cast concrete base of the OMS.

During wet weather the OMS also conveys:

- Stormwater from catchments that are fully combined or only partially separated
- Inflow, i.e. water that directly enters the wastewater system from sources such as roof downpipes, gully traps, and drain connections. Ideally these sources shouldn't be present but in practice they occur in almost all wastewater systems
- Infiltration, i.e. groundwater that seeps into the system through leaks in pipes and manholes. Whilst infiltration should be minimised as much as practical, it does occur in most wastewater systems.

The amount of wet weather flow typically increases with the severity and duration of the storm. Wet weather events are described in terms of Average Recurrence Interval (ARI), i.e. the average period between events of a certain size. For example, a six-month ARI flow will occur on average once every six months. It is important to note however that these are average return periods and there could be times when actual return periods are shorter or longer than the average.

The flowrate during a 6-month ARI event is 2,595L/s, i.e. 2.5 times the peak dry weather flow. The flow is 2.17m deep, so the concrete block arch is partially exposed to flows.

During severe storms, there can be a rapid rise in flows due to increased inflow caused by surface flooding overtopping gully traps and other wastewater features that have been positioned to avoid water entry under normal conditions. There can also be increased infiltration due to high ground water levels.

Two severe storms occurred during January and February 2023, refer Section 3.3. The storm during Auckland Anniversary Weekend in January 2023 is estimated to have an ARI greater than 100 years. During this storm, the OMS would have been running full (i.e. the top of the concrete arch would have been exposed to flows) and the pipe may have been subjected to up to 800mm of surcharge[2]. This surcharge could have applied upward pressure on the block structure causing the arch to destabilise and possibly washing out weak mortar.

[2] This is based on measures recorded by WSL elsewhere on the OMS. Surcharge refers to the additional pressure on a pipe when it's at full capacity.

3.2 GROUND CONDITIONS

The technical memo in Appendix D introduces the engineering geological setting and geotechnical conditions and discusses the potential impact the ground conditions might have on the failure mechanism of the sewer.

The site is located at base of an infilled stream gully. As such the top portion of the ground profile consists of fill comprising mostly clayey silt, silty clay, and silt, underlain by residual soils of the East Coast Bays Formation (ECBF) that are comprised of clayey silt, sandy silt, and silt, transitioning into highly to moderately weathered ECBF.

The inferred ground profile based on historical geotechnical reports for adjacent building sites (Tonkin & Taylor Ltd) is shown in Figure 3:2. The ground profile from site observations made after the discovery of the collapse (Baseline Geotechnical Ltd) are summarised in Table 3:1.

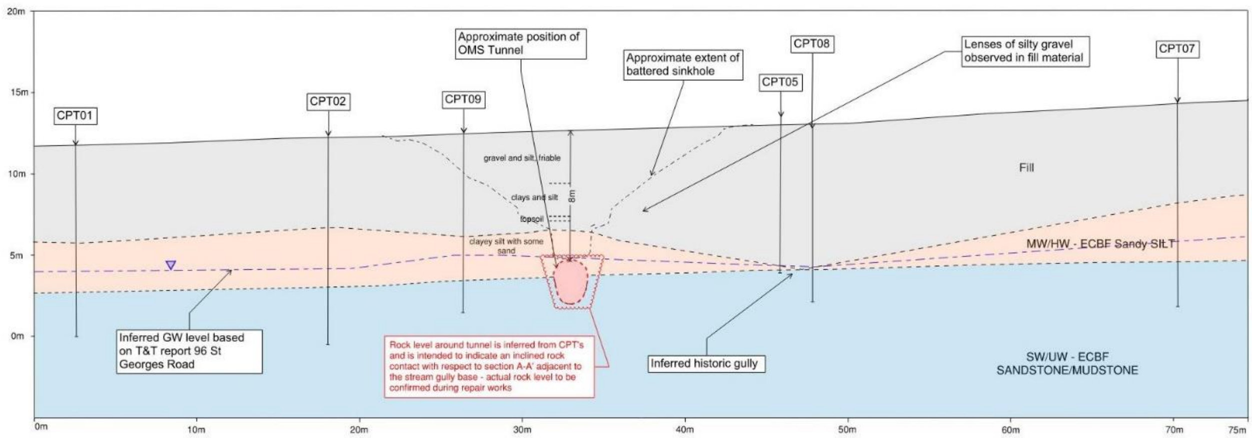

Figure 3:2 Cross-section showing the profile of the historic infilled gully in relation to the sinkhole and pipeline.

Table 3:1 Summary of soil profile above the crown hole collapse

Depth (mbgl)	Geology Unit	Material Description
0-3.5m	Fill	Gravel and silt, brown and friable
3.5-5.3m	Fill	Clays and silts, mottled orange, light brown and brown
5.3-5.6m	Buried Topsoil	Clayey silt with organics, dark brown to black
5.6-8.8m	Weathered ECBF	Clayey silt with some sand, brown

Note: mbgl = metres below ground level.
The invert of the OMS is approximately 10m below ground level at this location.

3.3 STORMWATER

Based on a desktop assessment of available information, and site observations, (refer to Appendix E) it is considered that stormwater may have contributed to the formation of the sinkhole in the following ways:

Overland Flow Paths (OLFPs)

- The collapse is in a low point. During large events, stormwater pipes become full, and the excess stormwater flows over the ground surface via OLFPs
- There were at least three rainfall events in the first half of 2023 that are likely to have led to the activation of OLFPs through 79 St Georges Bay Road. These events occurred in January (>100yr ARI), February (~20yr ARI) and May (2-5yr ARI)
- The high total rainfall during the nine months leading up to the formation of the sinkhole (1,690mm versus an annual average of 1,137mm) is likely to have resulted in saturated ground conditions and more frequent activation of OLFPs
- The existing OLFPs are obstructed by a solid brick wall near the location of the sinkhole, creating a trapped low point (refer Figure 3:3). This would cause water to pond above the location of the sinkhole to depth of ~0.25-0.50m
- Ponded water at the trapped low point would seep into the ground through weak points in the seal or other pervious areas (e.g. garden beds, gravel areas).

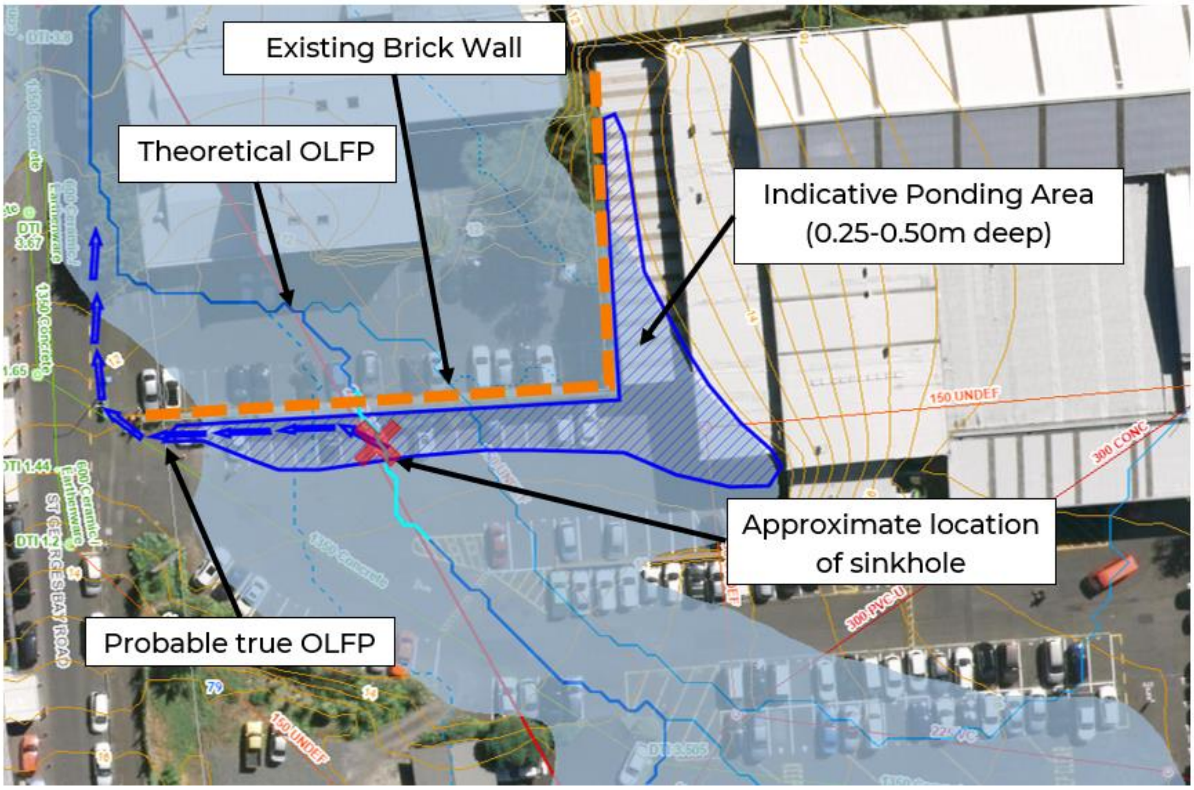

Figure 3:3 Overland flow paths near the sinkhole location

Reticulated Stormwater Network

- The existing stormwater network within 79 St Georges Bay Rd and upstream has an approximate capacity to accommodate between a 2 year and 10-year ARI event
- The existing stormwater network was likely to have been under surcharge pressure during the significant rainfall events in January, February, and May 2023
- It is understood that there are no significant faults in the DN1350 stormwater pipe that crosses above the OMS, upslope of the sinkhole location (we have not been able to sight CCTV inspections to confirm). However, it is likely that the surcharge conditions would have caused water to exfiltrate from the pipe into the surrounding trench material adding to subsurface groundwater flows.

The combination of the above would have likely led to significantly elevated local groundwater flows and levels in the vicinity of the OMS.

It is important to note that the stormwater factors above in and of themselves are considered potential contributory factors only and are unlikely to have been the primary cause of the sinkhole formation. While it would have led to elevated subsurface flows there would need to have been a preferential flow path in the soils above the OMS for this to have influenced the formation of the sinkhole.

3.4 PREVIOUS REPAIRS UNDERTAKEN IN THE VICINITY OF THE COLLAPSE

Since the OMS was constructed in 1911, several repairs have been undertaken, including work in the general vicinity of the collapse. Figure 3:4 is a map showing the location of the repairs in relation to the 2023 failure. The repairs are further described in the following subsections.

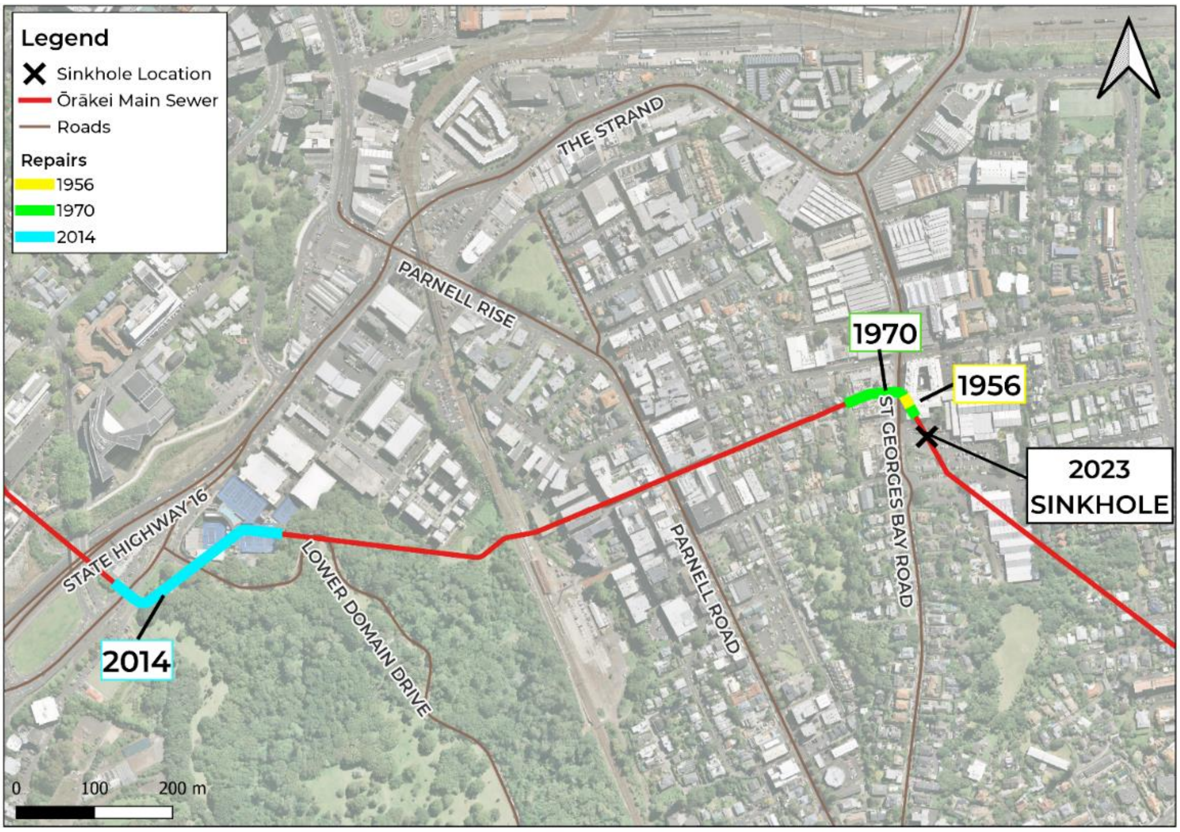

Figure 3:4 Locations of previous OMS repairs in relation to the 2023 failure

3.4.1 FAULT UNDER NESTLÉ BUILDING – 1956

In 1939, the Nestlé chocolate factory building was constructed on the corner of Cleveland Rd and St Georges Bay Road, see Figure 3:5. Part of the factory was built directly above the OMS.

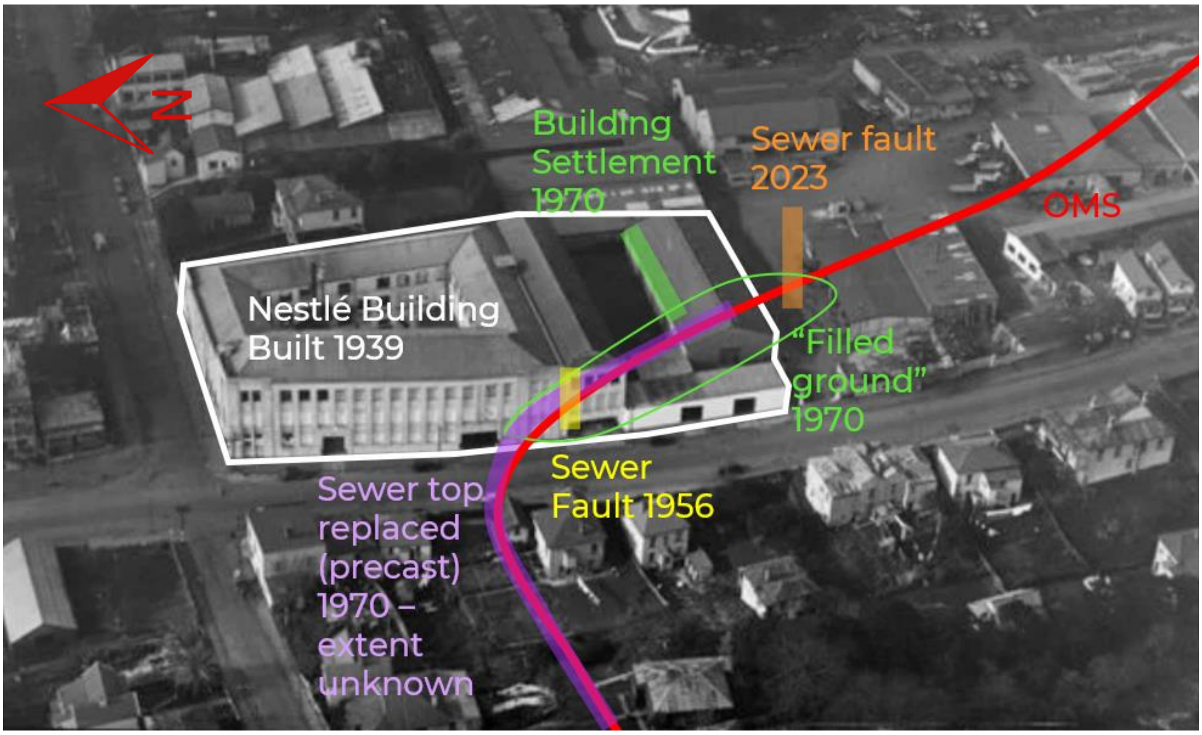

Figure 3:5 Annotated 1949 aerial image (supplied by WSL)

In 1956, a fault occurred in the OMS underneath the Nestlé building. The extent and cause of this issue is unclear. It would appear from the sketches from the time, shown in Figure 3:6, that the issue might have been localised and a result of damage caused by the building foundations. However, it has not yet been possible to confirm this.

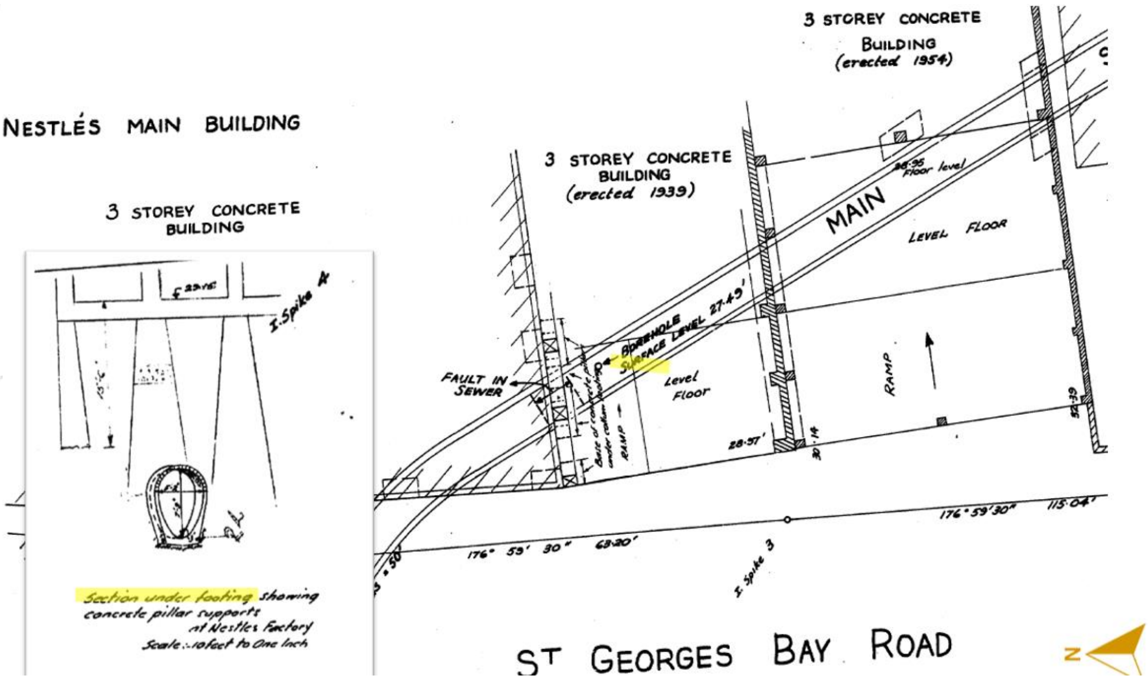

Figure 3:6 Fault Under Nestlé Building circa 1956

3.4.2 NESTLÉ SUBSIDENCE – 1970

Subsidence and cracking were recorded at the Nestlé building in the late 1960s. It is unclear whether this was directly related to the OMS. However, in 1970, the arch section of the OMS under the building was replaced with new precast concrete sections. This involved tunnelling above and to the side of the OMS, refer Figure 3:7. The timber tunnel sets were left in place. The gap between the sets and the OMS was backfilled with stabilised scoria. This repair extends from MH 16 to 47m downstream, i.e. less than 20m from the collapse.

There is currently no reason to believe that the issues that might have caused the arch section of the OMS to be replaced here extend through to the location of the 2023 collapse, i.e. there are no indications of cracking or general deformation of the pipe visible on the CCTV inspections or laser profiling.

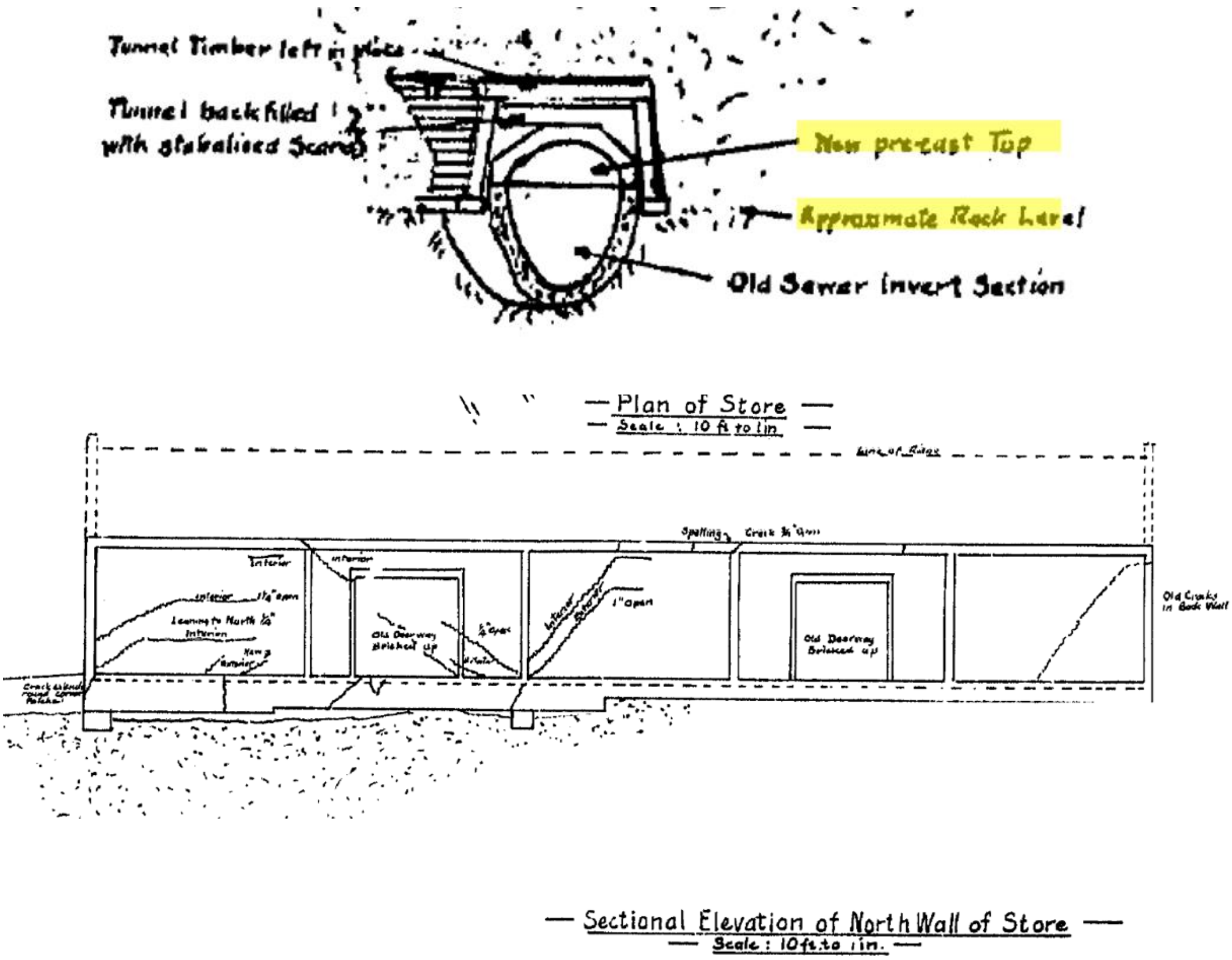

Figure 3:7 Nestle's subsidence/ cracks in 1970

3.4.3 STANLEY ST REHABILITATION – 2014

In 2011 the sewer between MH21 and MH19 (refer Figure 3:8) was identified as being in poor condition. Rehabilitation was recommended (AECOM, 2011). This section is approximately 1 km upstream of the section that collapsed in 2023.

The condition assessment undertaken at the time provides an indication of the possible condition of other sections of the OMS.

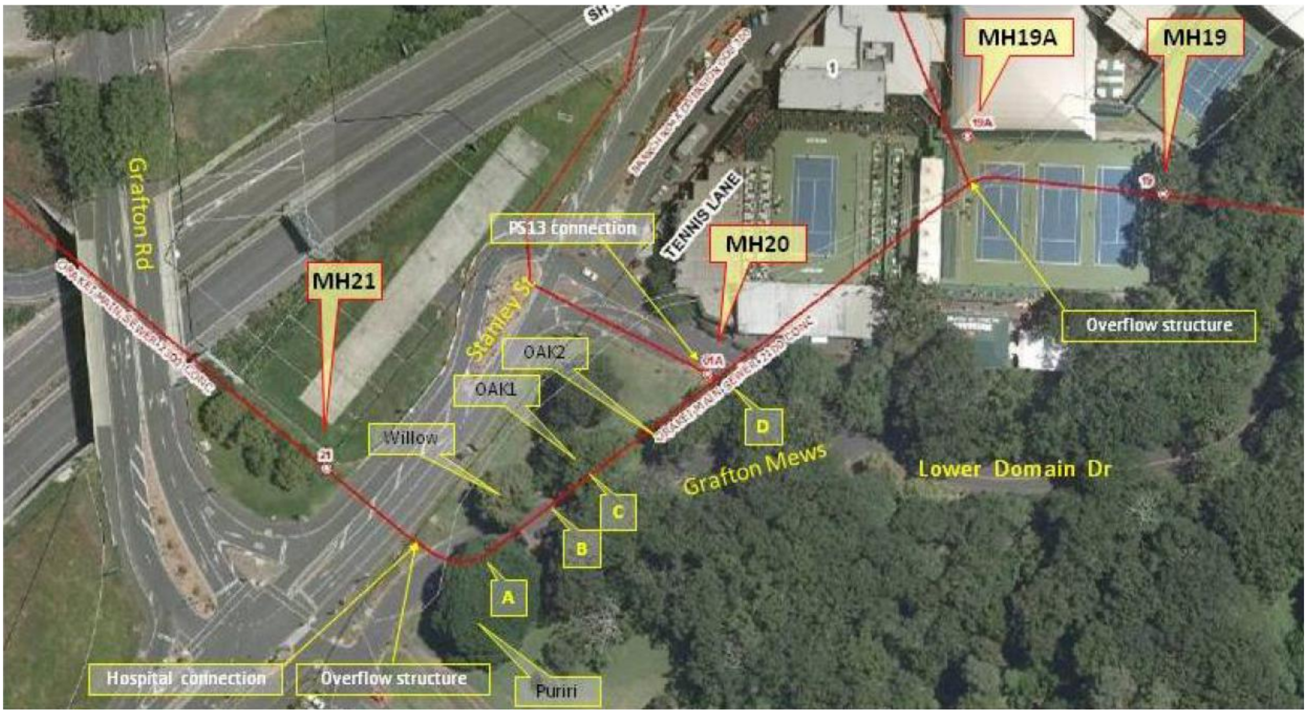

Figure 3:8 Location of Stanley St rehabilitation in 2014

The preliminary design report for the works noted that the "structure (of this section of the OMS) has suffered from attack by corrosive gases[3], and recently, penetration by tree roots, particularly in the area lying under Grafton Mews. During the last decade, the extent of the damage to this stretch of the sewer has become more apparent" (AECOM, 2011).

It was noted from a walkthrough inspection that there were "many joints (in the concrete block crown) where the mortar has completely disappeared." However, "there was no obvious indication of any serious damage to the cast-in-place lower part of the sewer".

From the compression tests carried out on core samples taken from block arch, AECOM deduced "that the majority of the blockwork is made up of material with a compressive strength of 15Mpa or less" (AECOM, 2011).

Figure 3:9 taken during the subsequent repair of the OMS demonstrates the poor condition of the block work and mortar joints. Note that the capping layer was installed during the repair and was not part of the original construction of the OMS.

[3] Strictly speaking, corrosion of concrete in pipelines that carry wastewater is caused by acid produced by microbes that convert hydrogen sulfide gas to an acid, but the gas itself does not attack the concrete.

The preliminary design report recommended that the section between points A and D, as marked on Figure 3:8, be rehabilitated "through the installation of glass reinforced plastic (GRP or "fibreglass") linings to provide a structural solution, and non-structural rehabilitation be undertaken on the remaining parts of the reach between MH21 and MH19 by application of corrosion resistant calcium aluminate cement (CAC) mortar." This work was undertaken in 2014.

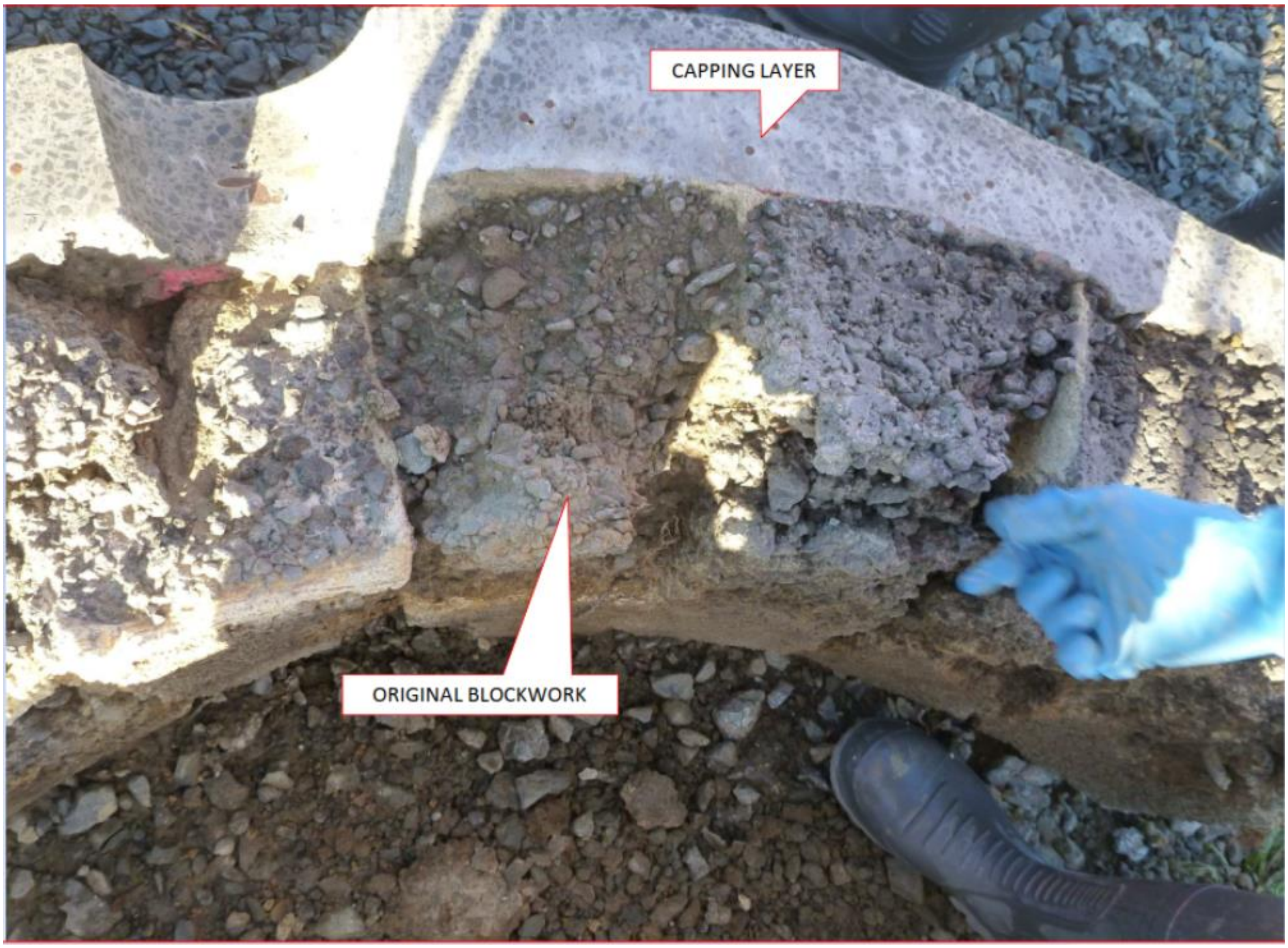

Figure 3:9 Photo of capping layer and original blockwork

4 REVIEW OF PREVIOUS OMS CONDITION ASSESSMENTS

4.1 INSPECTIONS UNDERTAKEN

The aim of a condition assessment is to establish current condition, to identify the risk of failure and to enable proactive interventions to be undertaken to keep the risk of failure below tolerable levels.

4.1.1 SUMMARY OF INSPECTIONS

The following inspections were undertaken on the OMS in the 11 years prior to the collapse:

— 2012 – CCTV, sonar, and laser profiling

— 2019 – CCTV, sonar, and laser profiling, but profiling data was not acquired

A description of the inspection techniques used, the information they provide, and their limitations is provided in Section 7.5.4.

The OMS had been cleaned with a plough prior to these inspections being undertaken, refer to Section 7.2.2.1 for more information.

The inspection data for the pipe assets shown in Table 4:1 was reviewed by WSP in the preparation of this report. These cover 1,600m of the OMS from manhole ORM018 which is located adjacent to Albert Park through to manhole ORM013 which is in Takutai Reserve. The failure occurred in the pipeline section between manhole ORM016 and ORM015 which is highlighted in bold in the table.

Table 4:1 CCTV Inspections and Laser & Sonar Profiling Reviewed

Asset no.	Inspection year 2019		Inspection year 2012	
	CCTV	Laser	CCTV	Laser
10007054 (ORM018-OMR017)	Yes	Yes	Yes	Yes
10007053 (ORM017-ORM016)	Yes	Yes	Yes	Yes
10007052 (ORM016-ORM015)	Yes	Yes	Yes	Yes
10007051 (ORM015-ORM014J)	Yes	Yes	Yes	Yes
10007050 (ORM014J-ORM014)	Yes	Yes	Yes	Yes
10007049 (ORM014-ORM013J)	Yes	Not available	Yes	Yes
10099260 (ORM013J-ORM013)	Yes	Not available	No	Not available

After the collapse the following inspections were undertaken:

- Analysis of the sonar and laser profiling completed in 2019
- Drone inspection of the section of pipe between the collapse and towards downstream manhole 14J (Pipe 10007052 part and Pipe 10007051 part)
- Visual inspection of the collapsed pipe after it had been exposed to undertake repairs.

4.1.2 CCTV INSPECTION AND PROFILING (CONDITION IN 2012 & 2019)

As part of the preparation of this report, WSP undertook an independent assessment of the CCTV inspection and laser & sonar profiling completed in 2012 and 2019 . This included reviewing the profiling reports from 2019 which were not procured at the time and only became available after the collapse had occurred. Refer to Appendix F for further details. WSP's key observations were:

- Both lots of CCTV inspections were of poor quality which limited the identification of defects and assessment of defect severity. The CCTV footage was of low resolution and the camera could only look straight ahead. The camera could not be turned towards the wall to show a better view of defects
- Condition Structural Grades[4] were assigned by WSP from the CCTV to provide an initial indication of the pipe condition as it was when the inspections were undertaken, i.e. 2012 & 2019. Defects were scored according to the criteria defined in the Conduit Inspection Reporting Code of Australia and structural grades assigned. The 2012 CCTV inspection indicated that two pipe sections were in Structural Grade 4, i.e. 100007050 & 10007051. The other three pipes were assigned Grade 5. The 2019 CCTV inspection indicated that all pipes were Structural Grade 5
- Common defects observed were exposed aggregate on the obvert and the cast in-situ base (indicating corrosion), encrustations (indicating that water was infiltrating between the blocks), missing mortar, predominately near the horizontal joint between the blocks and the concrete base
- Missing mortar was particularly prevalent on pipes 100007050, 10007051 & 10007052. In these pipes the number of locations where mortar was missing increased from one location every 2-5m in 2012 to a location every 1.5-2.5m in 2019
- The extent of corrosion observed in the 2012 laser profiling reports varied across the pipes inspected. No corrosion was identified on pipes 100007050 & 10007051. Isolated corrosion to a maximum depth of 29mm was observed on pipes 10007049 & 10007052. More extensive corrosion was observed at the obvert of pipes 10007053 & 10007054 to a maximum depth of 81mm
- The depth and extent of corrosion increased markedly between 2012 & 2019. The maximum depth of corrosion on pipes 100007050 & 10007051 was 54mm and extended across the length of the obvert (whereas no corrosion was observed in 2012). 20mm of additional corrosion was observed on pipes 10007053 & 10007054 to a maximum of 101mm

[4] Condition Structural Grades range from 1 (very good) to 5 (very poor) as per the IIMM (IPWEA, 2020)

- More extensive corrosion was observed on pipe 10007052 which is the pipeline section that failed in 2023. 125mm of corrosion was observed over a short section (less than 1m) near the location of the collapse. Up to 61mm of corrosion was observed across the obvert whereas in 2012 only isolated sections of corrosion were observed.

4.1.3 DRONE INSPECTION (AFTER COLLAPSE)

Two of the key observations from the post-collapse drone inspection were:

- The pipe downstream of the collapse was observed to be in noticeable worse condition than in the 2019 CCTV inspection. Blocks were missing adjacent to manhole 15. There was also a block missing part way between manhole 15 and the collapsed section. Exposed aggregate is clearly visible in the block arch and the concrete base.

  It is likely that the hydro- jetting undertaken to clear the blockage after the collapse caused most of the damage noted. However, some of the damage may occurred between the 2019 inspection and the collapse.
- The section of pipe between manhole 15 and 14 appeared to be in broadly similar condition to that observed in the 2019 CCTV. Noting that only part of this section of pipe was inspected.

Refer to the technical memo in Appendix G for further observations from this footage.

4.1.4 VISUAL INSPECTION OF COLLAPSED SECTION

Key observations of the collapsed section of the OMS from November and December 2023 are:

- It appeared that two types of blocks had been used to construct the OMS at the location that collapsed. Most of the blocks appear to be at least as competent as those observed during the investigations for the Stanley St Rehabilitation undertaken in 2014 (although compressive testing has not been undertaken). However other blocks were very weak and crumbled when touched. Refer to Section 4.2.2. for commentary on why these blocks might have been present
- The blocks constructed from the more competent concrete were approximately 130mm thick, i.e. about 100mm less than the 9inch (228mm) thick blocks shown on the drawings (Appendix C). There was a layer of weak corroded concrete on the inside of the blocks about 30mm thick
- The outside of the block arch was covered by a layer of concrete which was not shown on the drawings, see Figure 4:1. The layer is between 50mm and 120mm thick at the centre of the arch, but less than 30mm thick at the edges where the arch meets the cast in-situ concrete base, see Figure 4:2.

For further observations, refer to the technical memo in Appendix H.

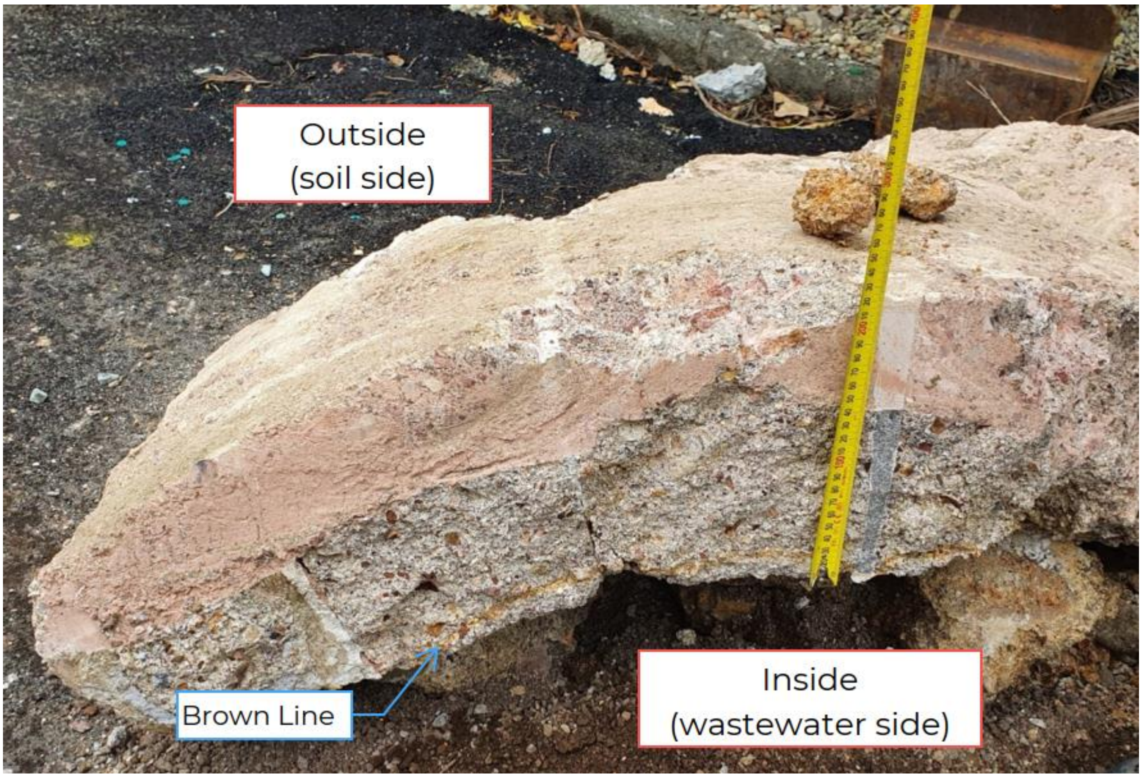

Figure 4:1 Typical block arch section after removal. Note the brown line indicating the extent of corrosion on the inside face

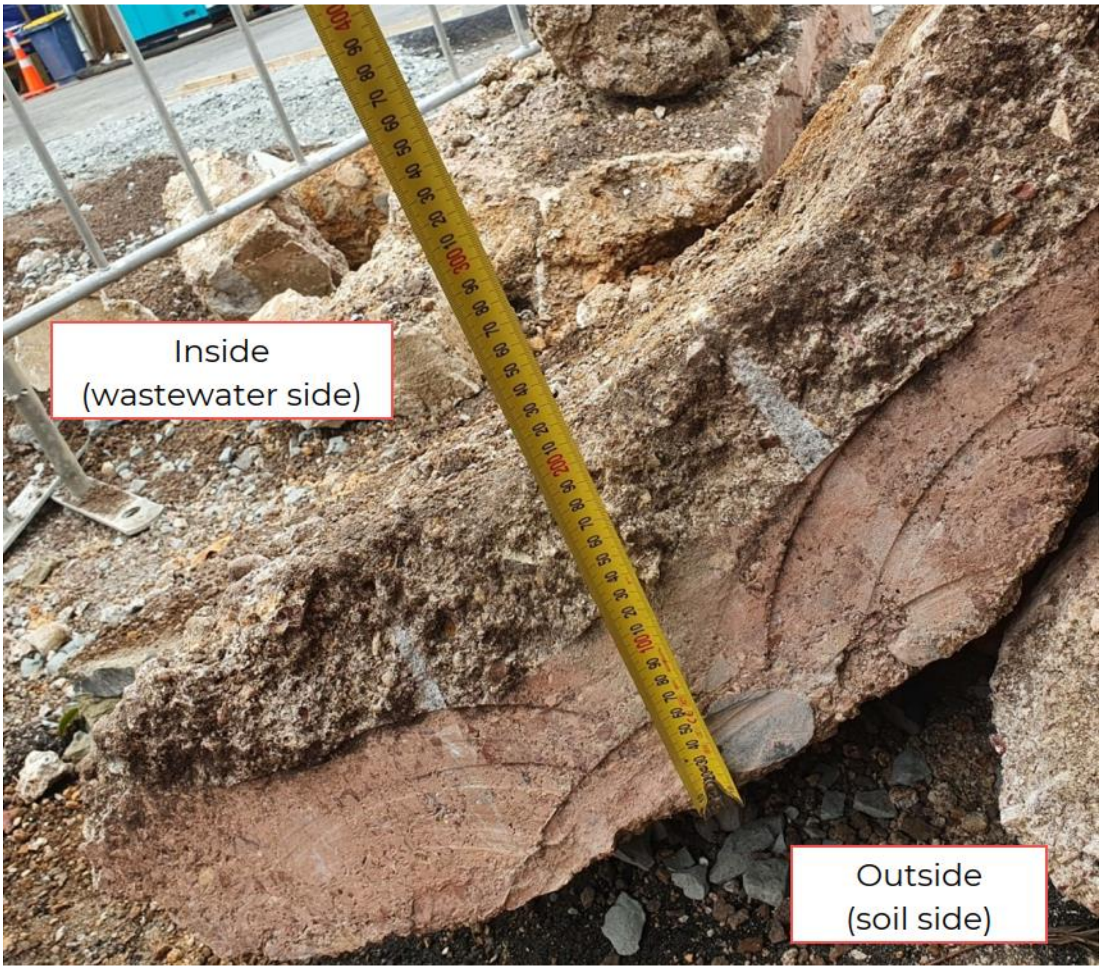

Figure 4:2 Weak blockwork after removal

4.2 COMMENTARY

4.2.1 CONDITION OF SEWERS

The condition grades assigned following WSP's review of the 2012 CCTV inspections (Grades 4 & 5) indicate that significant deterioration had occurred prior to 2012. For these grades the Conduit Inspection Reporting Code of Australia recommends that immediate action be taken to further investigate condition and assess risk (WSAA, 2020). This would involve activities like those undertaken in 2014 to investigate the section of OMS in Stanley St, i.e. walkthrough inspection, coring, and testing of pipe wall.

Corrosion increased between 2012 and 2019. This could not easily have been identified from the CCTV inspections but would have been apparent had the laser profiling undertaken in 2019 been analysed at the time and compared to the previous inspection. The increased corrosion would have been further cause to assess the risk of failure in more depth.

It appears from the drone footage undertaken after the collapse that the OMS has deteriorated further since 2019, particularly the section immediately downstream of the collapse. This is most likely due to the hydro- jetting undertaken to clear the blockage after the collapse.

4.2.2 BLOCK ARCH

Blockwork retrieved from adjacent to the collapse had variable strength. This is likely related to the original materials used for construction.

It is unclear why some of the blockwork was constructed from very weak concrete. One explanation is that when the OMS was constructed this location was used an access point for building the tunnel. When construction was completed, equipment was brought out through the opening and the arch formed from the outside. The weak section might have been a make-up piece constructed onsite to enable the arch to be closed. This has not been able to be confirmed, but it is plausible given that at the time the OMS was constructed the area was a natural low point.

The weak blocks could not have been identified from the CCTV and laser & sonar profiling. They might have been discovered if core samples had been taken as part of more in-depth investigation. However, it is quite possible that they would not have been picked up given they only cover a few metres of a pipeline that is 118m long, i.e. length of pipeline asset between manholes ORM016 & ORM015.

5 SINKHOLES AND SEWER FAILURES

5.1 SINKHOLES AND HOW THEY FORM

A sinkhole is a depression or hole in the ground caused by some form of collapse of the surface layer (Wikipedia, 2023). Sinkholes occur when an underground cavity forms and grows large enough to break through the ground surface, or the ground becomes unstable and collapses into the cavity. Sinkholes can form gradually or suddenly.

Sinkholes form naturally through (or a combination of):

— chemical dissolution of rock, e.g. in the case of the Waitomo where caves have been formed by the limestone substrate being dissolved by groundwater.

— erosion of soil by surface water, e.g. sea caves formed by wave action.

— erosion of soil by groundwater.

The groundwater memo in Appendix I discusses in more detail the formation of sinkholes through erosion of soil by groundwater. The likelihood of sinkholes forming is a function of the amount of groundwater movement and the nature of the soil, e.g. fine-grained soils like silt and clay are more prone to the development of sinkholes.

Human activities can accelerate the formation of sinkholes through erosion by groundwater increasing the movement of water, e.g. mining tunnels can cause sinkholes by providing a water path that increases the downward movement of groundwater, leaching out the ground above.

Underground pipelines can also increase movement of groundwater, accelerating the formation of sinkholes, through:

— groundwater leaking into pipes (infiltration) through cracks or other defects

— the fluid being conveyed by the pipe escaping (exfiltration)

— groundwater moving along the outside of pipes, e.g. along the pipe bedding or through voids around the pipe left during construction.

5.2 FAILURE MECHANISMS OF BLOCK SEWERS

The OMS consists of two components as described in Section 0 i.e. a concrete block arch and a cast in-situ concrete base. It is the concrete block arch that has failed at St Georges Bay Rd. This section describes the generic failure mechanisms and failure modes for block arch structures. Section 6 describes the specific factors that are likely to have led to the failure of the OMS.

5.2.1 ARCH STRUCTURES

Arches are compressive, self-supporting structures that are stabilised by the force of gravity pushing the blocks together. This makes them very stable, and they can support relatively large loads.

However, arches can fail in buckling if loads are not evenly balanced. For example, a sinkhole in the soil above one side of an arch could result in uneven loads being applied.

Also, arches are not able to support tensile loads. Upward forces applied to the underside of an arch may cause the blocks to separate. This could occur when a sewer is surcharged (as was possibly the case for the OMS during the January 2023 storm) if upward tensile forces exceed downward soil, groundwater, and live loads.

Arches formed from keystone (wedge shaped) blocks tend to be more stable than arches constructed from rectangle blocks (as appears the case in OMS). The wedge shape anchors the blocks together. Whereas rectangular blocks rely on the cement mortar to hold the blocks together. Degradation of the mortar can result in block displacement.

5.2.2 DETERIORATION OF BLOCK SEWER

As block sewers age the concrete blocks and the mortar between the blocks corrodes and weakens. The rate of deterioration is influenced by factors such as:

- Hydrogen sulfide (H_2S) – H_2S gas is produced in sewers under anaerobic conditions by sulfate-reducing bacteria. It is not corrosive to concrete. However, H_2S in the headspace above the sewer flow is then oxidized into sulfuric acid by microorganisms on the concrete surface, leading to concrete corrosion. The formation of H_2S in wastewater and its release is influenced by factors such as flow velocity, turbulence of the sewer flow, temperature, and detention time
- Strength of the concrete originally installed - Low strength or poorly placed concrete is more susceptible to corrosion.

The weakened concrete blocks and mortar may be further impacted by erosion caused by:

- Flow of wastewater in the pipe
- Infiltration and exfiltration passing through joints and cracks in the pipe wall
- Tree roots growing through cracks and joints.

Once the arch is weakened it may no longer be able to support soil and other loads. The load carrying capacity of the arch can be further reduced leading to collapse if:

- Blocks are displaced e.g. because of weakened mortar joints, due to tree roots putting pressure on blocks, or blocks are porous and become heavy from groundwater
- Voids are present above the arch causing uneven loading.

Other factors that can increase the likelihood of sewers collapsing include:

- Depth, i.e. increased soil loading
- Depth below groundwater, i.e. increased possibility of infiltration
- Ground type which impacts the loads that are transferred from the soil to the sewer and susceptibility of the soil to erosion and the formation of voids
- Ground movement e.g. shaking from earthquakes or settlement of underlying ground.

As a result of these factors there can be significant variance in the way that individual sewers behave and their rate of deterioration. Also, deterioration may not always be linear, and pipes can experience periods of rapid deterioration.

6 LIKELY FAILURE MECHANISM & POTENTIAL CONDITION OF OTHER SECTIONS OF THE OMS

6.1 LIKELY FAILURE MECHANISM

Based on the factors discussed in the previous sections it is likely that the failure was caused by the following factors acting together:

- General deterioration and weakening of the sewer over the 110 years that the OMS has been in service
- Exceptionally wet weather in 2023. The year was the wettest on record for Auckland. In the 9 months leading up to the collapse 50% more rain fell than the average for an entire year. Significant rainfall events include the Auckland Anniversary floods, Cyclone Gabrielle and heavy rainfall on 9 May 2023
- Weak blockwork at the location that collapsed. Some of the blocks retrieved after the collapse were a lot weaker than others. They had very little strength and crumbled when disturbed.

The likely failure mechanism of the OMS is:

- Deterioration – the block arch section of the OMS was slowly weakened over the 110 years since it was installed through a combination of corrosion and erosion of the internal surface.
  - Deterioration was observed from the 2012 & 2019 inspections and the section of pipe in Stanley St (that was rehabilitated in 2014).
- High sewer flows during the storms in January and February 2023 resulted in:
  - Further and relatively rapid erosion of the deteriorated concrete blocks and mortar joints
  - Wastewater seeping out into the surrounding soils through defects in the sewer pipe contributing to physical scouring of the soil above the pipe.
- Groundwater movement
  - Groundwater levels would have risen notably in January 2023 in response to the heavy rainfall and the surface flooding in the vicinity
  - When wastewater flows in the OMS receded, there would have been a head difference between the ambient groundwater level and the wastewater in the pipe. This likely led to increased seepage of groundwater into the pipe (infiltration). Fine sediment from the soil above the pipe could have been eroded, starting the formation of the sinkhole cavity, or increasing any existing cavities
  - The formation of cavities would have stabilised when the groundwater level dropped below the level of the cavities. However, the cavities would continue to grow when the process was repeated following the second and subsequent storms.

- **Excavation for installation of power cable to transformer** – this could have disturbed the sinkhole cavity further, causing it to grow and be visible at the ground surface. There is no indication, however, that the works triggered the formation of the sinkhole nor damaged the OMS
- **The sinkhole caused uneven loading on the block arch** which it was not able to withstand and eventually it failed under buckling.

The weaker blocks observed near the failed section most likely contributed to the sinkhole and collapse as they were:

- More susceptible to being eroded and dislodged by the high sewer flows in January and February 2023 which would have increased flow of groundwater and soil into the OMS and hastened the formation of the sinkhole
- Less able to withstand the uneven loading applied to the arch section of the OMS due to the sinkhole.

The very wet weather experienced in 2023 also contributed to the collapse by:

- Increasing wastewater flows in the OMS, causing the block arch to be fully exposed to flows which accelerated erosion of the blockwork and mortar
- Causing flooding above the OMS which increased the hydrostatic loading on the pipe. This increased flow of sediment ladened groundwater into the OMS starting the formation of the sinkhole cavity or increasing any existing cavities
- Repeated rain events causing fluctuations in groundwater loading which would have hastened the formation of the sinkhole.

Whilst it is possible that the sinkhole was caused by groundwater running along the outside of the OMS, the signs of infiltration observed from CCTV inspections indicate that groundwater seeping into the OMS is the more likely cause.

6.2 POTENTIAL CONDITION OF OTHER SECTIONS OF THE OMS

Review of the condition assessment covered in Section 4 indicates that the other sections of the OMS that were assessed (i.e. 1.6km from manhole ORM018 to ORM013) are in similar condition to the section that failed.

There are signs of deterioration due to corrosion and erosion. There are also indications that groundwater is infiltrating into the sewer, weakening joints, and providing a pathway for groundwater movement which is required for sinkholes to form.

It is therefore recommended that the section of OMS from ORM018 to ORM013 be scheduled for rehabilitation.

It is also possible that the remainder of the OMS is in similar condition.

It is therefore recommended that the following be undertaken to prioritise rehabilitation works on the portion of OMS that has been assessed and to determine the risk of failure for the remainder of the OMS:

- Complete CCTV and laser & sonar profiling to identify defects and determine the extent of deterioration
- Identify low points where tunnel portals for the construction of the OMS might have been located as there might be weak blockwork at these locations
- Undertake more in-depth condition assessment at low points and where there are concerns about the level of deterioration to determine the strength of blockwork. This could involve walkthrough inspection, coring, and testing of pipe wall
- Consider undertaking structural analysis of pipelines using finite element analysis to improve assessment of the likelihood of failure.

7 MAINTENANCE AND CONDITION ASSESSMENTS

This section covers:

— **Introduction**, outlining the role of condition assessment and the key steps for managing the condition of wastewater pipelines

— **Condition Inspection Techniques**, describing the techniques used by Watercare, outlining other available techniques, and commenting on the appropriateness of the inspections undertaken on the OMS

— **Condition Assessment Approach**, summarising Watercare's relevant asset management practices and how these are applied to the maintenance and condition assessment of Wastewater Transmission pipelines. Commentary is provided on the appropriateness of these practices.

7.1 INTRODUCTION

7.1.1 ROLE OF CONDITION ASSESSESSMENT

The aim of a condition assessment is to establish current condition, to identify the risk of failure and to enable proactive interventions to be undertaken to keep the risk of failure below tolerable levels.

In Section 5.2.2 it was noted that there are many factors that contribute to the deterioration of sewers which results in there being significant variation in the way that individual sewers behave and the rate at which they deteriorate. Deterioration is not always linear, and events can occur that can cause condition to rapidly decline.

Therefore, to target interventions appropriately it is important to understand the condition of individual assets.

7.1.2 KEY MANAGEMENT STEPS

The key steps for managing sewer assets are set out in Figure 7:1. These also apply to management of most other infrastructure assets. The steps are:

— Set priority for management/maintenance based on failure impact assessment

— Undertake condition inspection and assessment to determine actual state of assets

— Decide and undertake next steps which could involve scheduling future inspections, undertaking maintenance or renewing/rehabilitating assets.

Condition assessment does not alter the need to undertake maintenance or renewal/replacement works, rather it facilitates the mitigation of risk by helping to identify assets in need of maintenance or replacement so that works can be undertaken before they fail.

In some cases, it may be appropriate to skip condition assessment, going straight to rehabilitation or maintenance, e.g. if there is a high likelihood that the asset is in poor condition or if the consequence of failure justifies precautionary intervention.

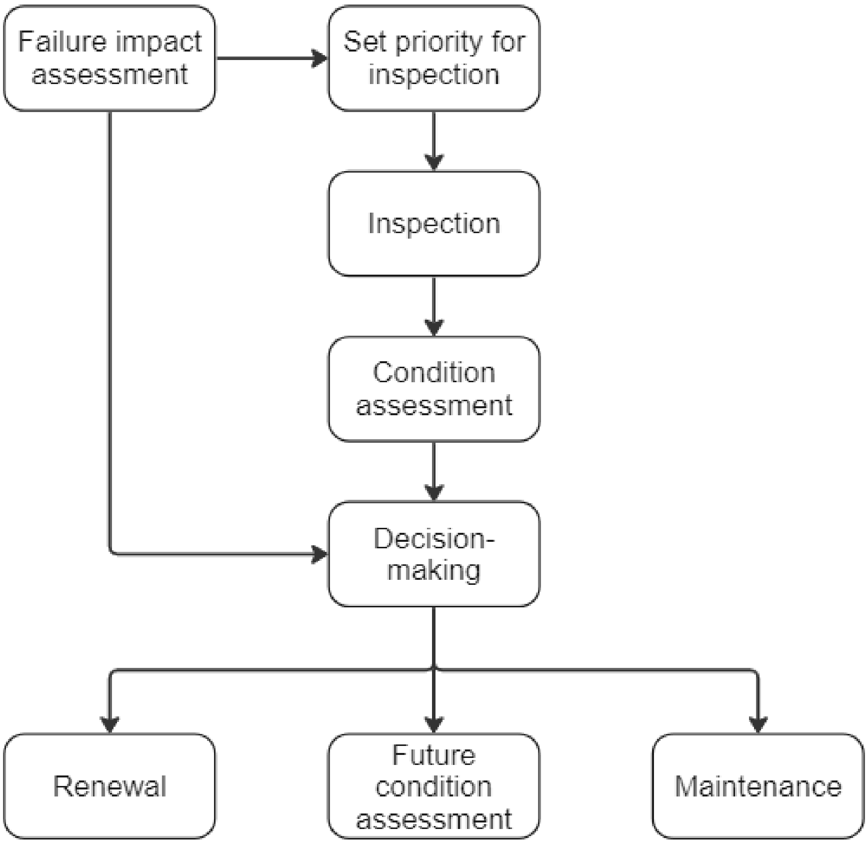

Figure 7:1 Steps for managing sewer assets (Zhao, McDonald, & Kleiner, 2001)

7.2 CONDITION INSPECTIONS

7.2.1 INSPECTIONS UNDERTAKEN ON TRANSMISSION PIPELINES

Watercare's standard practice is to inspect wastewater transmission pipelines every 5 years. More frequent inspections are undertaken on pipelines where there are concerns about condition.

Two sets of condition inspections have been undertaken on the OMS over the last 11 years, i.e. in 2012 and 2019. We note that these inspections occurred 7 years apart, rather than the standard 5-year interval. Refer to Section 4 and Appendix F for details of the inspections reviewed by WSP in the preparation of this report.

7.2.2 EXPLANATION OF METHODS USED

7.2.2.1 CLEANING

Prior to 2017 the OMS was cleaned with a plough prior to inspection.

As the plough moves down the sewer, a head of wastewater builds up behind it. Wastewater jets from a small aperture at its base. The jet of wastewater and turbulence from excess wastewater spilling over the plough scours debris downstream of the plough which is washed into the downstream grit chamber where it can be removed.

Cleaning makes it easier to identify defects when condition inspections are undertaken and provides the opportunity for the personnel who are required to travel down the sewer with the plough to observe the condition of the sewer and report on issues.

However, it presents health and safety concerns as staff are required to enter the confined space of the sewer where they could be exposed to hazardous conditions and rescue would be difficult. For these reasons use of the plough was stopped in 2017.

Watercare are currently reviewing practices for safe operation of the plough. Prior to the St Georges Bay Rd incident, they were planning to recommence ploughing within the next year, subject to being able address health and safety issues.

7.2.2.2 CONDITION INSPECTIONS ROUTINELY USED BY WATERCARE

Inspection typically involves CCTV inspection and laser & sonar profiling using an inspection unit like that shown in Figure 7:2.

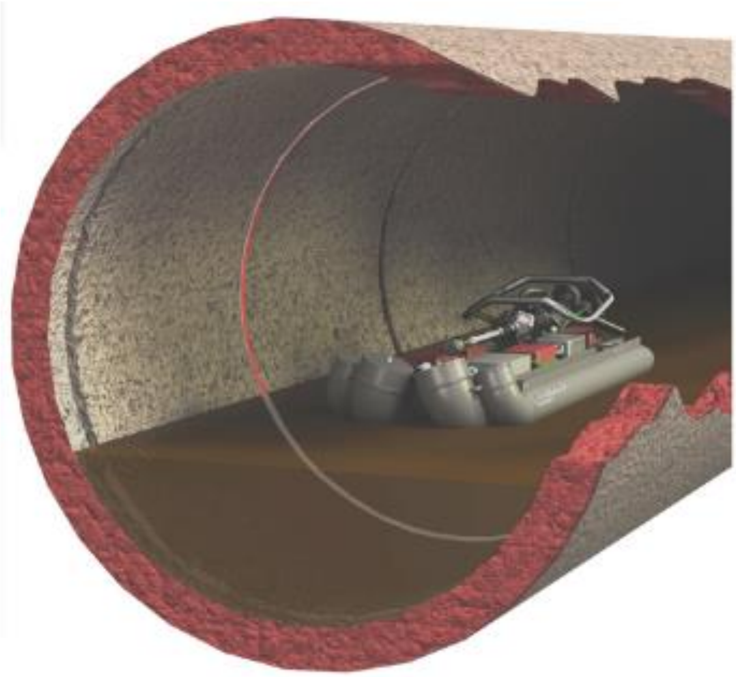

Figure 7:2 Pipe inspection unit (Kawasmi & Brown, 2019)

CCTV inspection provides a video showing the inside of the pipeline. Laser profiling measures the internal surface of the pipe above the waterline which is compared against a reference shape to quantify the depth of corrosion or protrusions. Sonar profiling measures the pipe surface below the waterline to quantify debris. These outputs are merged into an output like that shown in Figure 7:3.

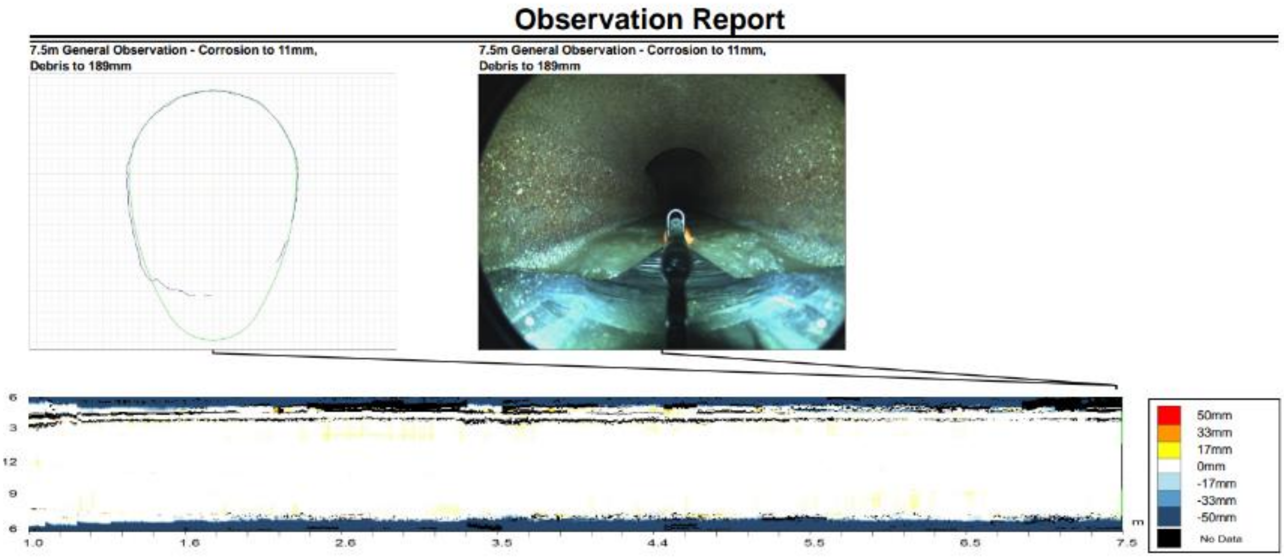

Figure 7:3 Pipe inspection observation report example (from the 2012 inspection of the OMS, supplied by WSL)

CCTV logsheets are also prepared in accordance with the 3rd Edition of the New Zealand Pipe Inspection Manual to record the defects and features observed (Water NZ, 2006). However, the logsheets prepared for the OMS do not record condition scores or condition grades, as is normal practice.

Also defects in brick sewers are not addressed well in the 3rd Edition of the New Zealand Pipe Inspection Manual. This has been improved to a degree in the 4th Edition of the Manual which was issued in 2021 (Water NZ, 2021) and in the Conduit Inspection Reporting Code of Australia (WSAA, 2020).

The practice over the last five years or so has been to undertake laser profiling and sonar profiling inspections but to only analyse the data to generate reports like those shown in Figure 7:3 if an issue is identified from the CCTV inspection. This practice risks overlooking corrosion of the pipeline wall as corrosion cannot easily be identified from CCTV alone.

The combination of CCTV inspection and laser and sonar profiling undertaken by Watercare on sewer transmission pipelines offers the following advantages:

— Provides a visual representation of the pipeline above the waterline

— Measures the internal dimensions of the pipeline and gives an indication of the extent of erosion in the pipe wall and of other defects.

However, it has the following limitations:

— The quality of the CCTV image can be poor, particularly in large pipes, which makes the identification of defects difficult. Refer Figure 7:4 for an example of CCTV image quality

— The CCTV camera only looks straight ahead. It is not possible to pan the camera towards the wall and zoom in on potential defects to get a better view

— Resolution of laser profiling measurements is not fine enough to identify corrosion of the mortar between the blocks

— Provides information on the inside of the pipe wall only. Information on the wall itself or what is behind the wall isn't provided. (This limitation applies to all CCTV inspections not just the units used for the Watercare inspections).

Recent upgrades by the manufacturer of the inspection units have attempted to address the first two points. The resolution of the image captured has been improved and the units now have multiple cameras, with cameras pointing at the pipe wall as well as straight ahead.

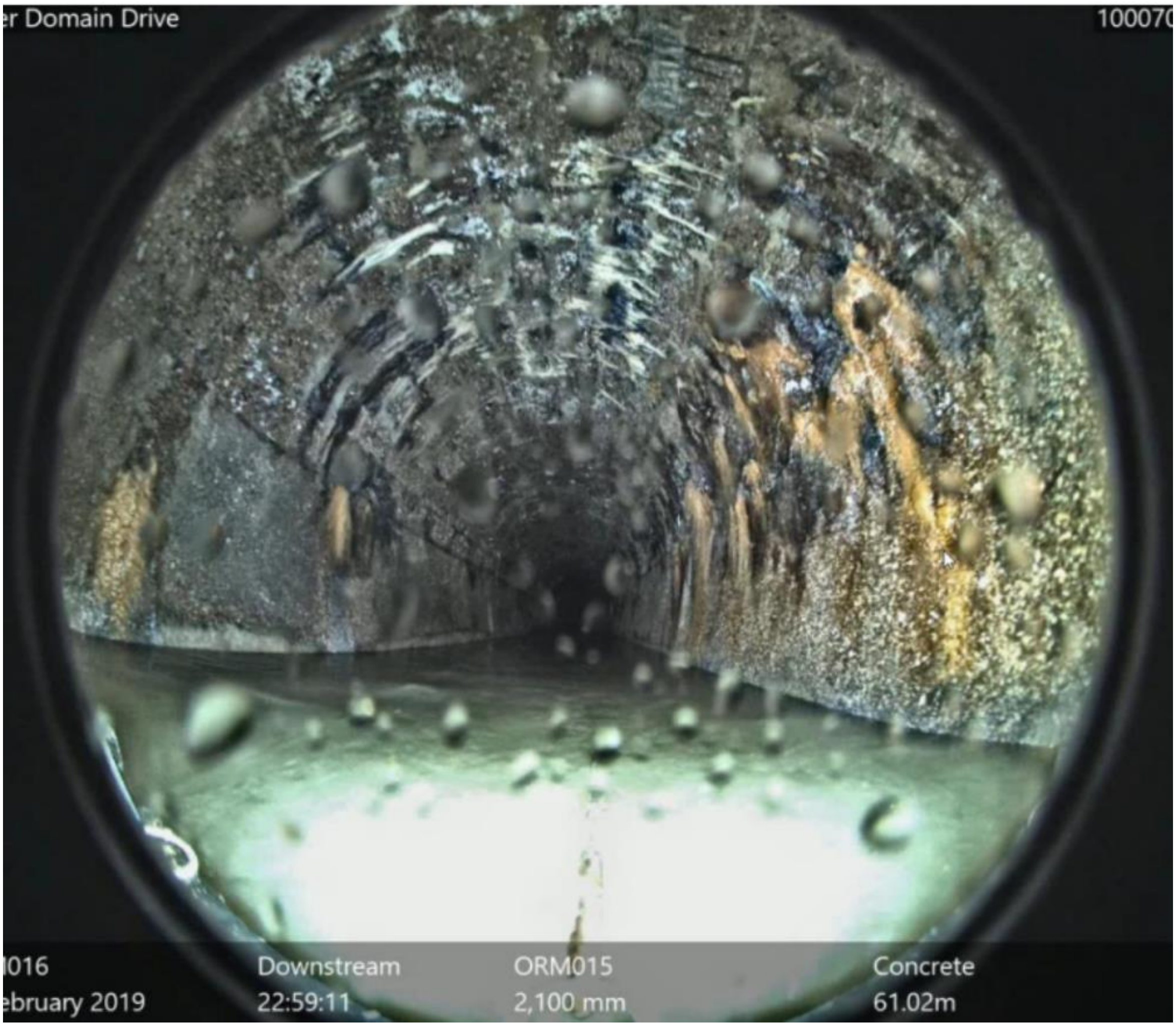

Figure 7:4 Sample CCTV output from OMS in 2019, shows the location near the collapse (supplied by WSL)

7.2.2.3 OTHER CONDITION INSPECTION TECHNIQUES

A summary of other potential condition assessment techniques is provided in Appendix J. This information was derived from the 'Guidelines for Condition Assessment and Rehabilitation of Large Sewers' (Zhao, McDonald, & Kleiner, 2001) and 'New Zealand Gravity Pipe Inspection Manual 4th edition' (Water NZ, 2021).

An emerging technology is the use of drones for the inspection of pipelines. This can be quicker and cheaper than traditional CCTV. However, CCTV and drone inspections only view the inside of the pipe wall.

There are limited techniques available for assessing the condition, strength and thickness of the pipe wall itself or for identifying voids behind the pipe wall. Intrusive techniques such as coring are generally required. These require staff to enter pipes which poses health and safety issues. Also, the testing itself can compromise the condition of the pipeline.

Non-intrusive techniques for assessing the pipe wall and identifying voids are being developed but are these are still in the experimental stage and not widely available. Current best practice is therefore to screen pipes using CCTV inspection, and potentially laser and sonar profiling, to identify areas of potential concern. Then to investigate areas of concern in more detail.

Sydney Water's condition assessment of the Bondi Ocean Outfall Sewer (BOOS) is an example of this approach. The BOOS is like the OMS being 2.4 metres high and constructed from bricks and concrete in an oviform. It was constructed in 1889 i.e. 20 years before the OMS. The following was undertaken:

- CCTV inspection and laser and sonar profiling was undertaken, and scans of the brick reviewed in accordance with Water Services Association of Australia (WSAA) conduit inspection code (WSAA, 2020). Key structural features that were relevant to the condition assessment were cracking, deformation, missing mortar, brick separation and missing bricks
- Laser profiles from previous inspections were also compared to estimate the extent of oviform deformation. Geotechnical records were reviewed to better understand the groundwater profile
- A structural assessment using finite element analysis (FEA) was then undertaken with assumptions for brick strength and ground conditions to assess the likelihood of failure and to determine whether renewal was appropriate.

7.2.2.4 COMMENTARY ON WATERCARE'S APPROACH

Watercare have been an early adopter of inspections using a combination of CCTV inspection and laser and sonar profiling for inspection of transmission pipelines. This reflects good practice, being aligned to practices adopted by other large utility providers. The approach provides a visual representation of the pipeline above the waterline and measures the internal dimensions of the pipeline enabling the extent of corrosion to be quantified.

Watercare's practice of inspecting transmission pipelines on 5-year cycle, with more frequent inspections if concerns are identified, is also in line with the approach used by other utilities.

However, the following improvements are recommended to reflect best practice:

- Timing of inspections
  - Undertake condition inspections after events that could trigger rapid decline in condition, e.g. after large storms
  - Improve the quality and resolution of the CCTV inspections to provide a clearer view of the pipe wall and aid the identification of faults. Either use a CCTV camera that can pan towards the pipe wall and zoom into issues. Or the latest CCTV and laser & sonar profiling units which have multiple cameras that look both straight ahead and towards the pipe wall.
- Inspection Practices
  - Reinstate cleaning of the OMS using the plough, subject to addressing health and safety concerns. This will improve the identification of defects as the personnel who are required to travel down the sewer with the plough will also be able to observe the condition of the sewer and report on issues. Watercare have already started investigating reinstating the use of the plough. It is also worth investigating whether suitable camera equipment could be installed to allow unmanned inspections while cleaning the line with the plough

    - Produce detailed CCTV logsheets to record inspections. Assign Structural Condition Grades to provide an initial grading of condition and to identify the need for more intensive condition assessment and likelihood of failure analysis. Use a logging and grading system better suited to assessing defects in brick pipelines, e.g. the 4th Edition of the New Zealand Pipe Inspection Manual or the Conduit Inspection Reporting Code of Australia, both of which were published since the last inspections were undertaken on the OMS.
- Analysis of Condition Assessment
    - Change standard practice so that laser and sonar profiling inspections are analysed as a matter of course as opposed to only analysing profiling data if issues are identified from the CCTV inspection. This will improve the likelihood of identifying faults not picked up from the CCTV inspection
    - Compare laser profiling against previous inspections to determine the progress of corrosion as increased corrosion could signal a trigger for renewal.

7.3 CONDITION ASSESSMENT APPROACH

7.3.1 WATERCARE'S RELEVANT ASSET MANAGEMENT PRACTICES

Watercare's approach to the maintenance and condition assessment of Wastewater Transmission Pipelines is a component of their overall asset management system. As such, it should be considered in the context of the overall system. Watercare's approach to asset management is set out in the Asset Management Policy (WSL, 2023b) and Asset Management Maintenance Manual (WSL, 2023c). These documents are supplemented by Watercare's Wastewater Transmission Operations Manual (WSL, 2017).

7.3.1.1 ASSET MANAGEMENT POLICY

The Policy is a high-level document that describes "how good practice asset management contributes to Watercare's corporate priorities, including giving effect to Te Mana o te Wai" (WSL, 2023b).

The policy sets out Watercare's commitment to delivering on the following Asset Management Principles:

- Zero harm: Public water systems will not cause harm to members of the public, our staff, or service providers
- Delivering customer outcomes: We deliver valued services to customers that are safe, reliable, and affordable
- Meeting demand: Our networks and operations will efficiently meet future demand for water services
- Protecting our environment: Minimise the environmental impact of our assets and our operations, in a manner consistent with Te Mana o te Wai
- Becoming resilient: Our networks and operations are resilient, including to the increasing impacts of climate change.

7.3.1.2 ASSET MANAGEMENT MAINTENANCE MANUAL

The Asset Management Maintenance Manual "sets out important information and guidance for Watercare asset management" covering aspects such as standards, tools, and systems for asset management (WSL, 2023c). It defines corporate level asset management practices to be adopted across all asset classes as well as providing practices for management of some specific asset classes. Management of wastewater pipes is not covered specifically but the corporate level practices are applicable.

The content of the Manual is aligned to international best practice. It references international standards for asset management, service planning and collection and exchange of reliability and maintenance data. It contains content aligned to these standards.

The Manual adopts the principles of Reliability Centred Maintenance (RCM). Maintenance activities are prioritised based on the risk of failure, i.e. the criticality of assets and their likelihood of failure. Generic process for assigning criticality grades and assessing the likelihood of failure are defined.

7.3.1.3 TRANSMISSION OPERATIONS MANUAL

The Transmission Operations Manual sets out tasks and procedures to be performed to operate the Wastewater Transmission System (WSL, 2017).

The Transmission Operations Manual is aligned to the Asset Management Maintenance Manual. It recognises:

- The importance of pipeline asset condition and risk determination, stating that "a good understanding of the level of risk associated with the structural or operational state of pipes and related structures allows for timely intervention as required"
- The asset condition assessment programme is structured based on criticality
- Criticality grades have been assigned to individual wastewater transmission pump station and sewer assets
- Gravity sewers greater than 1500mm diameter, which covers most transmission mains, are allocated the highest Criticality Grade, i.e. Grade 5
- Inspections are scheduled so there is an even distribution of workload over a 5-year period. Individual pipe inspections are scheduled on an annual basis
- A decision to adjust the frequency of pipe inspections may be made based on the most recent inspection if the pipe condition of the pipe is questionable
- Quarterly asset investigation meetings (AIMs) are held to the review the results of inspections and risk determinations. A key outcome of the AIMs is to make decisions regarding both maintenance requirements, frequency of future inspections and recommendations for rehabilitation/replacement.

7.3.2 BEST PRACTICE

7.3.2.1 RISK BASED ASSET MANAGEMENT

Best practice is to adopt a risk-based approach to asset management of pipe networks focusing on those pipes with the highest consequence and likelihood of failure. This is what Watercare are currently doing. It is recommended that they continue to do so as it is neither practical nor cost effective to manage all assets in the network the same due to the size of sewer networks. The Watercare Wastewater Network for example contains over 8,300km of pipes, 455km of which are transmission mains.

Figure 7:5 shows an extract from the New Zealand Wastewater Renewals Framework which illustrates the risk-based approach to the management of sewers (WSP for Quake Centre, 2018). Interventions, such as condition assessment, maintenance or renewals are prioritised based on the risk of failure, i.e. the combination of the consequence and likelihood of failure.

The intention of inspection and condition assessment is to identify the need for interventions to reduce the likelihood of failure so that the risk of failure remains below the maximum risk threshold.

Proactive interventions are undertaken on assets with a high consequence of failure, but assets with a low consequence of failure may be allowed to run to failure.

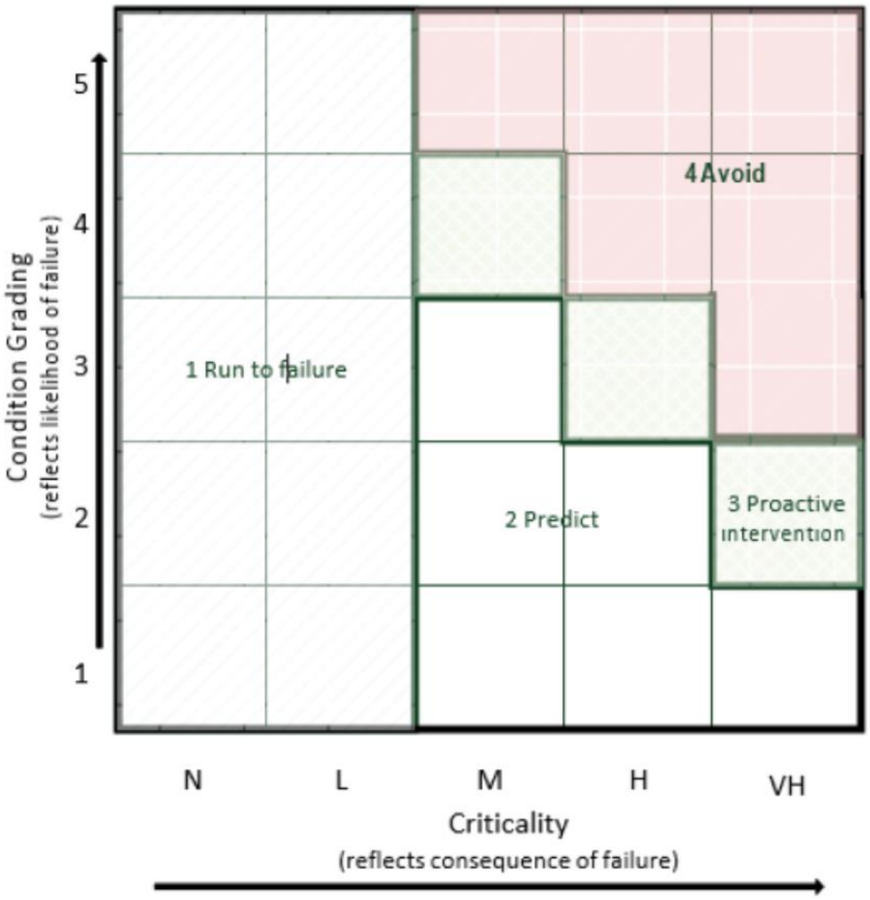

Figure 7:5 Example risk-based renewal matrix (WSP for Quake Centre, 2018)

As example of best practice, Wellington Water has established a Very High Criticality Assets framework which they use to prioritise the need for inspection and intervention. Each asset is assigned a criticality index which is an indicator of how critical the asset is, relative to other assets in the network, for delivering Wellington Water's Customer Service Goals. Asset management decisions such as the priority for undertaking interventions are based on this index. The process promotes transparency and enables a consistent comparison of assets and risks across the many asset types that make up Wellington Water's networks.

Watercare's Asset Management Maintenance Manual sets out a similar approach. However, the criticality of wastewater pipes is assigned solely on the size of the pipeline. It is recommended that

that the approach be updated to consider other aspects that might impact the consequence of failure or ease of repair, e.g. pipes under buildings, deep pipes, pipes near sensitive receiving environments.

7.3.2.2 DOCUMENTED PROCEDURES

A best practice management system includes documented procedures for condition assessment and maintenance management.

In practice, decisions on inspection and intervention may need to accommodate a mix of information that is well defined (e.g. cost, carbon and capacity), along with some information that is incomplete and some factors that are more loosely defined such as environmental, cultural and social impacts (IPWEA, 2020). These decisions can be made through a form of optimised decision making.

Documenting procedures and reasoning behind decisions plays an important role as it enables auditing of the decision quality and gives a starting point for refining the method over time to facilitate continuous improvement.

This an area that Watercare is improving.

Whilst the Transmission Operations Manual references other procedural documents, e.g. for undertaking condition inspections and determining risk scores, these documents were not able to be provided.

Most decisions are made by key staff based on their knowledge and experience which are then confirmed at the AIMs meetings. There is limited documentation describing the reasoning behind decisions.

A best practice management system would include a Condition Assessment Strategy which specifies:

- Condition inspections – the techniques to be used under various circumstances
- Timing of inspections, which depend on the consequence of failure, the previously observed condition of the asset and the forecast rate of deterioration
- Collection of other information that could influence the likelihood of failure, e.g. previous faults, presence of H2S, pipelines under OLFPs
- Processes for deriving likelihood of failure ranking from the condition inspections and other information collected
- Circumstances that may necessitate the need for additional inspections, e.g. after high flows that could cause rapid deterioration
- Trigger levels for undertaking more intensive investigations.

These procedures will feed into Interventions Strategy – that specifies the repairs and renewals to be undertaken and the urgency for undertaking these works based on the observed defects/condition and the consequence of failure. This is discussed in Section 8.3.3.

7.3.2.3 COMMENTARY ON WATERCARE'S APPROACH

Watercare are adopting a risk-based approach aligned to best practice. Wastewater Transmission Pipelines are classified as critical assets. They are regularly inspected and as far as practical they are proactively managed to avoid failures.

However, the following improvements are recommended to reflect best practice:

Consequence of Failure

— Update processes for determining the criticality of pipelines to include all factors that could impact the consequence of failure or ease of repair, e.g. pipes under buildings, deep pipes, pipes near sensitive receiving environments. This will highlight Transmission Mains that might warrant more frequent condition assessment or earlier renewal.

Likelihood of Failure

— Develop a Condition Assessment Strategy, as described in the previous section.

Watercare are in the process of improving the documentation of their processes in line with the recommendations above. They expect to be complete by July 2024.

8 RENEWAL PLANNING

This section covers:

— **Introduction**, outlining the purpose of renewals and the key steps for determining when to renew wastewater pipelines.

— **Renewals programme,** Watercare's planned renewal investments are summarised. Commentary is provided on Watercare's practices for prioritising wastewater pipelines for renewal.

— **Renewal Methods** – the methods currently used by Watercare are compared to typical worldwide industry best practice.

8.1 INTRODUCTION

8.1.1 PURPOSE OF RENEWALS

"Wastewater pipelines should be renewed when failures or the risk of failure can no longer be tolerated, and renewal is the most appropriate intervention" (WSP for Quake Centre, 2018).

For critical pipelines such as interceptor sewers the aim is generally to renew before failures occur because of the high consequence of failure. However, there will always be some residual risk of failure.

Renewals planning for critical assets should be underpinned by a robust programme to understand the condition of individual assets as there can be significant variation in the rate of deterioration of individual pipelines. Also, events may cause rapid deterioration of pipelines which might alter renewal priorities.

8.1.2 KEY MANAGEMENT STEPS

The key management steps for determining when to undertake renewals are the same as those set out in earlier in Figure 7:1, being.

— Determine the impact of failure i.e. consequence of failure

— Assess the condition to determine the likelihood of failure

— Intervene when the risk of failure (taking account of both the consequence and likelihood of failure) exceeds the maximum level that can be tolerated

— Determine the most appropriate intervention, i.e. repair only a section of the existing pipeline, replace with a new pipeline or rehabilitate the whole existing line.

Utilities are faced with a significant challenge when developing renewals programmes. If they renew assets too early, then they might be diverting funds away from investments in other works that will enhance environmental or community outcomes. If renewals are delayed too late, then the impacts of asset failure on the environment and communities could become unacceptable, and repair costs will increase (WSP for Quake Centre, 2018). Renewals planning must therefore be considered in the context of the overall asset management plan.

8.2 RENEWALS PROGRAMME

8.2.1 WATERCARE'S ASSET MANAGEMENT PLAN

Watercare's Asset Management Plan (AMP) sets out the programme of works that it intends to undertake over the 20-year period between 1 July 2021 to 30 June 2041.

Investment is targeted to provide the following beneficial outcomes:

- Catering for a growing Auckland
- Developing a resilient and diverse water system for tomorrow
- Protecting our environment
- Adapting to climate change impacts and reducing emissions
- Delivering value for money by running an efficient operation.

Over the next 20 years, Watercare intend to invest about $18.5 billion to build and maintain water and wastewater infrastructure. Over half of that amount (58%) will be invested into the wastewater network

Key Transmission Sewer Projects include:

- The 14.7km Central Interceptor Wastewater Tunnel to reduce overflows, help clean up local beaches and waterways and cater for growth
- The Northern Interceptor pipeline to divert flows currently treated by the Mangere WWTP to the Rosedale WWTP to support growth
- Construction of additional Transmission Sewers associated with the Western Isthmus Water Quality Improvement Programme to improve water quality in urban streams and the harbours.

These projects will also improve resilience of the wastewater system by diverting flows away from older sections of the network and improving operational flexibility. Watercare plan to invest $3.4b to improve wastewater catchments in the Auckland metropolitan area. The majority of this will be spent on improvements to transmission pipelines.

An additional $1.9b has been allocated to renew wastewater assets to improve system resilience. This includes replacing critical assets, such as transmission mains near the end of their useful lives and non-critical assets that have failed. $0.7b is planned to be spent in the next decade and a further $1.2b between 2031 and 2041.

The gross replacement cost for transmission wastewater pipelines is estimated to be $2.2b and $4.1b for local pipelines.

When establishing the investment programme set out in the AMP, Watercare have had to balance the desire to provide beneficial outcomes to its customers and the people of Auckland, the risk of failure of critical assets such as transmission sewer mains, against funding constraints due to debt restrictions and customer price impacts.

8.2.2 RENEWALS PROGRAMME

Watercare considers all transmission mains to be critical assets which they aim to proactively renew before failure.

In FY23 $10m was spent rehabilitating wastewater pipelines.

The renewals programme for the next 3 years is currently being developed. It is expected that there will be a significant uplift in renewals expenditure to circa $100m/yr. The majority of this will be invested into the renewal of transmission pipelines.

Practices for prioritizing renewals based on the consequence and likelihood of failure are being developed as part of this exercise.

8.2.2.1 RENEWAL OF THE OMS

Prior to the failure at St Georges Bay Rd, renewal of the OMS had been scheduled for the second tranche of work. Consideration was being given to starting its renewal in the next 3 years (expected before 2027), subject to budget constraints.

The intention was to first complete the Central Interceptor which will reduce the flows being conveyed by the OMS. This should provide more headroom for undertaking renewals and could potentially reduce deterioration of the OMS as it will flow full less often, reducing erosion to the arch section.

8.3 DISCUSSION OF WATERCARE'S RENEWALS PROGRAMME

8.3.1 GENERAL LEVEL OF INVESTMENT

When the Water Industry Commission for Scotland (WICS) reviewed Watercare's investment plan in 2020. They recommended that Watercare's investment to replace aging infrastructure be increased to 25% more than current levels to reduce ongoing maintenance and operations costs (WSL, 2021).

Watercare investment over the next 10 years is $827m lower than preferred and a further $1,338m of capital investment over the first four years has also been deferred. This includes deferral of a portion of planned network renewals, along with other improvements (WSL, 2021). Investment has been deferred to later than preferred due to its Auckland Council debt constraint.

8.3.2 WASTEWATER RENEWALS RATES

In recent years the rate of renewal of wastewater pipes has been low with only $10m being invested in FY2022, i.e. a renewal rate less than 0.1%/yr. However, the AMP forecasts a significant increase with $1.9b being allocated over the next 20 years.

This should enable 30% of the total network to be renewed (based on the gross replacement cost quoted in the AMP) and enable most of the wastewater pipelines installed prior to the 1960's to be renewed[5].

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[5] 36% of the wastewater network was installed before 1960.

It is likely that a greater percentage of transmission mains will be renewed due to Watercare's proactive approach to renewal of critical pipes. Also, the percentage of network renewed may be higher if a high proportion of pipes are rehabilitated rather than replaced.

The planned rate of renewal compares favourably with renewal rates from European Countries. Data collected from 2017 to 2019 show renewal rates for European Countries between 0.1%/yr and 1.5%/yr with a median of 0.4%/yr (EurEau, 2021). Compared to Watercare's planned rate of 1%/yr

8.3.3 PROCESSES FOR SELECTING SEWERS TO RENEW

Annual renewal rate, however, is not a good measure for comparison. It does not for example take into consideration the age of the respective networks. Even asset age is not the best indicator of asset health and the need for renewal because asset deterioration is dependent on a range of factors such as the interaction between soil type and pipe material and the availability of asset management interventions (NIC, 2023).

The practices used for identifying and prioritising renewal of individual pipelines are of more importance as these ensure that the most appropriate pipes are selected for renewal. Watercare are in the process of documenting these practices and using the practices developed to identify a prioritised list of pipelines for renewal. This is expected to be completed by July 2024.

As discussed in Section 7.3.2.1, best practice is risk based. For critical pipelines with high consequence of failure, such as wastewater transmission mains, the aim is to undertake renewals before failures occur. This requires a good understanding of the condition and likelihood of failure of individual pipelines, i.e. the condition assessment practices outlined in Section 7.

A best practice management system would also include a renewals intervention strategy that specifies the repairs and renewals to be undertaken and the urgency for undertaking these works based on the observed defects/condition and the consequence of failure.

Triggers for renewal would be established from analysis of individual pipelines, which for block or brick-built pipelines could include using finite element analysis to determine, for example, the extent of pipe wall deterioration that can be tolerated under various loading conditions. Sensitivity to uncertainties such as block strength and historic construction practices should also be considered. This approach was adopted by Sydney Water for renewal of the BOOS sewer, refer Section 7.2.2.3.

8.3.4 COMMENTARY ON WATERCARE'S RENEWAL PROCESSES

To reach best practice the following are recommended:

— Renewals interventions strategy be documented that specifies the repairs and renewals to be undertaken and the urgency for undertaking these works based on the observed defects/condition and the consequence of failure. This will improve transparency of renewals decisions

— For block and brick-built lines, consider using structural analysis of pipelines using finite element analysis to determine, for example the extent of pipe wall deterioration that can be tolerated under various loading conditions. Sensitivity to uncertainties such as block strength should be considered. This will improve assessment of the likelihood of failure and enable the setting of trigger levels for intervention

— Develop a prioritised list of pipelines for renewal based on observed condition and the triggers set out in the Renewals Intervention Strategy.

8.4 RENEWAL METHODS

There are two generic approaches to renewal of transmission sewers, being:

- Replacement – installing a new sewer to replace the existing sewer. This has the advantages in that the new sewer can be installed on a more favourable alignment to the existing and its size can be increased. However, installing large transmission sewers in built up areas can be complex and expensive. Installation by tunnelling is generally required
- Rehabilitation – local repair or installing a pipe (a liner) of some form inside the existing pipe. This can be cheaper and less disruptive than replacement. The new pipeline can be designed to either withstand all the loads expected to be applied, independent of the existing pipeline, or the new and existing pipe can be designed to work together as a system.

There are two main types of rehabilitation:

- Close-fit lining where the outside of the liner is in contact with the existing pipeline
- Slip Lining, where the outside diameter of the pipeline installed is smaller than the diameter of the existing pipeline.

Rehabilitation of reticulation sewers is fairly common practice in New Zealand albeit it is specialist work. However, rehabilitation of transmission sewers is more complex due to:

- Large diameters – Increases the thickness and weight of liner to be installed
- Shape – liners for oval and egg-shaped sewers are more complex to design and some techniques are only suitable for circular sewers
- Long lengths between manholes
- Large flows – extensive bypass pumping is required for installation of some techniques.

Common techniques for rehabilitation of transmission sewers are described in the following sections. Most of these techniques are available in New Zealand, however in some cases they have only been used on only a few projects.

8.4.1 CIPP LINING

Cured-in-place lining (CIPP)is a "close fit" lining technique that has been used around the world for over 50 years. Sections of Watercare's Eastern Interceptor have been rehabilitated using CIPP liners.

Whilst several variants are available, the common feature is the use of a fabric tube impregnated with a resin. The tube is inserted into the existing pipeline and inflated against the pipe wall, then cured (refer Figure 8:1). Curing may be by re-circulating hot water or steam. Some variations use ultra-violet light for curing.

Bypass pumping of wastewater flows is required as the existing pipeline is blocked during installation.

Cured-in-place liners can be manufactured to conform to almost any shape of pipe, making them suitable for lining of non-circular, e.g. ovoid cross-sections.

Pipes up to 2.7m diameter have been rehabilitated using CIPP although installation of CIPP liners in pipes greater than 1.2m is not common. The weight of the liner can be the limiting factor influencing the maximum length and liner thickness that can be installed. The need for bypass pumping can also be a limiting factor.

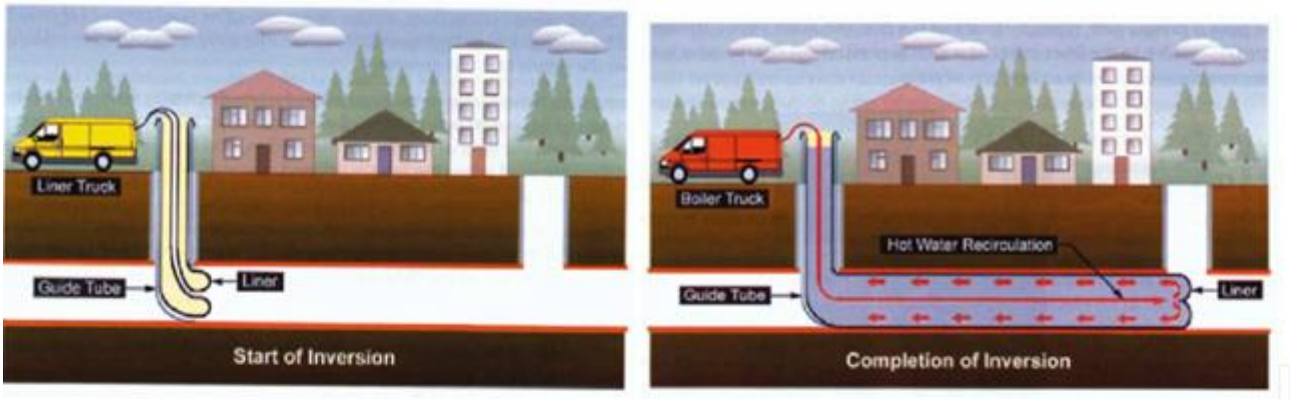

Figure 8:1 Overview of CIPP Process (Hot Water Cured). Figure from ISTT (International Society for Trenchless Technology)

8.4.2 SPIRAL WOUND LINING

A single strip of PVC is spirally wound into the existing pipeline using a patented winding machine, see Figure 8:2. The edges of the strip interlock to form a continuous liner inside the host pipe. The annulus between the host pipe and liner is typically filled with grout.

Spiral wound liners are suitable for lining circular pipes up to at least 1.8m diameter. A range of liner profiles with varying stiffnesses are available to provide liners that meet structural design specifications.

In pipes greater than 1000mm, the winding machine travels down the pipeline. Personnel are required to enter the pipe to change over liner spools, i.e. approximately every 10m of lining. Flow can pass through the liner during installation, avoiding the need for bypass pumping.

Portions of Watercare's Wairau Transmission sewer as well as other sewers have been rehabilitated using Spiral Wound Lining.

Figure 8:2 Spiral bound lining technique

The lining manufacturer has developed a technique for rehabilitation of non-round shapes such a box and arch shapes that uses a guide-frame constructed to mirror the pipeline's shape. These shapes can be anything from a box, horseshoe, teardrop, arch, etc. However, this is not currently available in Australia or New Zealand and it is unclear how successful the technique is in practice.

8.4.3 SEGMENTAL SLIP LINING

Segmental slip lining entails pushing segments of new pipeline into the old one, see Figure 8:3. The technique is suitable for rehabilitation of both circular and non-circular pipes. The annulus between the host pipe and liner is filled with grout after installation.

Segments are typically manufactured from Glass Reinforced Plastic (GRP) with the new segments matching the shape of the host pipe.

Pipe segments can be inserted during active sewer flow, avoiding the need for bypass pumping.

It is normally necessary to excavate the launch pit to provide enough space for jacking equipment. 500m or more of liner can be installed from a launch pit (Channeline International, 2011). Although bends or variations in the internal dimensions of the pipe being lined may reduce the length of installation.

The technique was used to rehabilitate the OMS at Stanley St.

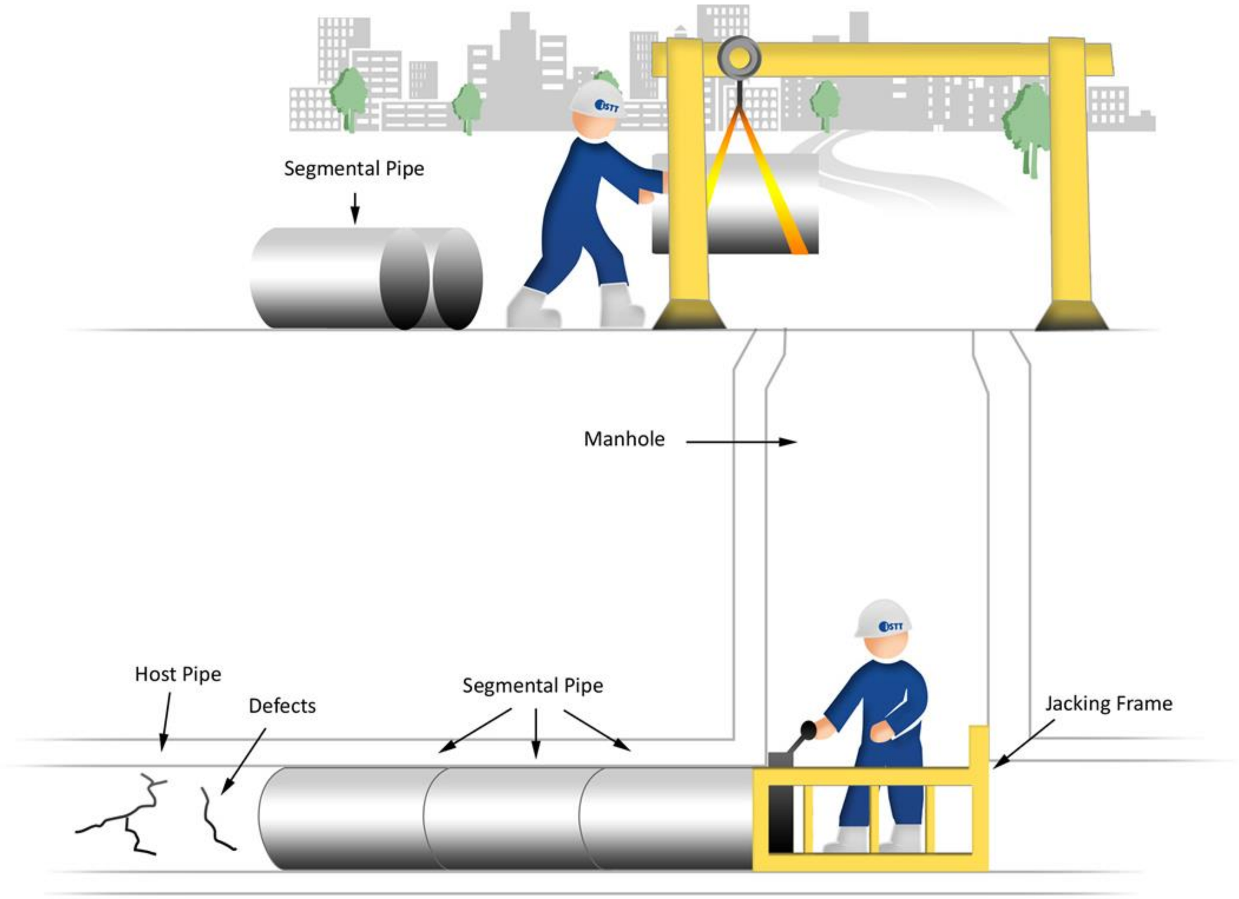

Figure 8:3 Segmental slip lining technique

8.4.4 PANEL LINING

Pre-formed panels are fabricated to fit closely to the interior surface of the pipe or structure to be lined and then are adhered to that interior surface. The technique is suitable for rehabilitation of circular and non-circular sewers between 1.2m and 3m diameter.

Personnel are required to enter the sewer to install the panels.

As far as we are aware, this technique has not been used in New Zealand.

8.4.5 SPRAYED LININGS

Inorganic or polymer products are sprayed onto the inside surface of the pipeline. When cured the sprayed material can act as a repair to restore or improve the strength of the structure, and as a protective barrier to slow or prevent further corrosion and wear. Most sprayed materials can also be trowelled on or cast if the composition is adjusted.

Personnel are required to enter the pipeline during installation of sprayed linings. Sprayed linings can't typically be applied where there is running water.

Sprayed concrete can range from relatively low specification shotcrete, through to specialist engineered concrete mixes that have high strength and faster setting times than conventional concrete. Corrosion resistance is similar to conventional concretes and a protective coating may be needed in corrosive environments.

Geopolymer (alumino silicates) are much like a synthetic stone when set. They provide a fast-setting repair and coating that is strong and very corrosion resistant.

Calcium aluminate cement (CAC) is a specially engineered material that has a faster strength gain and better corrosion resistance to wastewater than conventional concrete. Sections of the OMS in Stanley St were rehabilitated using CAC.

There are many types of sprayed polymer systems, some of which incorporate reinforcing fibres. Polymer coatings are typically very flexible compared to concrete and similar materials (which makes them good where movement can't be avoided) and they can be very resistant to corrosion. and cure times can be fast. However, polymer coatings may need dry and clean conditions to bond effectively, and some do not cure well at lower temperatures.

8.4.6 SUMMARY

The various rehabilitation techniques are summarised in Table 8:1.

Table 8:1 Summary of pipe rehabilitation techniques

Technique	Structural Rehabilitation	Shape	Close-fit	Person Entry	Bypass Pumping Required
CIPP	Yes	All	Yes	No	Yes
Spiral Wound Lining	Yes	Circular	No	Once or twice per day	No
Segmental Slip Lining	Yes	All	No	No	No
Panel Lining	Yes	All	No	Yes	Yes
Sprayed linings	Possibly [1]	All	Yes	Yes	Yes

Note 1. Sprayed linings can be applied as protective coatings or as structural linings.

The most appropriate techniques for rehabilitating transmission sewers will generally be:

- Non-circular pipelines – Segmental Slip Lining
- Circular pipelines – either Segmented Slip Lining or Spiral Wound Lining.

This is general advice. A full options assessment should be undertaken to select the most appropriate rehabilitation technique for each project.

9 SUMMARY

9.1 CONCLUSIONS

9.1.1 LIKELY CAUSE OF SINKHOLE AND SEWER COLLAPSE

The collapse occurred in the block arch portion of the sewer. It is likely that it was caused by a combination of

- **Deterioration** and weakening of the sewer over the 110 years that the OMS has been in service
- **Weak blockwork at the location that collapsed.** This blockwork would have been more susceptible to erosion and dislodgement and be less able to withstand the forces applied to the sewer
- **Exceptionally wet weather in 2023** which increased wastewater flows in the OMS and resulted in flooding and raised groundwater levels in the vicinity of the collapse.

The likely failure mechanism is:

- The internal surface of the sewer slowly deteriorated through a combination of corrosion and erosion
- High flows of wastewater in the OMS during the storms in 2023 caused further and relatively rapid erosion of the deteriorated concrete blocks and mortar joints. It is possible that some blocks were dislodged
- Groundwater infiltration into the OMS increased due to the deteriorated condition and the increased hydrostatic loading from the raised groundwater and flooding caused by the wet weather
- Infiltrating groundwater eroded fine soil particles from around the OMS causing cavities to form
- Fluctuating groundwater levels due to the repeated wet weather caused the cavities to grow and a sinkhole to form.
- Excavation for the power cable disturbed the sinkhole cavity, causing it to become visible at the ground surface
- The block arch was not able to withstand the uneven loading caused by the sinkhole. Eventually it failed under buckling and the material above it fell in and blocked the OMS.

9.1.2 REPAIR WORKS

Watercare stabilised the sinkhole and installed bypass pumping as emergency repair works.

Bypass pumping to convey wastewater around the collapsed section and avoid wastewater being discharged directly into the harbour during dry weather was installed within three weeks of the collapse. The pumping capacity of the temporary arrangement is comparable to some of the largest pumpstations in Watercare's network which normally take several years to design and construct.

The capacity of the temporary arrangement was limited, however, by the availability of pumps, the space required to site pumps, noise and vibration impacts and hydraulic constraints.

Installation of the bypass pumping enabled the collapsed section to be exposed and strengthening works around the collapse to be completed in December 2023.

Watercare intend to rehabilitate the section of OMS that collapsed using segmented slip-lining. This is expected to be completed in early 2024.

9.1.3 CONDITION INSPECTION PRACTICES

Watercare's practice of inspecting interceptor sewers every 5 years with inspections at tighter frequencies where there are concerns about the condition of particular pipelines is aligned to best practice.

The use of laser and sonar profiling combined with CCTV can be considered best practice and more sensitive inspection systems are available or will become available in the near future. The main limitation of these inspection techniques is that they only provide visual information of the inside of the pipe. Non-intrusive techniques for assessing the pipe wall and identifying voids around the pipe are still in the experimental stage and not widely available.

Current best practice is therefore to screen pipes using CCTV inspection, preferably combined with laser and sonar profiling. Then to investigate areas of concern in more detail using intrusive techniques such as coring and undertaking structural assessment using finite element analysis to assess the likelihood of failure.

As deterioration does not always proceed at a linear rate it is important to compare inspections to identify accelerating rates of deterioration. This is more valuable where good supporting information about the current and historic operating conditions, construction and surrounding environment is available.

9.1.4 CONDITION OF OMS

Significant deterioration has occurred along the 1.6km of the OMS that was assessed during the preparation of this report. There are indications that the rate of deterioration has increased since 2019. More in-depth investigations are required (as described in the previous section) to fully assess pipeline condition and likelihood of collapse.

It is likely however that some sections of blockwork have less than 100mm of competent concrete remaining, compared to the original thickness of 228mm.

Even if more in depth investigations had been undertaken back in 2019 when the last proactive inspections were completed, it is quite possible that the weak blockwork which may have contributed to the collapse might not have been picked up given the blocks only cover a few metres of pipeline section that is 118m long, i.e. length of pipeline asset between manholes ORM016 & ORM015.

9.1.5 PLANNED RENEWALS

Whilst Watercare's investment into renewal of wastewater pipes has been low over recent years, a significant increase in renewals is planned. The latest Asset Management Plan has allocated $1.9b over the next 20 years. This investment is less than and later than Watercare's preferred investment profile due to its Auckland Council debt constraint. However, it is more than double the median amount currently being spent by European Countries.

But it is important that the money available for renewals is invested in the right assets at the most appropriate time, particularly for wastewater interceptor pipelines where there is a high consequence of failure. This requires assessment of information collected from condition inspections and other sources to determine the likelihood and consequence of failure.

Watercare's corporate level policies and practices are aligned to international standards. A risk-based approach is adopted which prioritises investment into assets with high consequence of failure like transmission main sewers.

However, guidance on how these corporate level documents are applied to the management and renewal of wastewater systems is still being developed. Many decisions are made by key staff based on their knowledge and experience.

9.1.6 RENEWAL TECHNIQUES

A review of techniques for renewal of interceptor sewers has been undertaken which identified that in general:

— Segmental Slip Lining will generally be the most appropriate for lining non-circular wastewater transmission mains

— For circular wastewater transmission mains either Segmented Slip Lining or Spiral Wound Lining will generally be the most appropriate.

As this is general advice, a full options assessment should be undertaken to select the most appropriate rehabilitation technique for each project.

These techniques are available in New Zealand, albeit there is limited experience in their use in New Zealand for renewing transmission sewers.

9.2 RECOMMENDATIONS

9.2.1 CONDITION INSPECTION PRACTICES

It is recommended that Watercare continue to inspect transmission sewers every 5 years using CCTV and laser and sonar profiling with inspections being undertaken on tighter frequencies on pipelines where there are concerns about condition. However, to improve identification of condition issues it is recommended that the following be adopted:

- Timing of inspections
  - Undertake condition inspections after events that could trigger rapid decline in condition, e.g. after large storms.
- Inspection Practices
  - Improve the quality and resolution of the CCTV inspections to provide a clearer view of the pipe wall and aid the identification of faults. Either use a CCTV camera that can pan towards the pipe wall and zoom into issues or use the latest CCTV and laser & sonar profiling units which have multiple cameras that look both straight ahead and towards the pipe wall
  - Reinstate cleaning of the OMS using the plough, subject to addressing health and safety concerns. This will improve the identification of defects as the personnel who are required to travel down the sewer with the plough will also be able to observe the condition of the sewer and report on issues. Watercare have already started investigating reinstating the use of the plough. It is also worth investigating whether suitable camera equipment could be installed to allow unmanned inspections while cleaning the line with the plough
  - Produce detailed CCTV logsheets to record inspections. Assign Structural Condition Grades to provide an initial grading of condition and to identify the need for more intensive condition assessment and likelihood of failure analysis. Use a logging and grading system better suited to assessing defects in brick pipelines, e.g. the 4th Edition of the New Zealand Pipe Inspection Manual or the Conduit Inspection Reporting Code of Australia, both of which were published since the last inspections were undertaken on the OMS.
- Analysis of Condition Assessment
  - Change standard practice so that laser and sonar profiling inspections are analysed as a matter of course rather than only if issues are identified from the CCTV inspection. This could identify faults not picked up from the CCTV inspection
  - Compare laser profiling against previous inspections to determine the extent and severity of corrosion that could signal a trigger for renewal.

9.2.2 CONDITION ASSESSMENT AND RENEWAL PRACTICES

It is recommended that Watercare continue with a risk-based approach to the management of assets. This approach prioritises condition assessment and renewal of assets with a high consequence of failure like transmission sewers.

However, to help ensure that the condition assessment and renewal works are undertaken on the the right assets at the most appropriate time, it is recommended that Watercare develop the guidance documents and practices outlined below for the management of interceptor sewers.

Documentation of procedures and the reasoning behind decisions will also help auditing of decision quality and give a starting point for refining practices over time to facilitate continuous improvement.

- Consequence of Failure
  - Update processes for determining the criticality of pipelines to include all factors that could impact the consequence of failure or ease of repair, e.g. pipes under buildings, deep pipes, pipes near sensitive receiving environments. This will highlight transmission mains that might warrant more frequent condition assessment or earlier renewal.
- Likelihood of Failure
  - Develop a Condition Assessment Strategy that specifies:
    - Condition inspections – the techniques to be used under various circumstances
    - Timing of inspections, which depend on the consequence of failure, the previously observed condition of the asset and the forecast rate of deterioration
    - Other information be collected, e.g. past issues in the vicinity of the sewer
    - Processes for deriving likelihood of failure ranking from the condition inspections and other information collected
    - Circumstances that may necessitate the need for additional inspections, e.g. after high flows that could cause rapid deterioration
    - Trigger levels for undertaking more intensive investigations.
- Renewals Processes
  - Renewals interventions strategy be documented that specifies the repairs and renewals to be undertaken and the urgency for undertaking these works based on the observed defects/condition and the consequence of failure. This will improve transparency of renewals decisions
  - Consider undertaking structural analysis of block and brick-built sewers using finite element analysis to determine the extent of pipe wall deterioration that can be tolerated under various loading conditions. Sensitivity to uncertainties such as block strength should be considered. This will improve assessment of the likelihood of failure and enable the setting of trigger levels for intervention
  - Develop a prioritised list of pipelines for renewal based on observed condition and the triggers set out in the Renewals Intervention Strategy.

It is understood that Watercare are in the process of improving the documentation of their processes in line with the recommendations above. They expect to be complete by July 2024.

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APPENDIX A: TERMS OF REFERENCE

1. Purpose

On the 26th of September, Watercare was alerted to the formation of a large sinkhole over the Ōrākei Main Sewer (OMS) at 79 St Georges Bay Road in Parnell.

The OMS is a 2.1 - 2.4-metre diameter egg-shaped sewer constructed in 1911. The sewer has a precast concrete base and a brick arch.

The sinkhole grew over the next 24 hours and on the 27th of September the sewer collapsed, and the OMS became fully blocked. Further site investigation showed the blockage spanned 25 metres and is comprised mainly of large rocks and clays. The roof of the OMS collapsed over four metres.

The OMS is currently discharging all flows via several EOPs (Engineered Overflow Points) with the majority of flow discharging via an EOP at Mechanics Bay and another at Daldy Street.

This engagement will review the incident and determine the cause of the failure mode. In addition, the review will include an assessment of Watercare's current condition assessment planning and maintenance practises, asset management procedures, AMP planning process for rehabilitation works on Ōrākei Main Sewer.

2. Terms of reference

The scope of incident review shall be undertaken in three parts and will consider the current incident failure analysis and a wider review of Watercare's current asset management practises.

The three parts of the assessment are as follows:

Part 1: Site investigation and failure review

- Review of all available data from incident (including a site visit to view temporary work, review of CCTV etc.)
- Explanation of failure mechanisms for interceptor sewers like the Ōrākei Main Sewer
- Definition of sinkholes including an explanation of how they form both naturally and around infrastructure.
- Review of available geotechnical data at the failure site (79 St Georges Bay Road)
- Review of the stormwater management at 79 St Georges Bay Road
- Review of available as-built and condition assessment data for the Ōrākei Main Sewer
- Development of a theory for the cause of the sinkhole at 79 St Georges Bay Road and a failure mechanism of the Ōrākei Main Sewer
- Assessment of any proven or potential damage to the Ōrākei Main Sewer outside of the immediate failure site
- Review of the over pumping set up at Weld Street relieving surcharge on the Ōrākei Main Sewer.

Part 2: Maintenance and condition assessment review

- Review of the historical wastewater transmission planned maintenance and condition assessment programs with a brief explanation of the methods used.
- Review of all planned maintenance and condition assessment activities undertaken on the Ōrākei Main Sewer over the last 10 years
- Commentary on the appropriateness of the planned maintenance and condition assessment programmes and a gap analysis between Watercare's current practice and typical worldwide best practice for large diameter brick sewers. A commentary should be added if there are any new or emerging technologies.

Part 3: Rehabilitation and renewal planning review

- Review of Watercare's current capital programme for wastewater transmission sewer rehabilitation including methods for prioritisation
- Provide commentary on how the current methods align with typical worldwide industry best practice.

Any assessment of the impact of the failure on the environment and stakeholders is excluded from this review and will be undertaken as a separate engagement.

It is intended that the outcome of this investigation will be made public. Accordingly, the report's executive summary should take this into account.

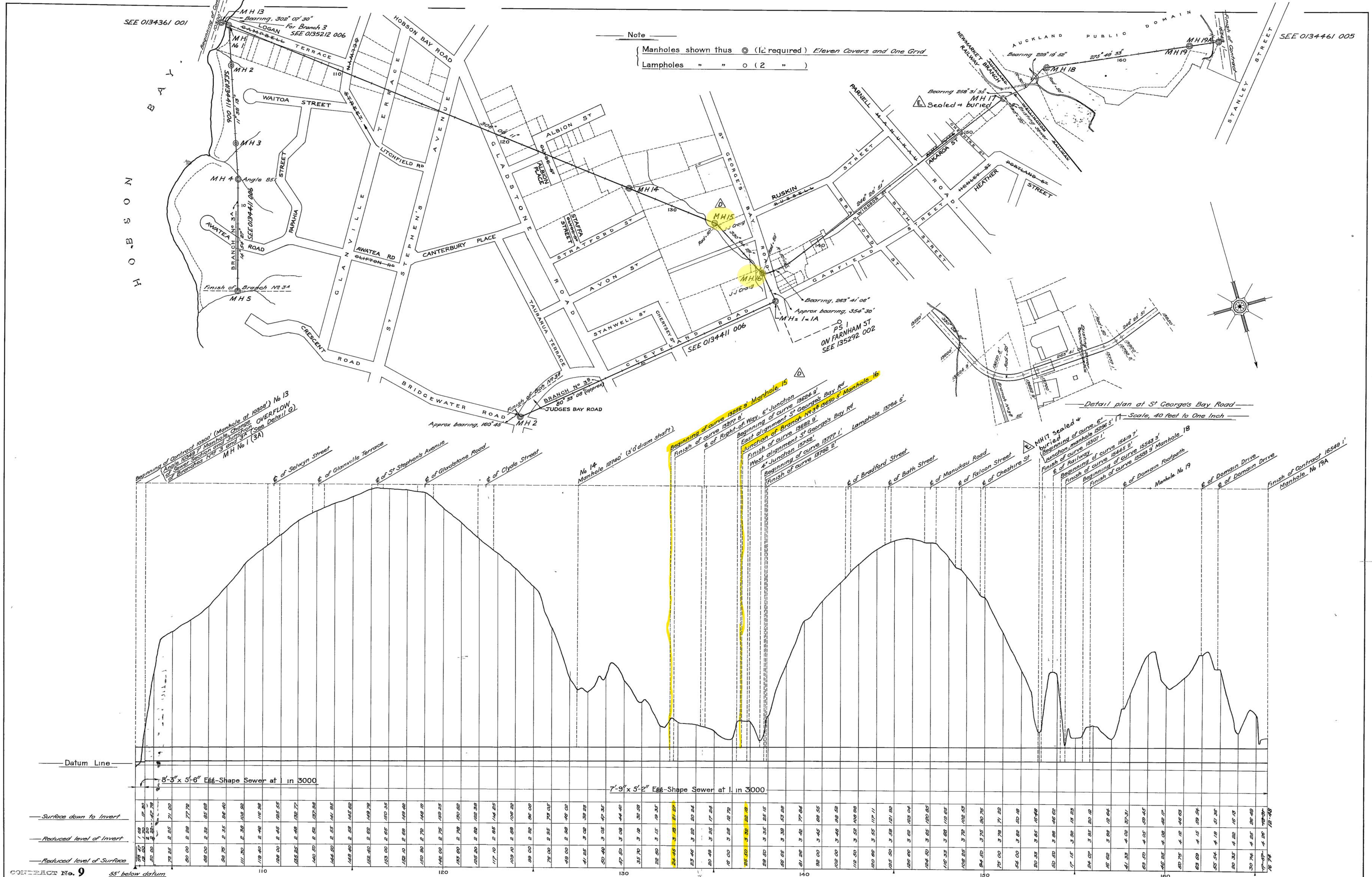
The scope of work should allow for the preparation of a draft report for review by Watercare prior to completion of the final report. Watercare will restrict comments to factual correctness and the final report will be the authors'.

APPENDIX B: TIMELINE

Table B:1 Timeline of events regarding the OMS

Date	Event	Source
1911	OMS Constructed	Drawings supplied by WSL (Appendix C)
1956	Recorded Fault	WSL
1960	Mangere Wastewater Treatment Plant opened	(WSL, 2018)
1962	Eastern Interceptor – connection to OMS	(Smith, Rogers, Graham, & Brown, 2016)
1968	Recorded fault repair	WSL
1970	Recorded fault repair/ Nestle subsidence	WSL
2014	Stanley St repair	WSL
Jan 2023	Auckland anniversary weekend floods	(NZIC, 2023)
26 Sep 2023	Sinkhole first observed by a Contractor and reported to Watercare	WSL
27 Sep 2023	Major collapse blocking OMS and causing wastewater overflows to Waitemata harbour	(WSL, 2023a)
28 Sep 2023	Temporary repair work begins	(WSL, 2023a)
17 Oct 2023	Bypass solution (6 pumps) operational	(WSL, 2023a)

APPENDIX C: OMS DRAWINGS



CONTRACT No. 9

'E'	4'08	I M	MH17 SEALED + BURIED
'D'	10-8-94	I M	MH 15 RELOCATED
'C'	12-10-93	M D	OVERFLOW BR 3/3A JUNCTION
'B'	24-8-93	M D	AMENDED/UPDATED

AUCKLAND & SUBURBAN DRAINAGE BOARD ENGINEER'S OFFICE SEP 24 1910

107/1

Scale Horizontal 200 feet / Vertical 20 feet to One Inch.

MAIN SEWER.

Section No. 3. and BRANCHES 3A and 3B

Drainage Engineer.

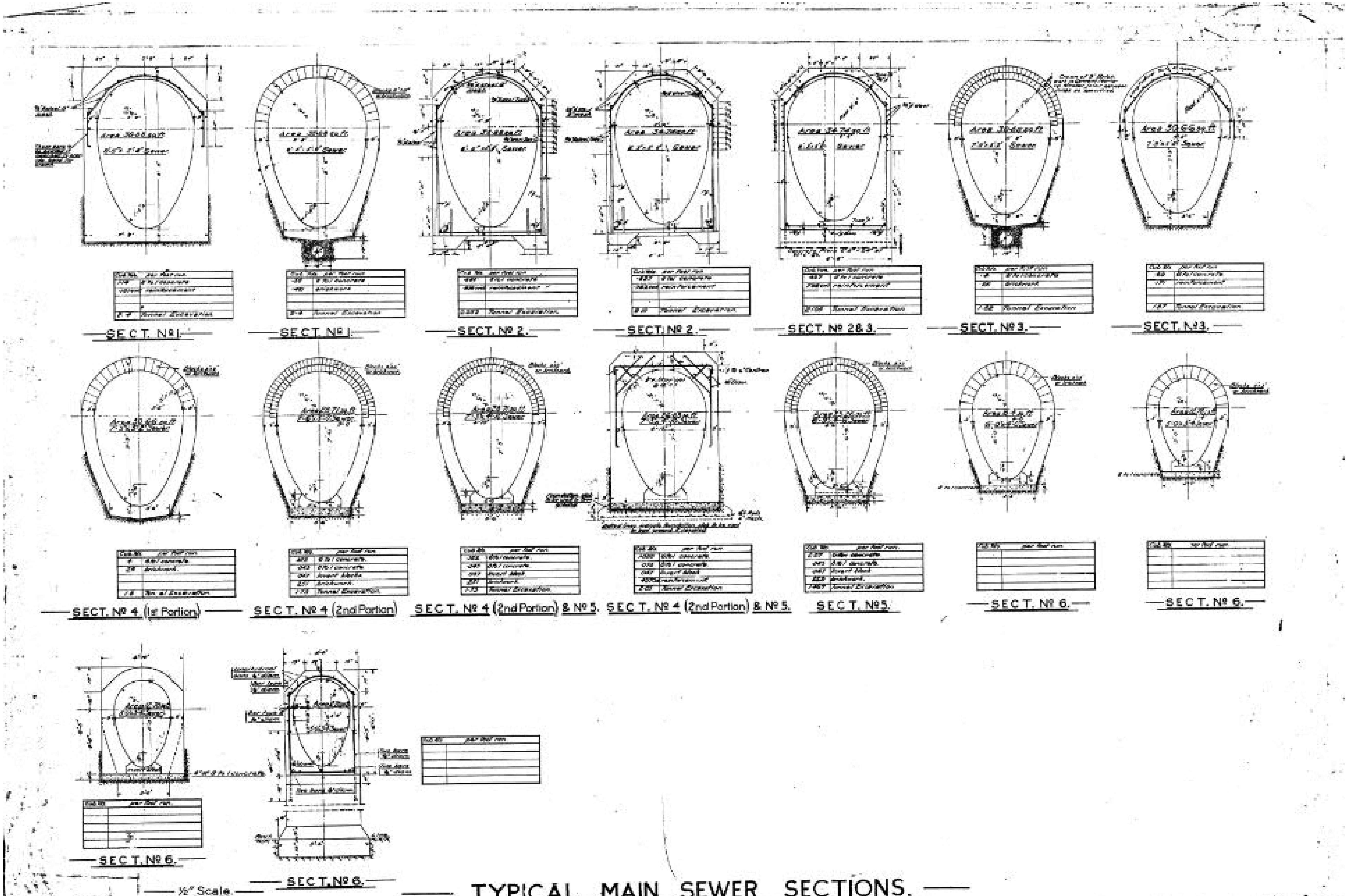
Drawn by

Checked by

MICROFILMED

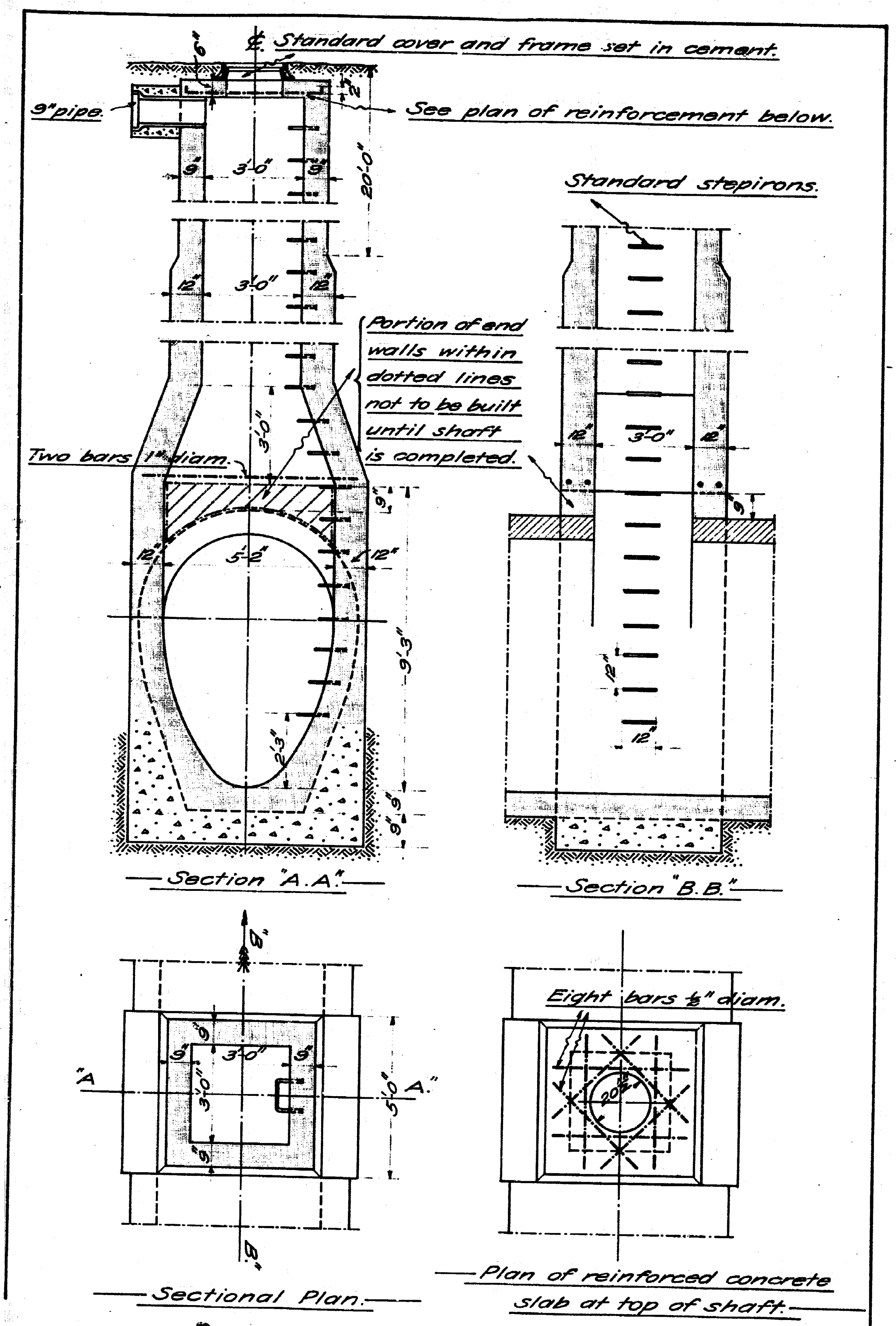
0134411.005E

8



TYPICAL MAIN SEWER SECTIONS.

1E4



CONTRACT No. 9

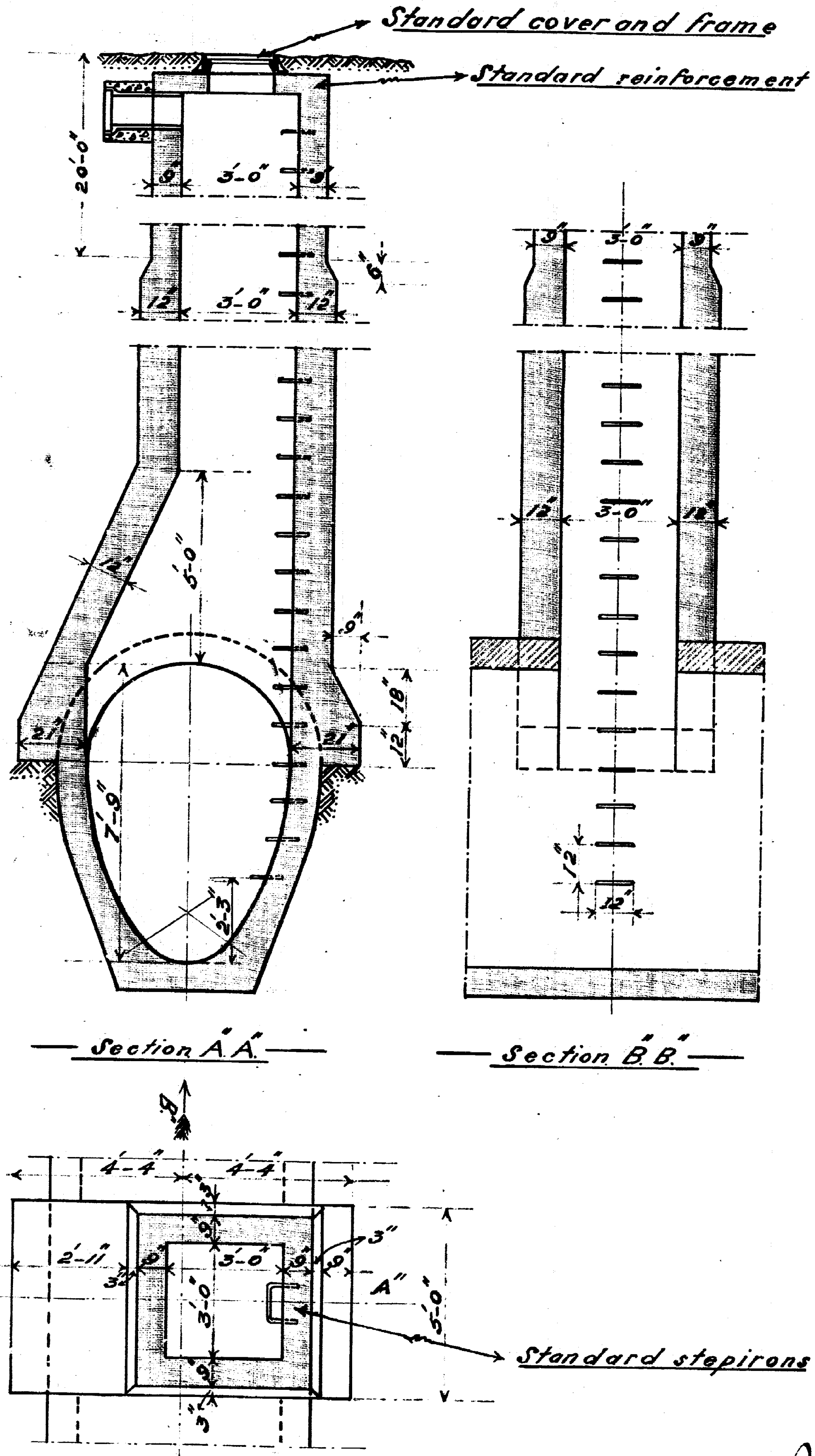
Main Sewer, Section No 3.

Amended details of Manhole at 127+

¼" Scale.

W.H.W.

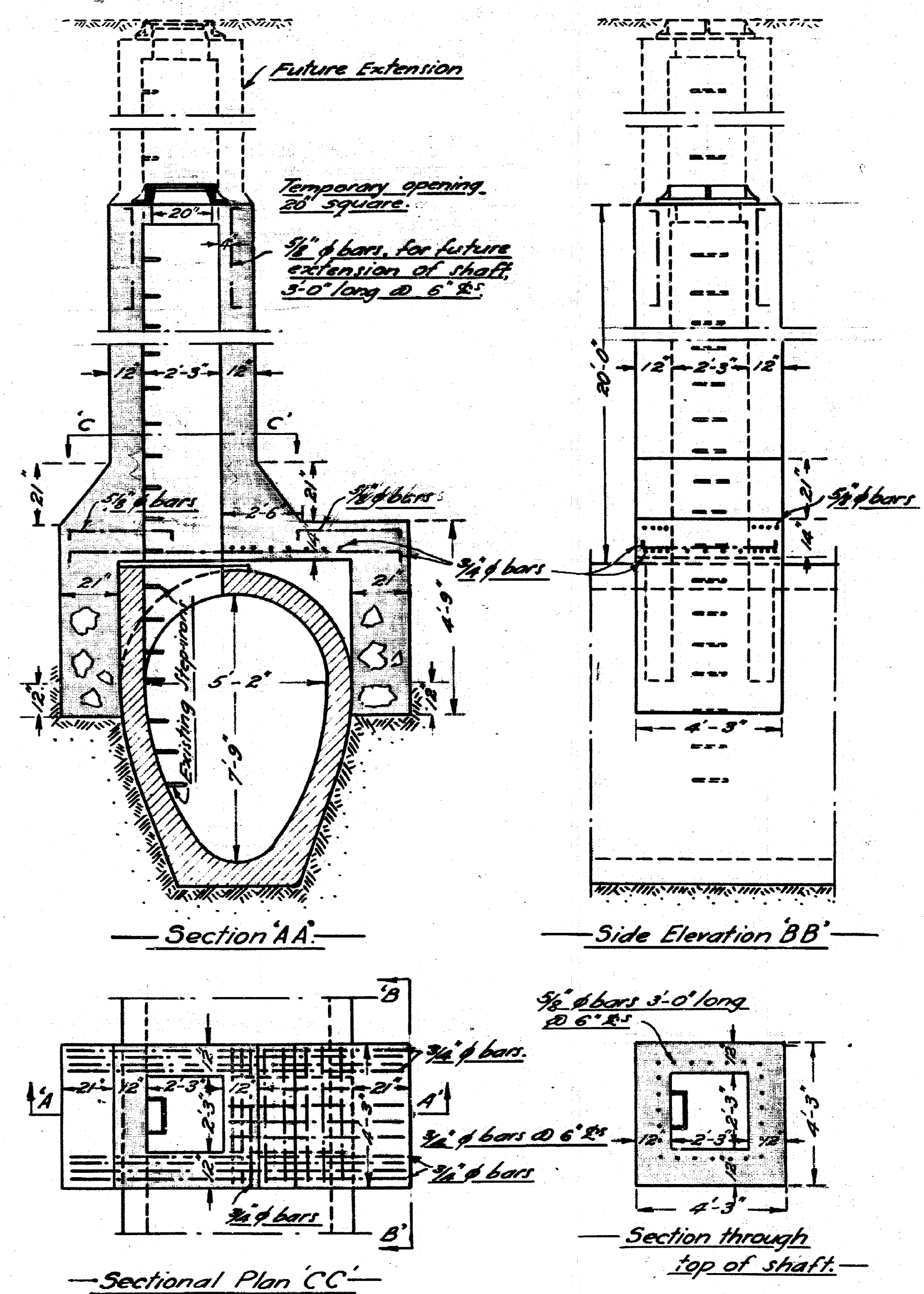
135/3 Dr 10



Main Sewer, Section Nº3

Amended details of Manhole at 13257.

¼" Scale



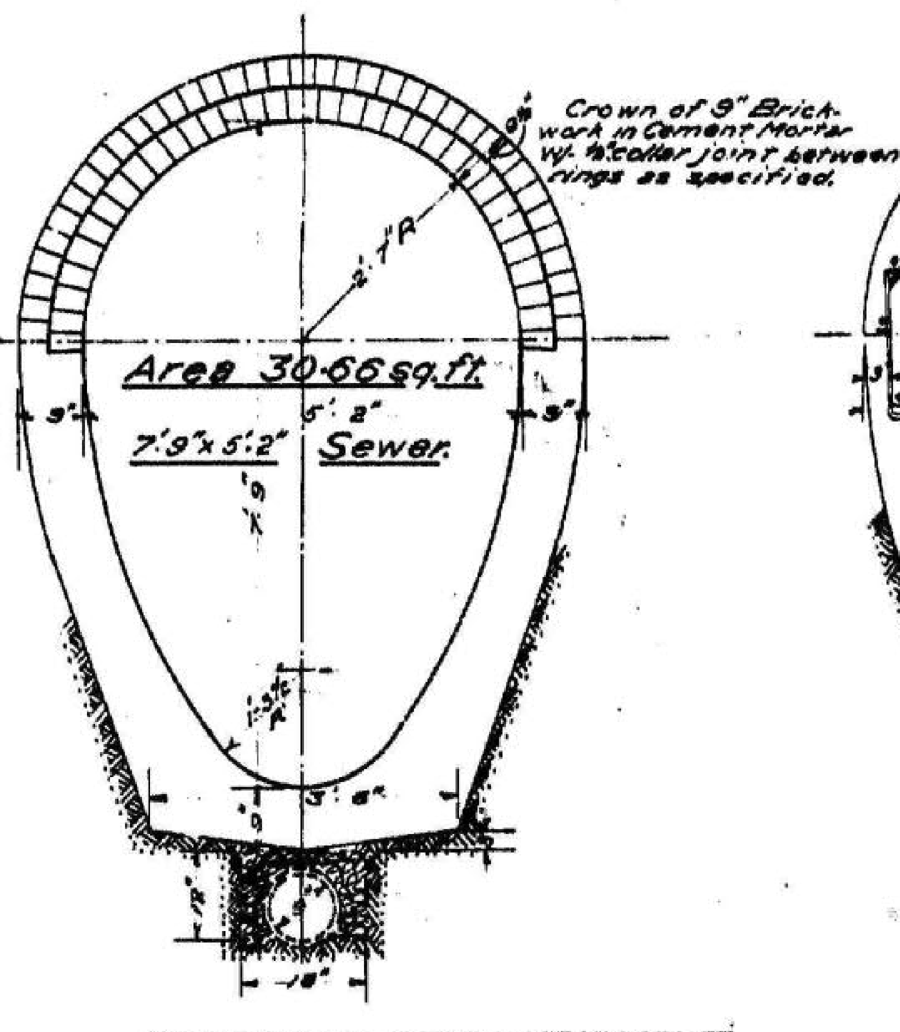
MAIN SEWER. SECTION N°3.

Alteration to Manhole N° 13

1/4" Scale.

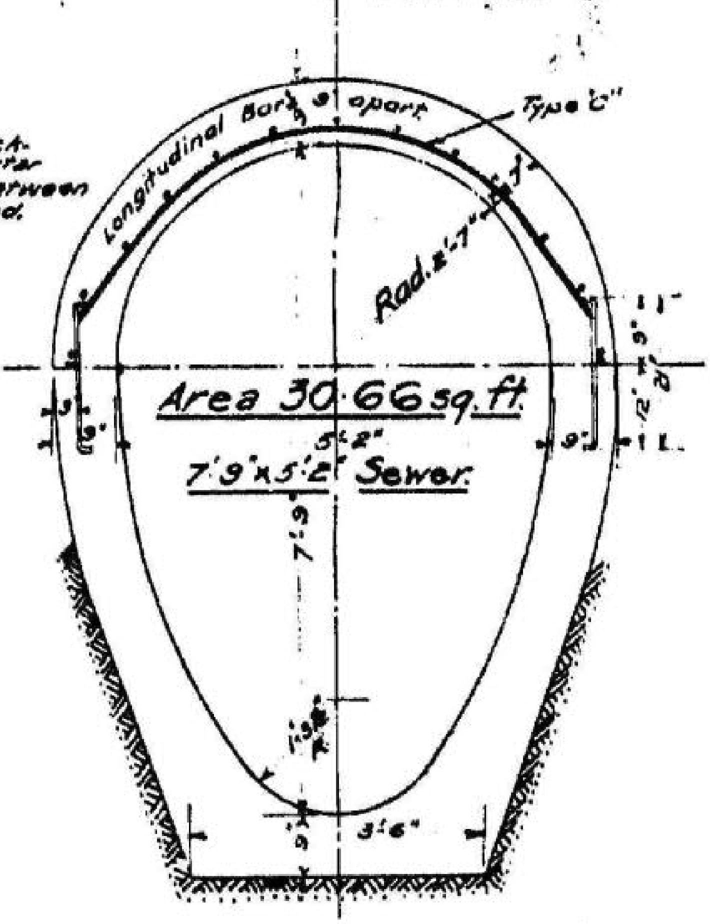
306/2

Drainage Engineer
Drawn by
Checked by



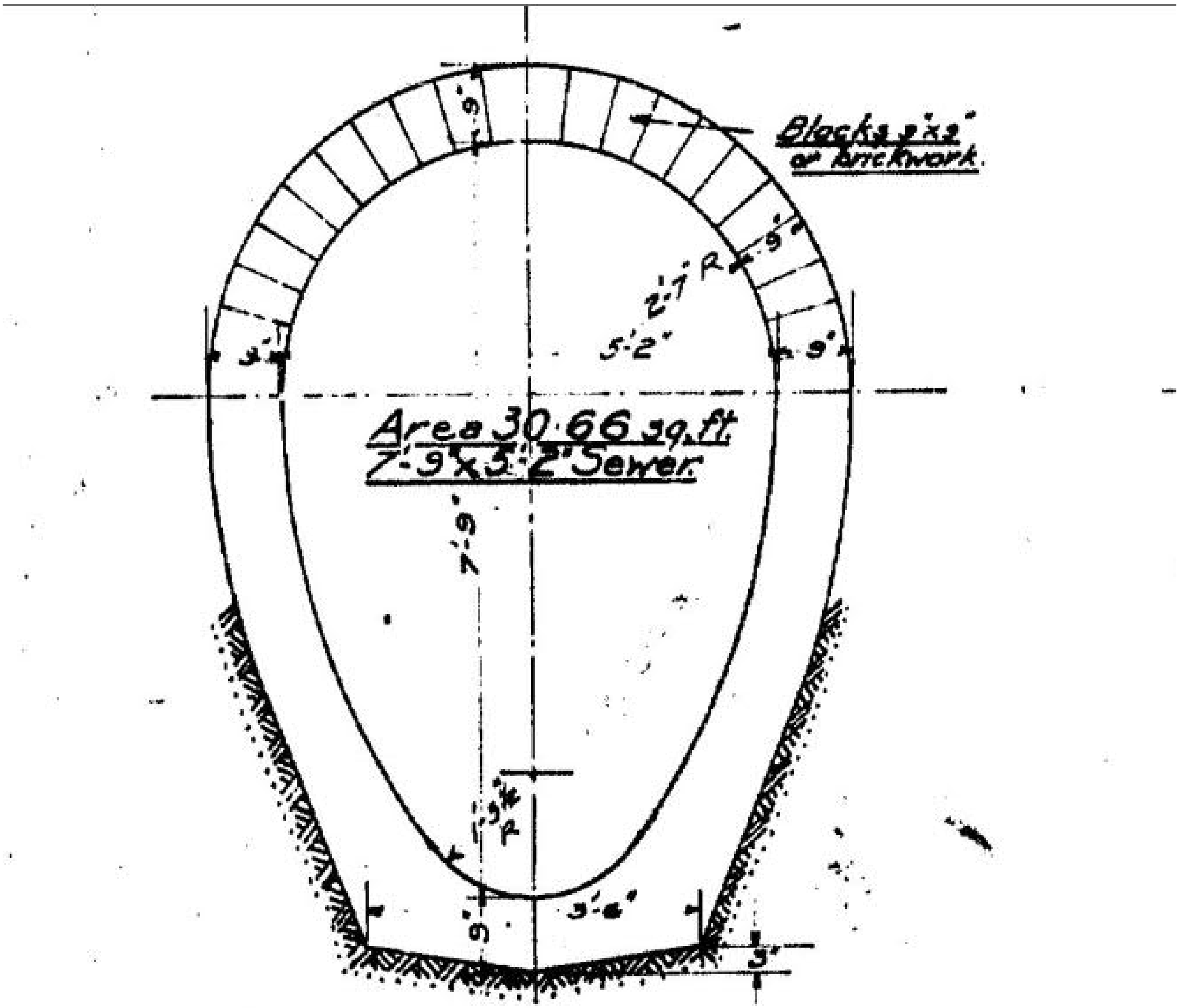
Cub Yds.	per foot run
·4	6 to 1 concrete
·26	brickwork
1·82	Tunnel Excavation.

SECT. № 3.



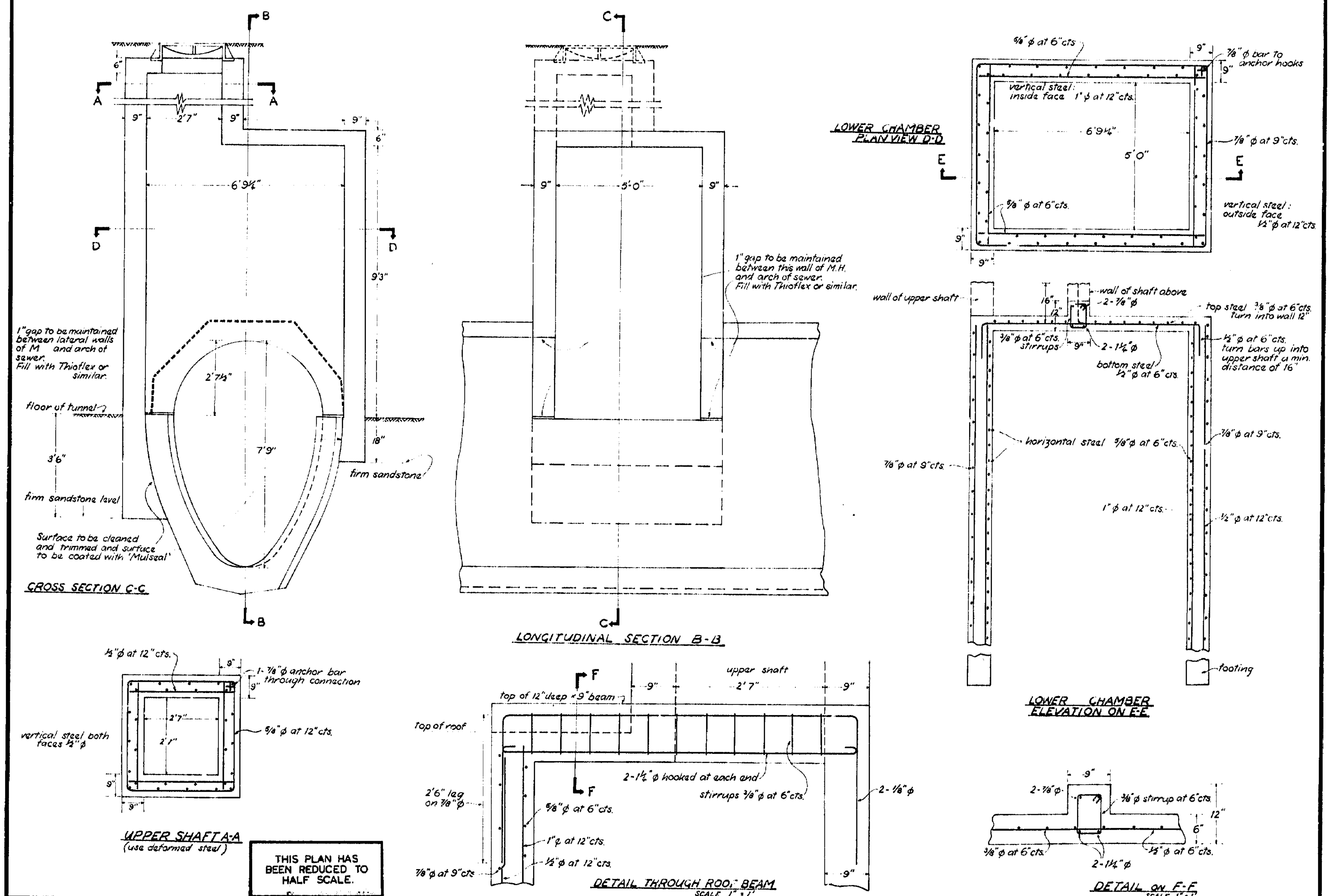
Cub Yds.	per foot run
·69	6 to 1 concrete
·171	reinforcement
1·87	Tunnel Excavation.

SECT. № 3.



Cub. Yds.	per foot run.
·4	6 to 1 concrete.
·26	brickwork.
1·8	Tun. al Excavation.

SECT. Nº 4 (1st Portion.)



THIS PLAN HAS BEEN REDUCED TO HALF SCALE.

Issue	Date	AMENDMENTS	Initials

Surveyed / Field Book No. / Drawn / Checked / Designed O.B.H. / Drawn A.P.C. 7.1.70 / Checked

AUCKLAND REGIONAL AUTHORITY – WORKS DIVISION

Approved — Date 7.1.70 — ENGINEER — CHIEF ENGINEER — DIRECTOR OF WORKS

MAIN SEWER REPAIRS
RECONSTRUCTION OF MANHOLE 16-ST GEORGES BAY RD

CONTRACT No. — SHEET 13 — SCALE ½" TO 1' UNLESS OTHERWISE STATED — MICROFILM NUMBER 134418

INCHES 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20
METRIC 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 32 33 34 35 36 37 38 39 40 41 42 43 44 45 46 47 48 49 50 51

APPENDIX D: GEOTECHNICAL MEMO

Orakei Sewer –Failure Mechanism

Geotechnical Memo

Background

Following development of a sinkhole adjacent to 79A Saint Georges Bay Road, Parnell, which was subsequently found to be related to the collapse of a section of the crown of a block/brick lined 2.1m diameter wastewater main (Orakei Main), WSP have been engaged by Watercare to provide a desktop assessment of likely failure mechanism that led to the collapse.

This memo introduces the engineering geological setting and geotechnical conditions and discusses the potential impact the ground conditions have on the mechanism of tunnel failure.

The site is a sealed carpark in an urban commercial and residential area

Geological setting

The site has been mapped on published geological maps[1] as Miocene aged East Coast Bays Formation (ECBF), which is comprised of predominantly alternating sandstone and mudstone with variable volcanic content and interbedded grits.

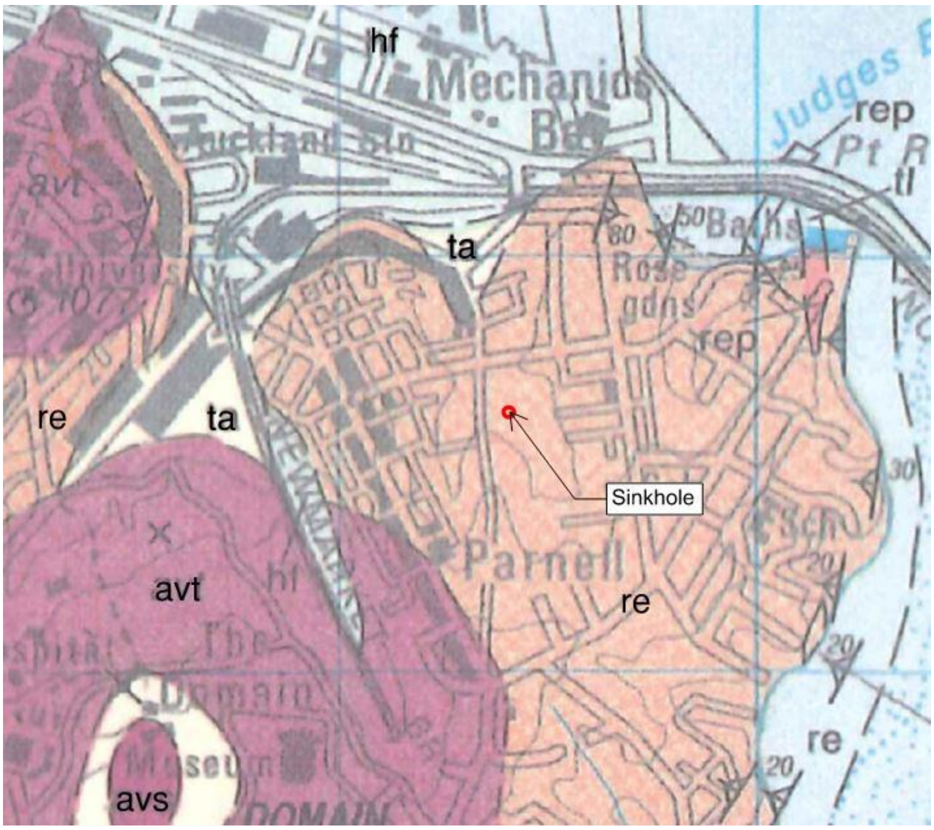


Figure 1: Geological map after Kermode 1:50 000[1]. ta - Tauranga Group, avt - Lithic Tuff, re - East Coast bays formation (ECBF), hf - Construction Fill

The near surface layers of these very weak to weak alternating sandstones and mudstones tend to progressively weather in situ to form firm to hard clays and silts.

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[1] Kermode, L.O. 1992: Geology of the Auckland urban area. Scale 1:50,000. Institute of Geological and Nuclear Sciences geological map 2. 1 sheet + 63 p. Institute of Geological & Nuclear Sciences Ltd., Lower Hutt, New Zealand.

The base of an infilled stream gully is inferred to run through the site. As such the top portion of the ground profile is comprised of fill, see Figure 2 and Figure 3.

Figure 2: Contours showing the presence of an infilled gully leading from Alberon Reserve through The Strand leading from an RL of 15m to 5m.

Figure 3: Historic stream overlaid across the site and position of the sewer and sinkhole.

Geomaterials and Ground Profile

East Coast Bays Formation (ECBF) – Forming the basal unit, the strength of the ECBF rock is generally very weak to weak. The ECBF weathers residually into various states, SW-to MW rock which is often described as a transitional rock zone is characterised by a loss of cementation forming a brittle extremely weak rock mass that can also be described as a dense sandy and silty soil. As weathering continues the rock mass develops further soil properties through the highly weathered to completely weathered state where although some of the rock fabric is preserved the material is a soil.

Tauranga Group – There is a possibility that there are localized areas of old stream sediments that would form part of the Tauranga Group in the vicinity of the sinkhole. Whilst it is most likely that any sediments associated with temporary works during the construction of the Orakei Sewer pipeline were undercut before being subsequently infilled, the presence of remnant lenses of soft Tauranga Group soils cannot be ruled out.

Fill – The old gully feature has been infilled and fill materials are likely to be comprised of a variety of materials emplaced to form the carpark level and to backfill excavations that may have associated with the Orakei Sewer construction.

Site observations[2] made by Baseline Geotechnical Ltd have confirmed the ground profile above the pipeline collapse and provide a good assessment of the ground profile conditions.

A summary of the ground profile is presented in Table 1.

Table 1: Summary of ground profile above the crown hole collapse

Depth (mbgl)	Geology Unit	Material Description
0-3.5m	Fill	Gravel and silt, brown and friable
3.5-5.3m	Fill	Clays and silts, mottled orange, light brown and brown
5.3-5.6m	Buried Topsoil	Clayey silt with organics, dark brown to black
5.6-8.8m	Weathered ECBF	Clayey silt with some sand, brown

Historic geotechnical investigation work undertaken for the development at 96 St Georges Road[3] and CPT testing undertaken by the Contractor for the repair works have also been used to help develop the ground model in particular to confirm the depths to rockhead – see Figure 4 and Figure 5.

[2] Memo: St Georges Bay Road Sinkhole Ref. BGL000180 Adjacent to 79A St Georges Bay Road for ACH (Brett Chick) dated 28 September 2023
[3] Tonkin and Taylor ltd ref. 30507/Rev A 96 St Georges Bay Road, Parnell Geotechnical investigation Report for Mansons TCLM Ltd dated April 2015

Figure 4: Plan of site showing sinkhole, historic streams, recent CPT's and section lines.

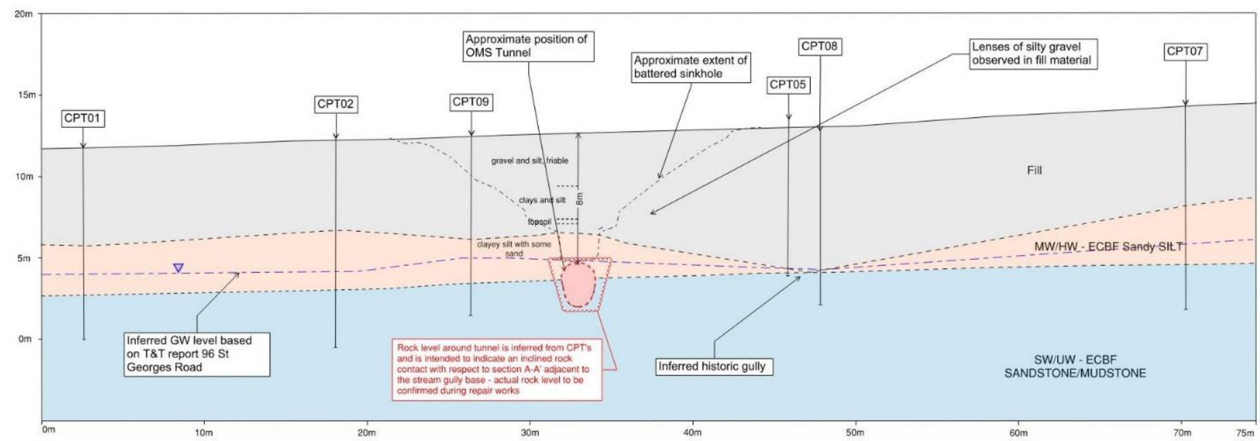


Figure 5: Cross-section showing the profile of the historic infilled gully in relation to the sinkhole and pipeline.

During the Baseline Ltd site inspections several observations were noted:

- it was noted that there were no signs of groundwater seepage from the soil profile above the tunnel
- At the time of their visit late on Tuesday 26 September whilst a large void had developed above the sewer, "The sewer line was exposed at this time and wastewater flows could be observed through the hole in the crown of the tunnel".
- By midday 27 September the void sidewalls had collapsed blocking the wastewater line.

These site observations indicate that the crown hole collapse did not initially result in blockage of the sewer, and it we infer that debris had been entering the sewer and flushing away over time in the lead up to the sinkhole daylighting the surface.

Groundwater setting

The groundwater setting reflects the topography and geology of the site. Historic geotechnical investigation work undertaken for the development at 96 St Georges Road[4] includes a discussion on the groundwater setting that is relevant to the sinkhole site.

The base of the former gully falls from south to north and is located near to where the sinkhole has formed see Figure 4. The site is approximately 350m from the inferred natural coastline and 800m from the current coastline, comprising the Auckland Port.

The natural groundwater flows within the upper portion of the site within the fill are inferred to comprise surface infiltration and groundwater seepage from east and west of the old gully axis. Within the lower portion of the historic gully near where the OSM tunnel is located natural groundwater flows are inferred to comprise south to north seepage below the valley floor towards the coastline. In the base of the gully the measured groundwater levels from historic piezometers are near the interface of the fill and natural soils[5].

It is anticipated that during extended periods of rainfall then antecedent groundwater levels would be higher within the fill for example at the end of winter, whilst in summer these groundwater levels would drop down towards the base of the old gully. Intense rainfall events such as 2023 Auckland anniversary and Cyclone Gabrielle (see Figure 6)following a period of dry weather, would likely result in a temporary perched slug of water in the fill that would pass down through the fill before merging with the natural groundwater level. This can result in temporary hydraulic gradients that can cause subsurface erosion if soils are susceptible to erosion and there is a pathway for soils to be eroded out through and away such as a pipeline or tunnel.

[4] Tonkin and Taylor ltd ref. 30507/Rev A 96 St Georges Bay Road, Parnell Geotechnical investigation Report for Mansons TCLM Ltd dated April 2015

[5] Tonkin and Taylor ltd ref. 30507/Rev A 96 St Georges Bay Road, Parnell Geotechnical investigation Report for Mansons TCLM Ltd dated April 2015

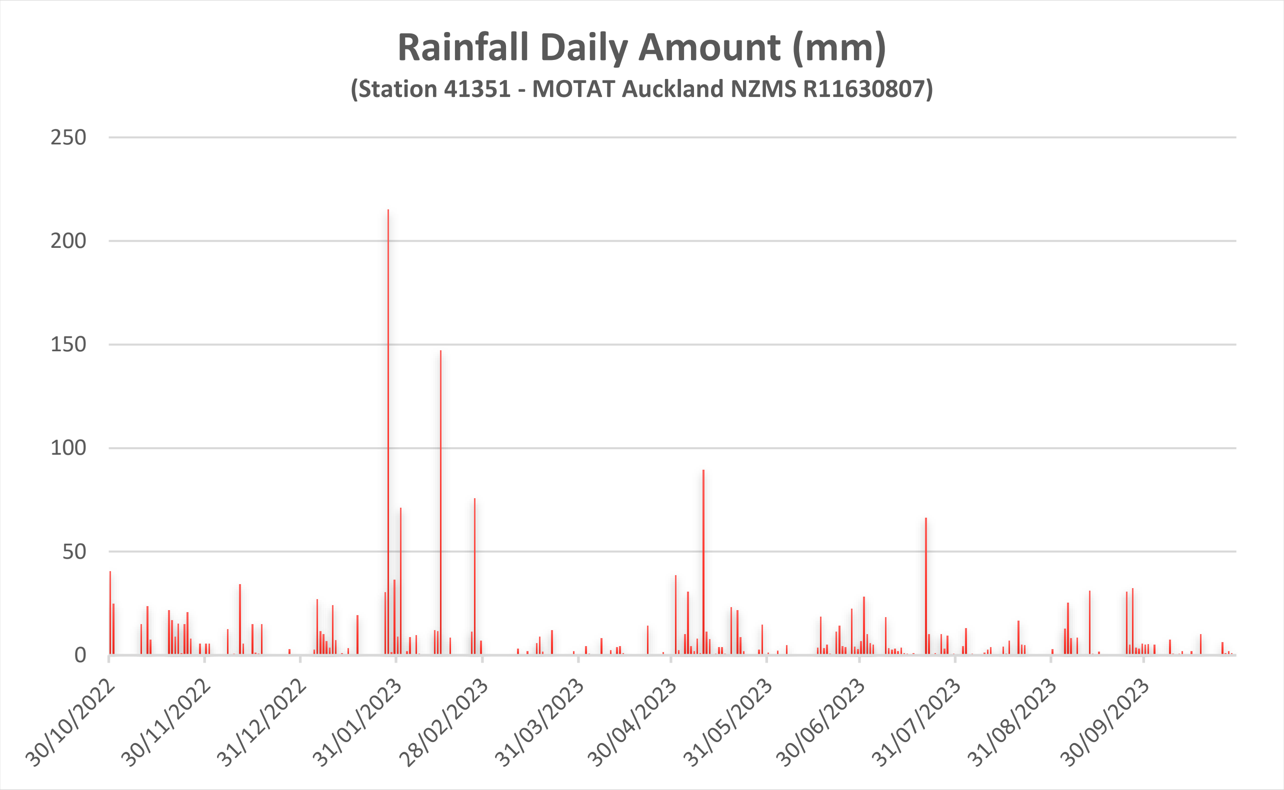


Figure 6: Daily rainfall for Auckland from the period 30 Oct 2022 to 30 October 2023

APPENDIX E: STORMWATER MEMO

Memorandum

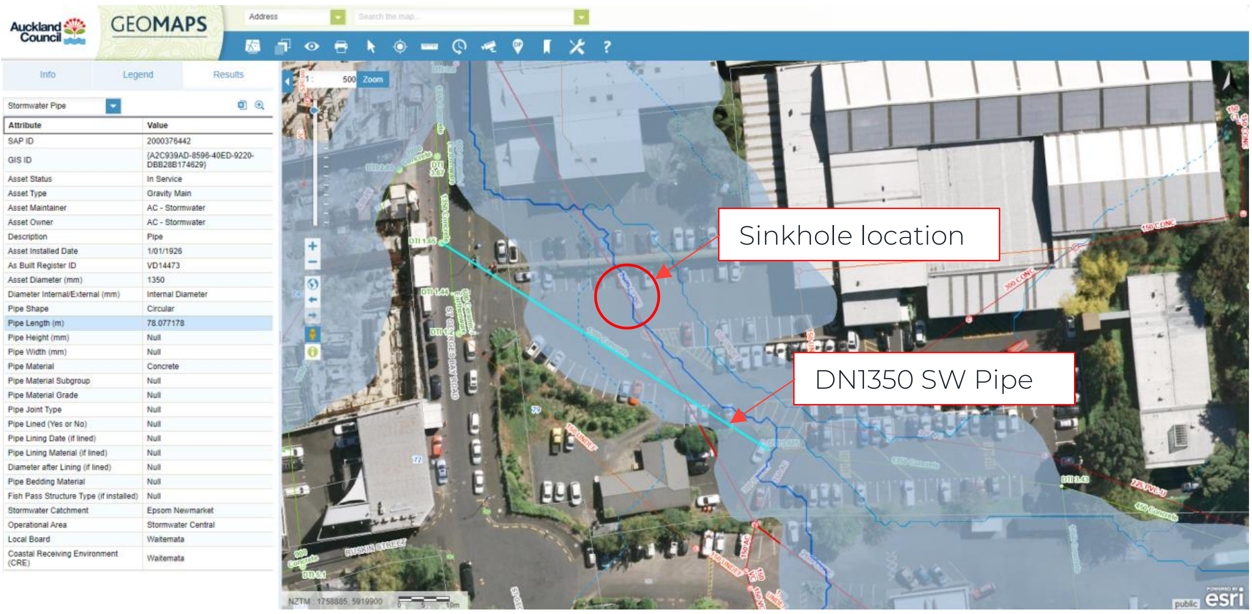
To	Philip McFarlane
Copy	
From	Zeb Worth
Office	Auckland
Date	4 December 2023
File/Ref	W-SL021.02
Subject	Ōrākei Sewer Main Failure – Stormwater Observations

1 Background

This memorandum summarises the findings of an assessment into potential contributions by stormwater and overland flow/flooding to the September 2023 failure of the Ōrākei Sewer Main at 79 St Georges Bay Rd, Parnell.

This assessment is based on a desktop review of available information relating to the existing infrastructure and events leading up to the failure, and observations taken on site following the collapse of the sewer main.

2 Existing Stormwater Network



There is an existing DN1350 stormwater pipe which crosses the OMS immediately south of the location of the sinkhole/collapse. GIS records show that his pipe was installed in 1926 and is a

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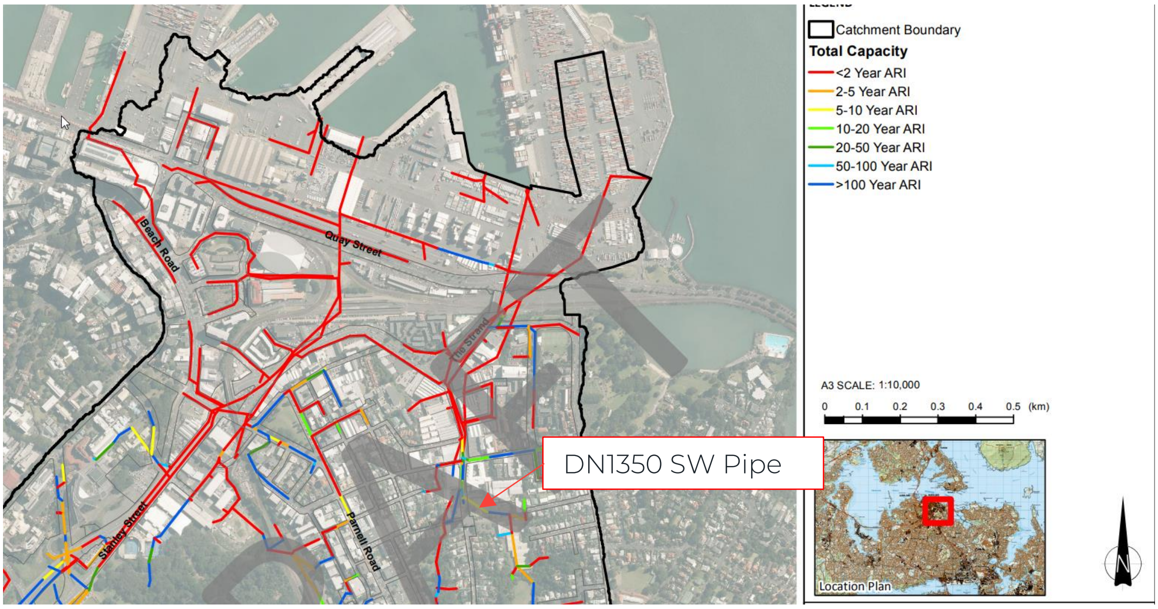
concrete pipe approximately 3.5m depth to invert. The structural and hydraulic condition of this pipe is currently unknown. However, given its age it is possible that there is some deterioration of the pipe joints and seals which may allow water to exfiltrate into the surrounding trench material, particularly when the system is under surcharge pressure. This could result in subsurface flows following the trench and locally contributing to groundwater flows.

Auckland Council's Stanley Stormwater Model Build Report (DRAFT) indicates that the total capacity of this section of pipe and much of the upstream pipe network is between the 2yr and 10yrARI event under Maximum Probable Development (MPD) conditions[1]. It is therefore likely that the stormwater network in and around 79 St Georges Bay Road was surcharged and the overland flow paths active in at least the January and February 2023 events. This may have also been the case during the May 2023 event, although this would have been dependent on antecedent conditions and downstream tailwater levels at the time.



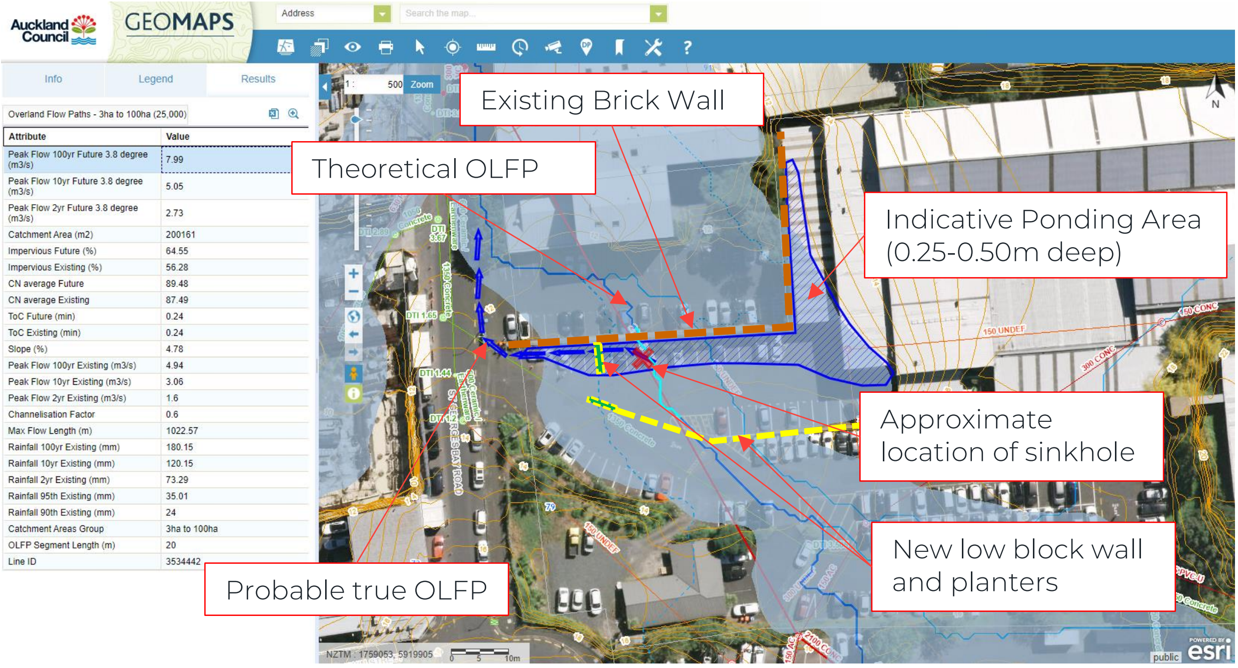
3 Overland Flow Paths

GeoMaps shows an overland flow path (OLFP) that runs from south to north through the carpark area at 79 St Georges Bay Road. Based on the GeoMaps data, this OLFP has a theoretical peak flow of $4.94m^3/s$ in a 100yr ARI event under existing climate and development conditions and a time of concentration of ~14mins.

However, based on site observations and existing contours, the true OLFP is likely to be diverted west by an existing brick wall (this had been partially demolished at the time of the site visit for safety reasons. The existing brick wall runs almost perpendicular to the flow path line shown in GeoMaps. There is a low point located midway along the brick wall roughly where the theoretical flow line is shown in GeoMaps. This low point is almost directly above the OMS near the location of the collapse. During extreme rainfall events when the capacity of the primary stormwater pipe system is exceeded, overland flow intercepted by the wall would need to pond to a depth of around 0.25m-0.50m before it was able to overtop the high point at the western boundary of the site with St Georges Bay Road as shown below. It is considered likely that this occurred during the extreme weather events of January and February 2023. Any defects in the asphaltic concrete, or pervious surfaces within the ponding area would allow this water to infiltrate into the underlying soils and trench backfill, contributing to subsurface groundwater flows.

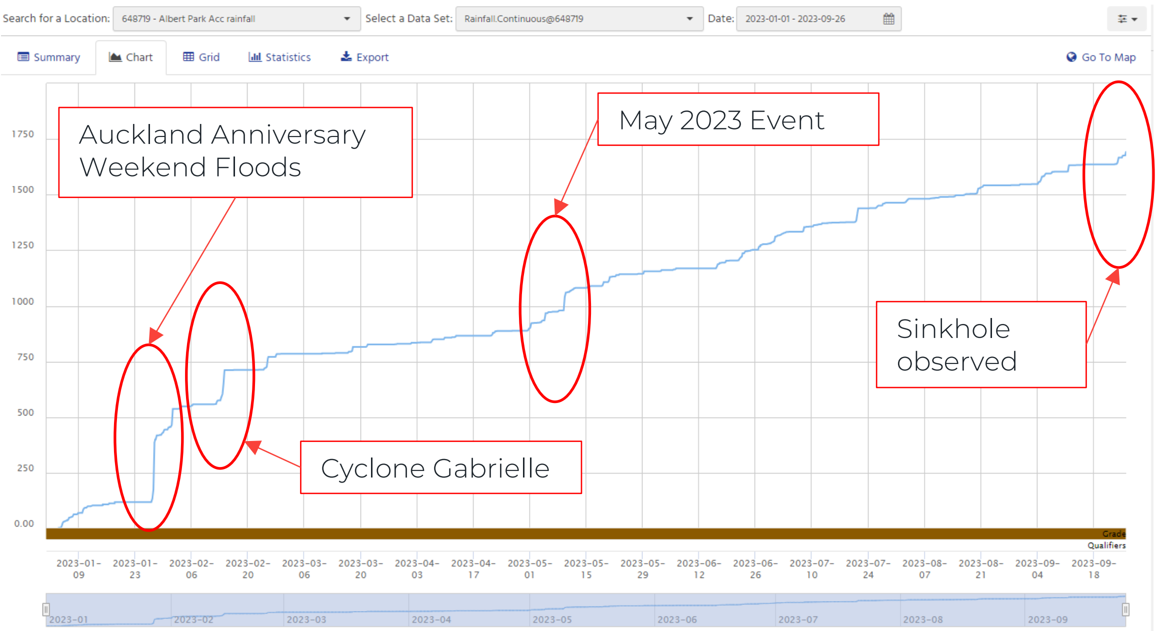
[1] ID_120 Part 2 Stanley MODELBUILD-DRAFT ISSUE 2015.pdf (aucklandcity.govt.nz)

At some point following the Auckland Anniversary event a new low concrete block wall and planter boxes were installed across the OLFP, south (upstream) of the low point. It is unclear what the intended purpose of this wall is, but it may have been intended to reduce divert the OLFP away from the low point. However, given the extent of the wall and the existing surface contours, it is unlikely that it would have successfully diverted all the overland flow and may in fact have forced water to pond deeper because of the planter box return from the existing block wall.



4 Significant Weather Events and Rainfall

There were several significant rainfall events during the first half of 2023. In total 1690mm[2] of rainfall fell between 1st January 2023 and 26th September, when WaterCare were first notified of the formation of the development of a sinkhole at 79 St Georges Bay Road.

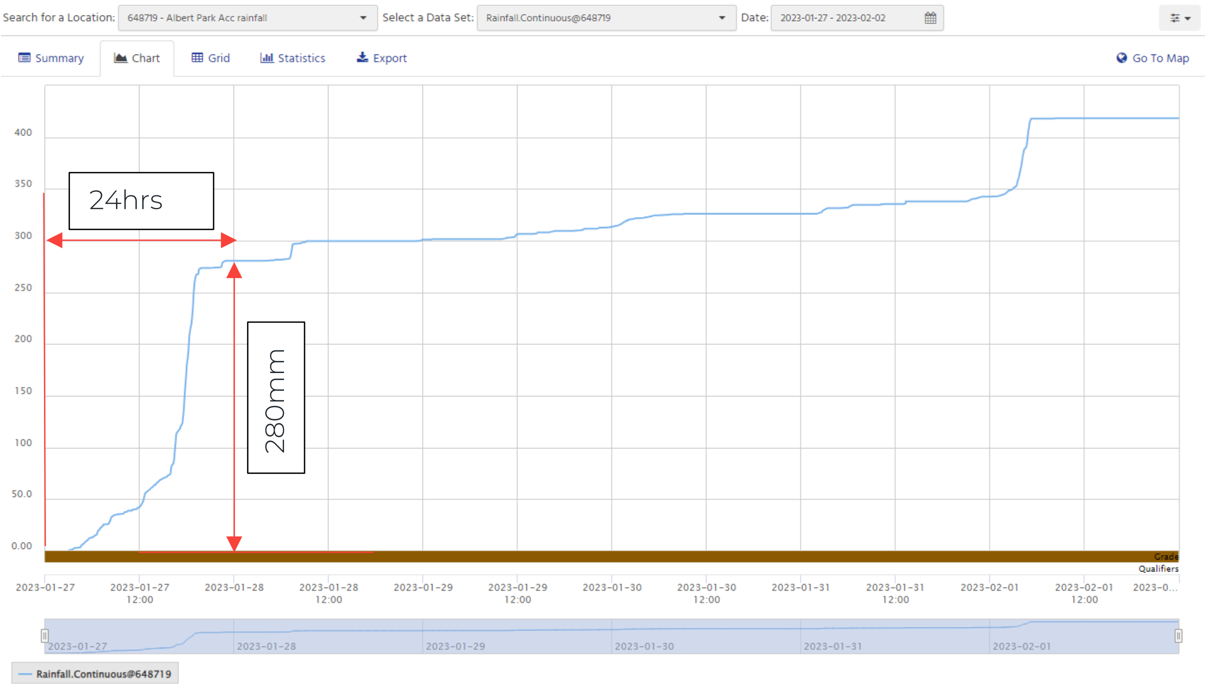


[2] https://environmentauckland.org.nz/

This compares to an average annual rainfall in Auckland of 1137mm[3]. This indicates in general that the rainfall during the first nine months of 2023 was well above the 'normal' annual rainfall for the area. As a result, the capacity of local depressions and pervious areas within the contributing catchment to store and hold back runoff is likely to have been limited, resulting in an increase in surface flows during even relatively minor rainfall events. Much of the excess rainfall in the first nine months of 2023 fell during the following significant weather events.

4.1 Auckland Anniversary Weekend Floods (27th January – 1st February 2023)

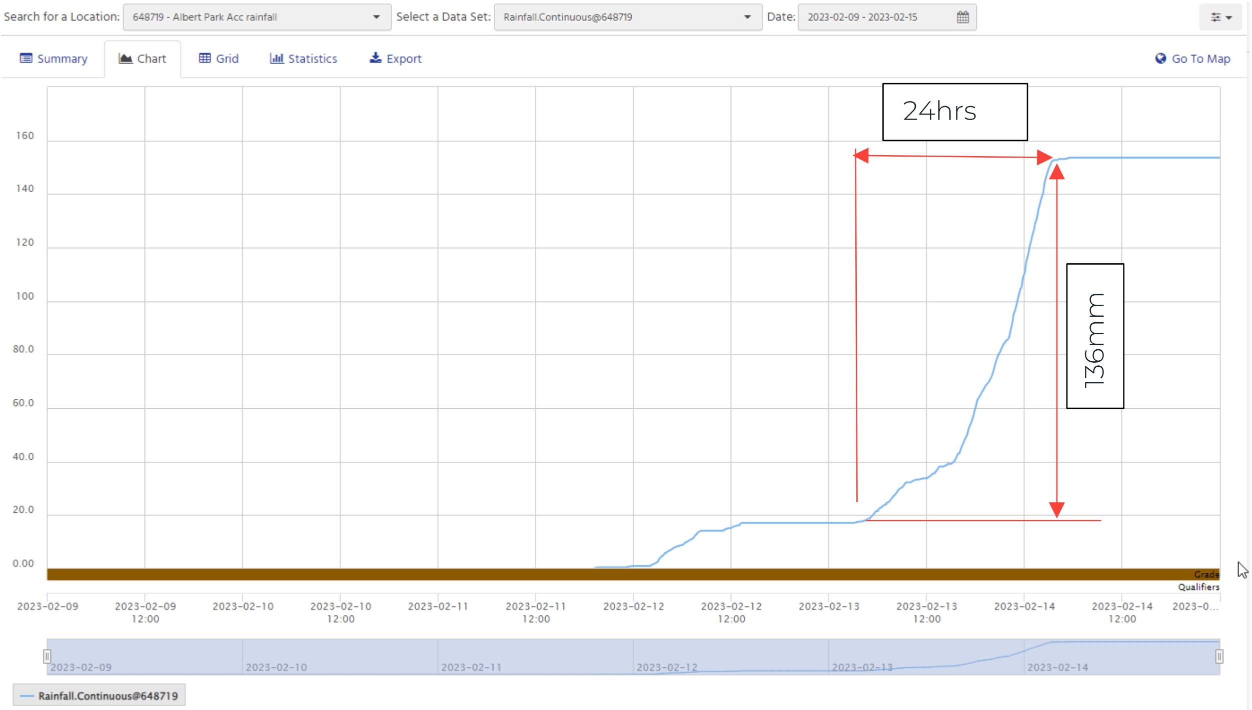
Albert Park Rain gauge shows 280mm of rainfall (24hrs 00:00 27/01/23 to 00:00 28/01/23) – cf. TP108 24hr rainfall maps 100yr ARI 180mm. This was followed by a further 138mm (102hrs 00:00 28/01/23 to 06:00 01/02/23). The peak 14min rainfall intensity coinciding with the critical time of concentration of the upstream catchment was ~115mm/hr which is greater than the theoretical TP108 100yr ARI peak rainfall intensity of ~103mm/hr for the same critical duration.



4.2 Cyclone Gabrielle (12-14 February 2023)

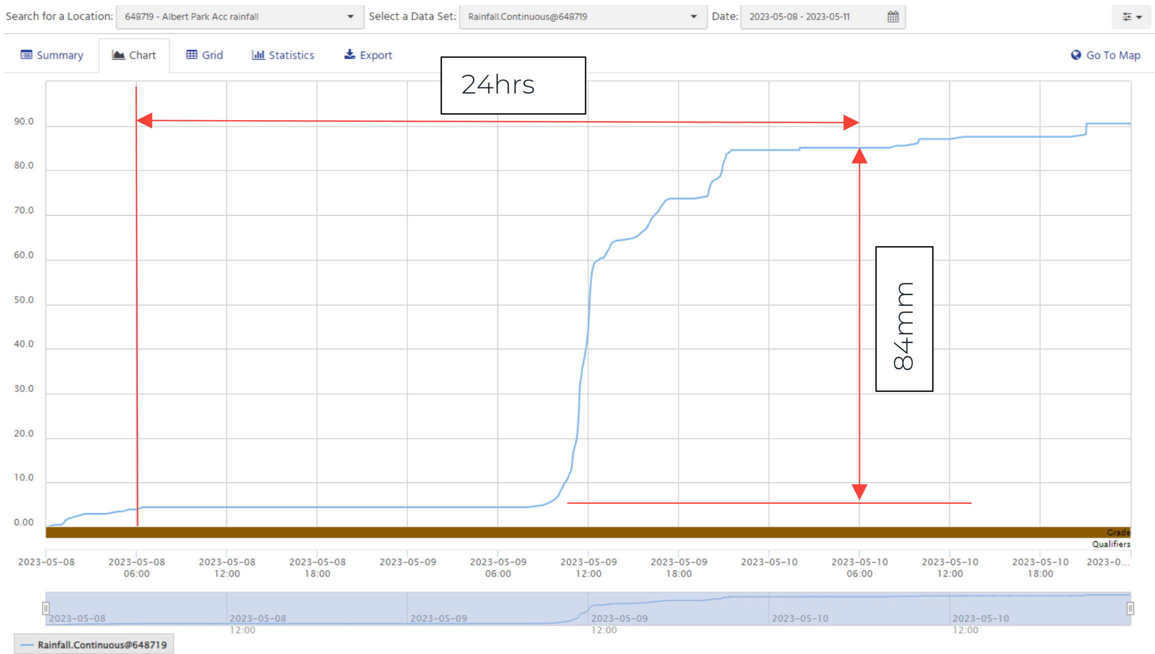
153mm in 55hrs between 00:00 12/02/23 and 07:00 14/12/23, including 136mm in 24hrs between 04:00 13/02/23 and 04:00 14/02/23 (cf. TP108 24 hr Rainfall maps 20yr ARI 140mm). The peak 14min rainfall intensity was ~19mm/hr. While this is not considered a significant peak intensity (<2y ARI for the same critical duration), the duration of the peak lasted for more than 2hrs. Combined with the antecedent conditions relating to the Auckland Anniversary Weekend floods less than two weeks earlier, this could have caused heavily saturated ground and pervious areas unable to absorb the rainfall, resulting in significant runoff.

[3] Auckland ClimateWEB.pdf (niwa.co.nz)



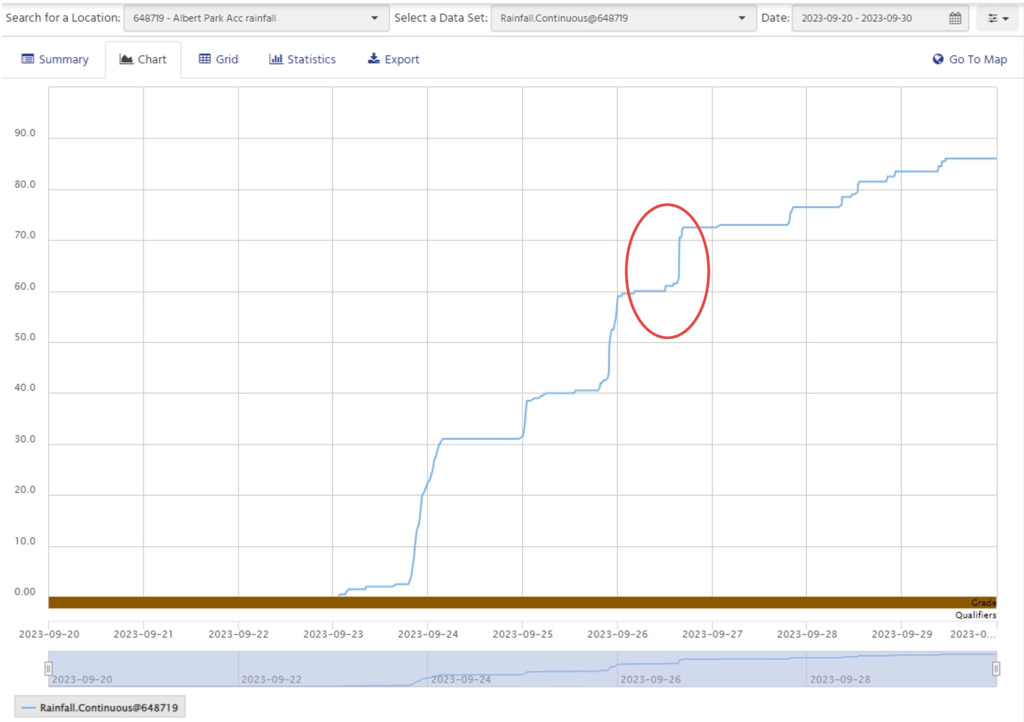
4.3 May 2023 Event

84mm in 24hrs between 06:00 09/05/23 and 06:00 10/05/23 cf. TP108 24hr rainfall depth for 5yr ARI of 100mm. The peak 14min rainfall intensity during this event was ~54mm/hr, similar to the theoretical TP108 5yr ARI peak rainfall intensity of ~57mm/hr for the same critical duration.



4.4 Rainfall leading up to first observation of Sinkhole

Approximately 70mm of rain fell in the three days leading up to the first observation of the sinkhole above the OMS at 79 St Georges Bay Road on 26th September 2023. While this rainfall was not likely to have activated overland flow paths, it would likely have further contributed to subsurface groundwater flows in already saturated ground. Based on photographs provided from the initial site visits immediately following the first observation of the sinkhole, there were several areas of open gravel where the carpark asphaltic concrete had been removed for trenching of the ducts leading to the future transformer. This would have provided an additional route for ponded water to infiltrate into the ground.



5 Summary and Conclusions

Based on the above, it is considered that stormwater may have contributed to the formation of the sinkhole in the following ways:

- There were at least three rainfall events in the first half of 2023 that are likely to have led to the activation of overland flow paths through 79 St Georges Bay Road. These events occurred in January (>100yr ARI), February (~20yr ARI) and May (2-5yr ARI).
- The high total rainfall during the nine months leading up to the formation of the sinkhole is likely to have resulted in saturated ground conditions and more frequent activation of secondary/overland flow paths.
- The existing overland flow paths are obstructed by a solid brick wall near the location of the sinkhole, creating a trapped low point. This would cause water to pond above the location of the sinkhole to depth of ~0.25-0.50m before being able to continue flowing downslope.
- The ponding depth at the trapped low point may have been exacerbated by the construction of a low block wall and planter boxes upslope of the sinkhole location.
- Ponded water at the trapped low point would seep into the ground through weak points in the seal or other pervious areas (e.g. garden beds, gravel areas).
- The existing stormwater network within 79 St Georges Bay Rd and upstream has an approximate capacity to accommodate between a 2yr and 10yr ARI event.
- The existing stormwater network was likely to have been under surcharge pressure during the significant rainfall events in January, February and May 2023. If there were any weak points or defects in the existing DN1350 stormwater pipe that crosses above the OMS upslope of the sinkhole location, water may have exfiltrated from the pipe into the surrounding trench material adding to subsurface groundwater flows.
- The combination of the above would have likely led to significantly elevated local groundwater flows/levels in the vicinity of the OMS.

It is important to note that the stormwater factors above in and of themselves are considered potential contributory factors only and are unlikely to have been the primary cause of the sinkhole formation. While it would have led to elevated subsurface flows there would need to have been a preferential flow path in the soils above the OMS for this to have influenced the formation of the sinkhole. Therefore the above advice needs to be read in conjunction with the other SME assessments (Geotechnical, Tunnels/Structures, and Hydrogeology) to form a complete picture of the likely failure mechanism.

APPENDIX F: 2012 AND 2019 CONDITION ASSESSMENT MEMO

MEMO

TO:	Philip McFarlane
FROM:	Kiran Gokal
ACTIONED:	Amulya Prakash
REVIEWED:	Kiran Gokal
SUBJECT:	Orakei Sewer – CCTV and Laser Footage Review
OUR REF:	W-SL021.02
DATE:	13/02/2024

1. BACKGROUND

WSP has been requested to undertake a condition assessment review of the Orakei trunk sewer. The Orakei main sewer (OMS) is a part of combined network, collecting both wastewater and stormwater. It serves a large part of central Auckland.

On Tuesday 26 September 2023, a contractor who was excavating a trench for a power cable in St Georges Bay Rd noticed a hole in the ground above the OMS and alerted Watercare. The sinkhole grew over the next 24 hours. On 27 September the sewer collapsed, and the OMS became fully blocked.

The location of the trunk sewer assessed is shown in the figure below.

Figure 1-1: Location and alignment of the subject trunk sewer

2. OBJECTIVE

The purpose of this desktop review is to:

- provide an analysis of 2012 and 2019 (pre-failure) CCTV and laser footage of the Orakei sewer to better understand failure modes that may be visible from these records.

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3. SCOPE

The scope of work involves:

- A review of CCTV footage
- A review of the laser scan data
- Assess the defects in accordance with the *Inspection Reporting Code of Australia – WSA 05.*
- Record key observations.

The Assets within the scope are listed in Table 4.1.

The CCTV and laser records provided for the analysis is listed in Table 3.1.

ASSET NO.	INSPECTION YEAR 2019		INSPECTION YEAR 2012	
	CCTV	**Laser**	**CCTV**	**Laser**
10099260 (ORM013J-ORM013)	Yes	Not available	No	Not available
10007049 (ORM014-ORM013J)	Yes	Not available	Yes	Yes
10007050 (ORM014J-ORM014)	Yes	Yes	Yes	Yes
10007051 (ORM015-ORM014J)	Yes	Yes	Yes	Yes
10007052 (ORM016-ORM015	Yes	Yes	Yes	Yes
10007053 (ORM017-ORM016	Yes	Yes	Yes	Yes
10007054 (ORM017-OMR018)	Yes	Yes	Yes	Yes

Table 3.1: Records received for review

4. BUILT CHARECTERISTICS

As-constructed drawing indicate that the subject trunk sewer main:

- was constructed in 1910 & 11.
- was constructed as oviform (egg-shaped)
- mostly constructed from a single skin block at the obvert (upper half) in most sections and precast (assumed reinforced) concrete panels for some sections, and a concrete base (lower half). There were no specific material details of the blocks.
- has cover depths between 5 and 30 m.

— with a concrete lower half that may be mass concrete with some connecting rebar at the upper sections.

A typical excerpt from the drawings is shown below showing a typical cross section of the oviform with the block obvert and a precast obvert. A longitudinal section of the trunk sewer is also shown below.

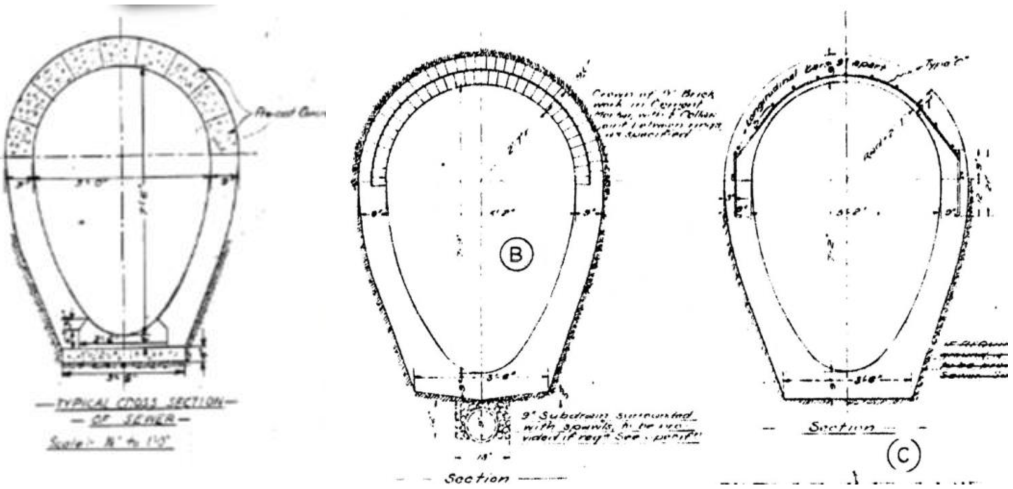


Figure 4-1: Typical block obvert shown on cross section of oviform

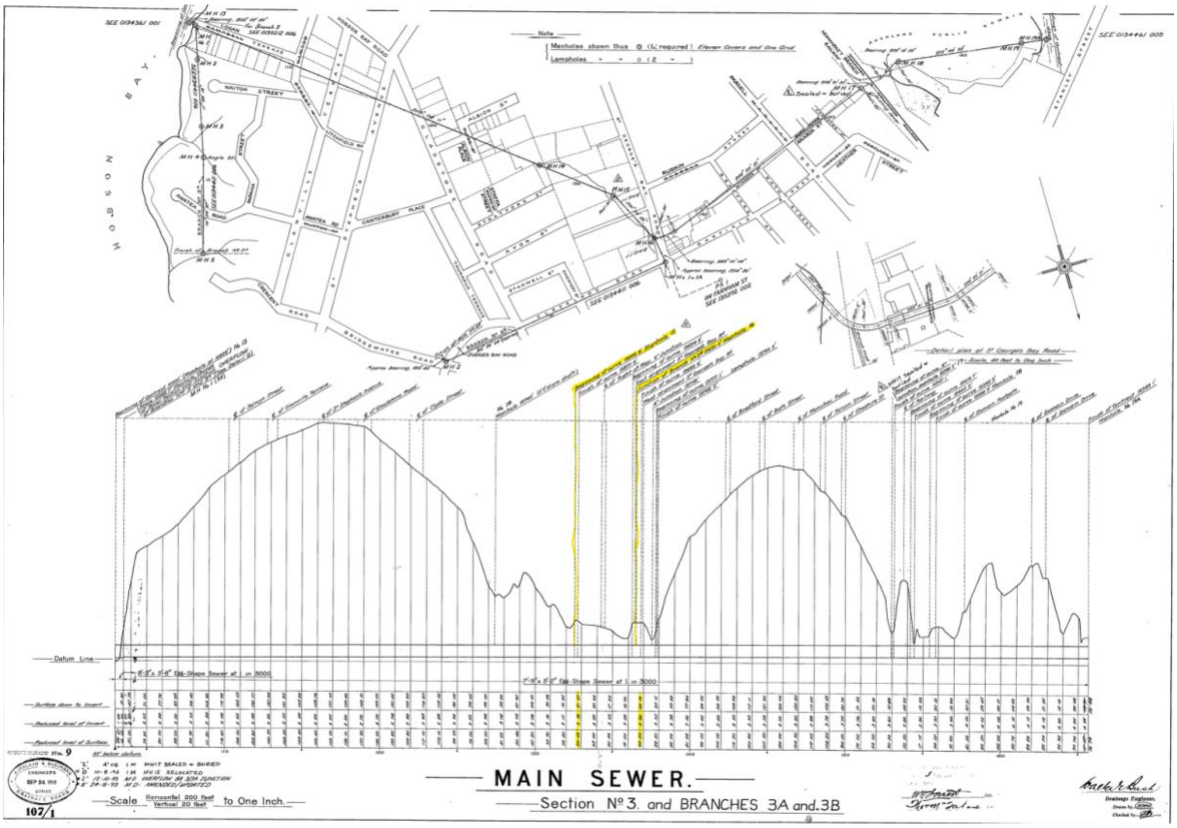


Figure 4-2 Longitudinal section of the sewer main, with failure section between the yellow highlights

Table 4.1 List of Section IDs and details of the Oviform trunk sewer

LINE SEGMENT NUMBER	VINTAGE	MATERIAL	US MH (ASSET ID)	DS MH (ASSET ID)	LENGTH (M)[1]
10099260	1910	Precast concrete panel obvert, mass concrete invert	ORM013J	ORM013	17.60
10007049	1910	Block obvert, mass concrete invert	ORM014	ORM013J	720.17
10007050	1910	Block obvert, mass concrete invert	ORM014J	ORM014	109.99
10007051	1910	Block obvert, mass concrete invert	ORM015	ORM014J	62.56
10007052	1910	Precast concrete arch and mass concrete invert up to 46.62 m from MH 16, then block arch, mass concrete invert	ORM016	ORM015	117.95
10007053	1910	Block and precast concrete panel obvert, mass concrete invert	ORM017	ORM016	501.56
10007054	1910	Precast concrete panel obvert, mass concrete invert	ORM018	ORM017	105.13

[1] This measurement is taken from document "8050_DSORM_Summary Sheet"

5. APPROACH TO CCTV REVIEW

5.1 APPROACH

Conduit condition is commonly assessed in 2 significant areas:

- Structural - the ability of the conduit to maintain its structural integrity (handling internal/external forces and corrosion factors, etc.)
- Service – the ability of the conduit to convey material (sewage flows, water, sludge, air).

Condition assessments are used to analyse defects and score them based on their severity, occurrence, and ability to impact the performance of the conduit. Listed defects are scored according to detailed criteria in the Conduit Inspection Reporting Code of Australia. Peak scores and mean scores are then used independently to nominate a condition grading (from 1 to 5, with 1 being excellent and 5 being very poor) for both service, and structural factors.

The following review of the CCTV and Laser/SONAR data has been completed utilising the criteria set out in the Conduit Inspection Reporting Code of Australia – WSA 05 by the Water Services Association of Australia (WSAA).

5.2 COMMENTS ON FOOTAGE PROVIDED

It should be noted that:

- The resolution of the CCTV footage was very poor, and no camera rotations (panning) were done by the CCTV operator due to the apparent fixed/non-articulated camera system. These quality assurance issues limited the identification and defect severity assessment. It appears that the footage was not obtained to WSAA requirements.
- The CCTV camera system records travelled length. The starting position of the camera may not coincide with the asset connection to the maintenance hole. Therefore, a maintenance hole to maintenance hole camera travel may not provide the exact length of the main or exact location of defects.

- Due to the constant flow of sewerage, the serviceability condition of the invert was identified using Sonar equipped with floating inspection device. The device also included a laser for determining ovality changes along the length of the trunk main.

5.3 OBSERVATIONS

CCTV inspections provide a visual condition assessment of a pipeline's internal surfaces, features, and connections. This process can return adequate service level and structural grading; however, the reader should be aware of the process's limitations. It should be noted that condition scoring is based on the interpretation of the condition assessment engineer. Thus, the reviewer must maintain awareness and make balanced and informed judgements when identifying defects and conducting the assessment.

CCTV inspections provide baseline visual condition assessment of the pipe internals. The true structural integrity of the pipe wall cannot be accurately determined through a visual process alone. Further, the quality of the video and laser footage provided poor quality data that presented challenges in analysing critical elements of the structure. The following observations have been made based on the footage and data provided:

— External defects could not be identified, and it is likely that blocks could externally corrode and erode making them more pervious, increasing the likelihood of infiltration and increased self-weight.

— Structural integrity of the block walls can be compromised due to the corrosion and erosion of the cement mortar that binds the blocks together. The block material itself was not determined from the footage, however it can be borne in mind that blocks tend the be more pervious than bricks which are more compacted material. Consequently, blocks are more susceptible to corrosion and erosion, internally and externally. This failure mode is not easily detected visually, let alone from poor video footage.

— From the vintage drawings, it is unclear if the shape of the blocks is rectangular or trapezoidal like a keystone block used for vintage arch construction. For this assessment, it is assumed that the blocks are rectangular and rely on the binder to maintain its position along the arch.

— Complete loss of the binder, because of corrosion, lead to an initial displacement of the block and ultimately loss of the block from the wall matrix. The available CCTV footage did not provide sufficient evidence of the extent of cement mortar loss. Therefore, observations were made of infiltration/ingress and displaced blocks that suggest loss of the binder.

— Where connecting block surfaces have been displaced, it was assumed that the crack would have occurred between the blocks where the mortar binder has weakened. The cause of the deformation may be surcharge above the obvert of the oviform. The surcharge can result from various factors including groundwater, soil cover, other network infrastructure and foundation of buildings and roads. The estimated number of block displacements are listed in the tables below.

— Where the appearance of mass infiltration or ingress was evident and no clear appearance of missing blocks, it was assumed that mortar was used to fill a non-standard block space. The ingress material would have led to accumulation of slime at those isolated locations. The ingress locations were mostly detected at the horizontal joint between the blocks and the lower concrete base.

— The structural integrity of concrete base walls can be compromised due to the leaching of the cement binder and loss of aggregate material. The leaching of the binder is of particular interest. Once the binder loses its integrity after the corrosion front has passed, the aggregate is exposed. Initially, the aggregate becomes visible on its surface and then progressively projects/protrudes from the remaining surface. The corrosion front would be behind the exposed aggregate, progressing towards the reinforcing (if present) and ultimately the total wall thickness. This was clearly evident on the pre-cast obverts. The initial loss of concrete sections resulting from the loss of binder and aggregate, results in spalled sections. This is particularly prevalent in pre-cast concrete sections of Asset 10007052 and Asset 10007054.

— Lasers were used to assess deviations from the reference shape above the sewage line. This is helpful when reviewing inwards/outwards displacement of blocks or severe spalling of concrete. Where blocks may be corroded and eroded, a loss of thickness occurs, appearing as an outward displacement. The data obtained does not have sufficient resolution to detect the extent of lost cement mortar between the blocks. Without high resolution laser and video footage, only rough visual estimations could be done of these critical structural elements.

6. STRUCTURAL AND SERVICE GRADING (2019 DATA)

The structural and service scores have been calculated and a list of defects and accompanying photos can be found in the Appendix B. These defects were identified through an analysis of the CCTV inspection footage. It is noted that the determination of the defect location and type is impacted by CCTV footage quality, CCTV operator control and reviewer interpretation.

The scoring and grading system, in accordance with WSA 05, is shown in Table 6.1 for the subject assets. Appendix B provides images of the defects.

Table 6.1 Structural and Serviceability grading scores

ASSET ID	PEAK STRUCTURAL SCORE	MEAN STRUCTURAL SCORE	WSAA STRUCTURAL GRADE	PEAK SERVICE SCORE	MEAN SERVICE SCORE	WSAA SERVICE GRADE
10099260	20	19	**5**	5	5	**3**
10007049	80	3	**5**	10	0	**3**
10007050	55	10	**5**	5	0	**1**
10007051	15	8	**5**	10	2	**3**
10007052	25	21	**5**	5	1	**2**
10007053	80	10	**5**	30	0	**3**
10007054	20	21	**5**	5	0	**1**

Common key defects that contributed to the condition grade 5 and increase the risk of failure, are:

- Visible aggregate on the obvert and base concrete panels, suggesting that the corrosion front is deeper than that visible.
- Spalling on the obvert concrete panels resulting from the leaching of the cement and loss of aggregate.
- Isolated locations of reinforcing exposure at the obvert of the pre-cast concrete panels
- Deformation of the blocks along the length of the sewer, found at Asset 10007049, suggesting weakened bond between the mortar and the blocks accompanied by external surcharge.
- Mortar missing predominantly near the horizontal joint between the blocks and the concrete base, resulting in infiltration and ingress of (soil) material.

  The future trend of cement mortar degradation is unlikely to be linear as corrosive moisture accumulates in the pores of the cement mortar (and block). The risk of failure also increases when groundwater accumulates in the pores of the pervious blocks, increasing its weight and circumferential stress on the adjacent mortar and blocks. This can result in an inward displacement of blocks.

  *Note that the deviations could also represent thinned blocks resulting from corrosion and erosion. As mentioned above, outward displacement as reported in the laser report, can represent eroded or thinned blocks resulting from its erosion. Apart from the above measurements, the laser profiling report also shows general 'outward displacement' across the trunk sewer (see Figure 6-1).

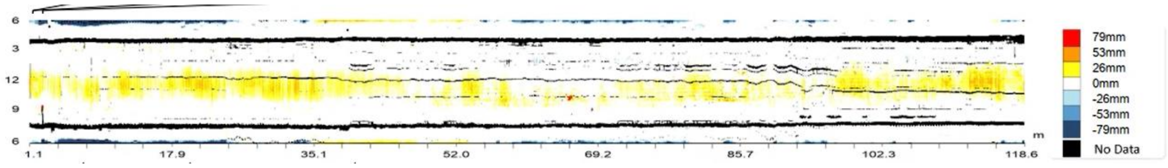


Figure 6-1: General 'outward' displacement/erosion shown by yellow colour across the full length of Asset 10007052

This suggests that block erosion is more likely the failure mode, although block displacements can also occur. This failure mode coincides with the earlier observations and comments made about the pervious construction of blocks as compared to bricks. It is expected that the corrosion and erosion of the blocks would continue at an indeterminable rate, progressing to a point where its structural integrity would be compromised to withstand external loading and internal pressure surcharges. To quantify the structural integrity of the existing blocks, finite element analysis is recommended. Assumptions would be needed to be made about block strength, backfill materials etc.

— The number of locations where the mortar was observed to be either missing totally or moderately is listed below.

ASSET ID	10007049	10007050	10007051	10007052	10007053	10007054
Mortar Missing at no. of Locations	15	42	40	58	39	10
Number of location missing mortar per m	0.02	0.38	0.64	0.49	0.08	0.14

Asset 10007052 has the greatest number of observable mortar loss at the block and concrete interface. The WSAA code increases the weighting score of mortar losses from the conduit obvert to the conduit invert. This is because of the increased physical bearing weight with depth. The weak bond at the conduit spring line (3 and 9 o'clock) can result in instability of the block section constructed above it.

7. STRUCTURAL AND SERVICE GRADING (2012 DATA)

The structural and service scores have been calculated and a list of defects and accompanying photos can be found in the Appendix B. The scoring and grading system, in accordance with WSA 05, is shown in Table 6.1 for the subject assets.

ASSET ID	PEAK STRUCTURAL SCORE	MEAN STRUCTURAL SCORE	WSAA STRUCTURAL GRADE	PEAK SERVICE SCORE	MEAN SERVICE SCORE	WSAA SERVICE GRADE
10099260	No inspection data provided					
10007049	80	2	**5**	10	0	**3**
10007050	6	2	**4**	5	1	**1**
10007051	6	3	**4**	5	1	**1**
10007052	25	6	**5**	5	1	**1**
10007053	80	1	**5**	5	0	**1**
10007054	100	3	**5**	5	1	**1**

Table 7.1 Structural and serviceability grading according to 2012 CCTV analysis

Common key defects that contributed to the condition grades and increasing risk of failure, are:

— Loss and leaching of cement mortar resulting in infiltration at locations, particularly at Asset 10007052 CH61-68m

- An appearance of a spall/crack and/or reinforcing exposure along the concrete base of Asset 10007049 and Asset 10007053
- Roughened concrete base suggesting corrosion and loss of cement binder for the length of the mains
- A longitudinal crack at 1 o'clock (CH 712-717m) of Asset 10007049
- The surface appearance of the blocks did not clearly show that erosion has significantly occurred
- Surface corrosion of the pre-cast obverts
- Hole/partial/full loss of a block at the obvert at CH83m of Asset 10007054.
- The number of locations where the mortar was observed to be either missing totally or moderately is listed below.

ASSET ID	**10007049**	**10007050**	**10007051**	**10007052**	**10007053**	**10007054**
Mortar Missing at no. of Locations	52	24	30	25	18	15
Number of location missing mortar per m	0.07	0.22	0.48	0.21	0.04	0.14

8. KEY COMPARISON BETWEEN THE 2012 AND 2019 INSPECTIONS

A comparison was initially made between the WSAA-based structural condition grade scores between the 2012 and 2019 inspections, as tabulated below.

ASSET ID	2012			2019		
	Peak structural score	**Mean structural score**	**WSAA Structural Grade**	**Peak structural score**	**Mean structural score**	**WSAA Structural Grade**
10099260	No records	No records	No records	20	19	**5**
10007049	80	2	**5**	80	3	**5**
10007050	6	2	**4**	55	10	**5**
10007051	6	3	**4**	15	8	**5**
10007052	25	6	**5**	25	21	**5**
10007053	80	1	**5**	80	10	**5**
10007054	100	3	**5**	20	21	**5**

It is clear that the peak and mean scores (and condition grades) have increased from 2012 to 2019. This suggests that defects have deteriorated more, and the extent has increased across the length of the assets.

Of significance, a review was conducted of the general inwards/outwards laser-based measurements across the full length of the assets inspected in 2012 and 2019. Table 8.2 shows a comparison the extent of outward deviations between the 2012 & 2019 inspections i.e. the extent of erosion. Key observations are:

— The extent of corrosion observed in the 2012 laser profiling reports varied across the pipes inspected. No corrosion was identified on pipes 100007050 & 10007051. Isolated erosion to a maximum depth of 29mm was observed on pipes 10007049 & 10007052. More extensive erosion was observed at the obvert of pipes 10007053 & 10007054 to a maximum depth of 81mm.

— The depth and extent of erosion increased markedly between 2012 & 2019. The maximum depth of corrosion on pipes 100007050 & 10007051 was 54mm and extended across the length of the obvert (whereas no erosion was observed in 2012). 20mm of additional erosion was observed on pipes 10007053 & 10007054 to a maximum of 101mm.

— More extensive erosion was observed on pipe 10007052 i.e., the pipeline that failed in 2023. 125mm of erosion was observed over a short section (less than 1m) near the location of the collapse. Up to 61mm of erosion was observed across the obvert whereas in 2012 only isolated sections of erosion were observed.

Table 8.1 Comparison of Corrosion Identified from 2012 & 2019 Laser Profiling Inspections

ASSET NO. **(MANHOLE NOS)**	**INSPECTION YEAR 2012**	**INSPECTION YEAR 2019**
10099260 (ORM013J-ORM013)	Not available	Not available
10007049 (ORM014-ORM013J)	Max 29mm Only isolated erosion observed	Not available
10007050 (ORM014J-ORM014)	No erosion identified	Max 41mm

		24mm to 35mm recorded across obvert (not as extensive as other sewer sections)
10007051 (ORM015-ORM014J)	No erosion identified	Max 54mm 12mm to 37mm recorded extensively across obvert
10007052 (ORM016-ORM015	Max 24mm Only isolated erosion observed	Max 125mm 27mm to 62mm recorded extensively across obvert
10007053 (ORM017-ORM016	Max 51mm 20mm to 51mm recorded extensively across obvert	Maximum 71mm Similar extent of erosion to 2012
10007054 (ORM017-OMR018)	Max 81mm 29mm to 81mm recorded extensively in obvert of upstream half of pipe	Max 101mm Similar extent of erosion to 2012

Table 8.2 shows a summary of the flat graphs provided in the laser reports along with a colour coded scale represented by positive numbers (outward measurements from original ovality), and negative numbers (inwards measurements from original ovality).

Although the scales are different between the two reports, the colour coding can be used to infer the general condition of the assets.

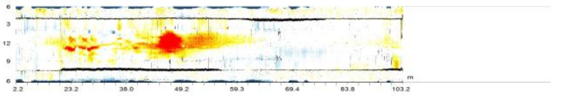
It was observed that there are more outward measurements from the reference shape, more yellows and reds, in 2019 than 2012. This suggests that more outwards displacement/erosion of the inside surface and along the asset's length, has been measured in 2019 when compared to the data obtained in 2012.

Table 8.2 Inspection summary of general inwards/outwards measurements

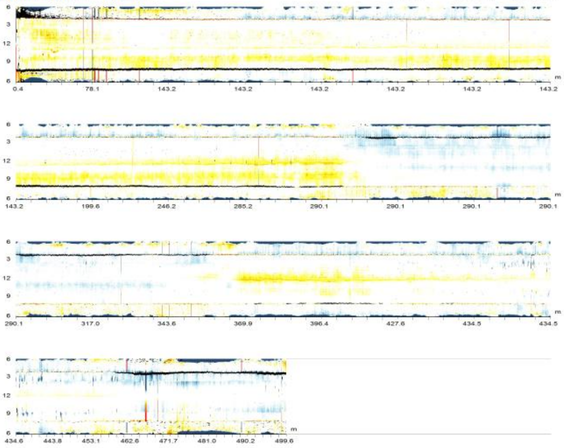
2012 Inspection summary flat graph

2019 Inspection summary flat graph

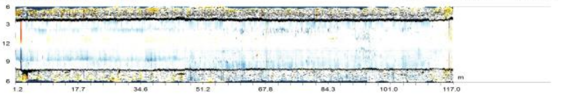
10007054, 1575mm Page 10 CCTV LASER SONAR



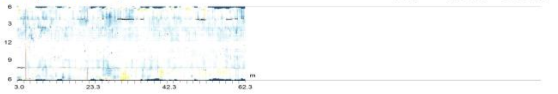
10007053, 1575mm Page 15 CCTV LASER SONAR



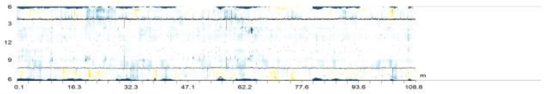
10007052, 1575mm Page 25 CCTV LASER SONAR



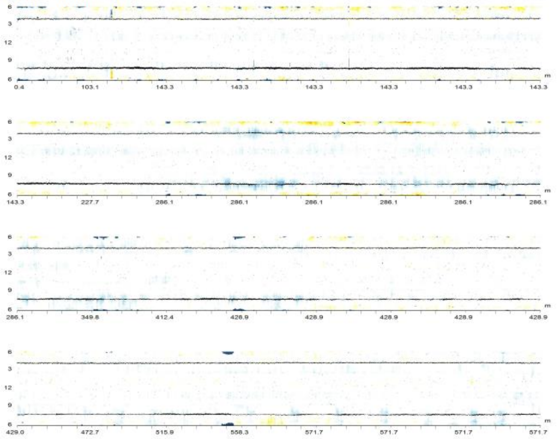
10007051, 1575mm Page 30 CCTV LASER SONAR



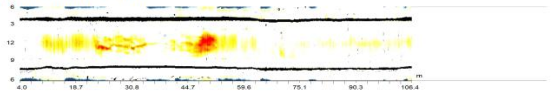
10007050, 1575mm Page 34 CCTV LASER SONAR



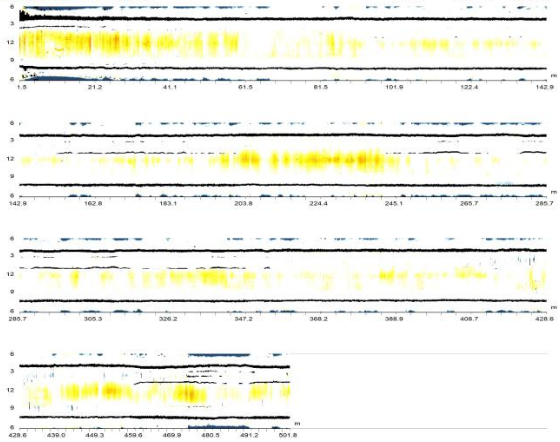
10007049, 1575mm Page 39 CCTV LASER SONAR



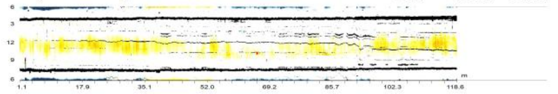
10007054, 1575mm Page 7 CCTV LASER SONAR



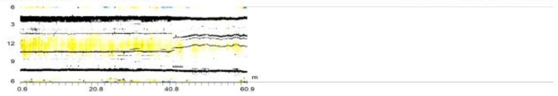
10007053, 1575mm Page 12 CCTV LASER SONAR



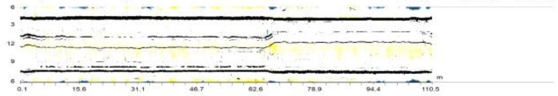
10007052, 1575mm Page 23 CCTV LASER SONAR



10007051, 1575mm Page 28 CCTV LASER SONAR



10007050, 1575mm Page 32 CCTV LASER SONAR



9. CONCLUSION

The following conclusions were drawn:

- Common key defects that were observed and which contribute the increased risk of future failure are:
  - Visible aggregate on the obvert and base concrete panels
  - Spalling on the obvert concrete panels and visible corroded reinforcing
  - Mortar missing predominantly near the horizontal joint between the blocks and the concrete base
  - Erosion of the blocks along the length of the sewer
  - The risk of trunk sewer failure increases when external groundwater accumulates in the pores of the pervious blocks, increasing its weight and circumferential stress on the adjacent mortar and blocks.
  - There is observable mortar loss at the block and concrete interface increase stability concerns of the blocks constructed above it.
  - A non-linear future trend of cement mortar and block thickness degradation is likely as internal corrosive moisture and gases accumulate in the pores of the cement mortar and blocks,
- The structural condition grades based on the WSAA conduit inspection code is 5.
  - The WSAA code suggest that this grade relates to a failure that has occurred or imminent.
  - Apart from the 2023 failure, there were no other reported failures.
  - Given that the visual-based inspection is considered a screening assessment, the WSAA code suggests that an immediate risk assessment and further investigation be undertaken including rehabilitation/renewal.
- The depth and extent of corrosion increased markedly between 2012 & 2019. From the 2019 inspection:
  - 125mm of corrosion was observed over a short section (less than 1m) of pipe 10007052 near the location of the collapse.
  - corrosion to a maximum of 101mm was observed on pipes 10007053 & 10007054.

10. RECOMMENDATIONS

The following actions are recommended:

- Undertake a technical analysis of the oviform structure using finite element analysis (FEA) to confirm structural integrity.
- Factor the findings from the FEA to update likelihood of failure
- Update the risk assessment of the trunk sewer considering the criticality of the main and determine risk appetite for future failure/s
- Undertake an economic options analysis of renewal and replacement methods.
- Depending on the outcomes of the above actions, renew or replace Assets 10007049 to 10007054.

APPENDIX A WSA-05 GRADING THRESHOLDS AND RECOMMENDATIONS

TABLE C5

STRUCTURAL GRADING OF SEWERS

Grading	Description	Appropriate response in normal circumstances[1]	Peak score[2]	Mean score
1	Insignificant deterioration of the sewer has occurred. Appears to be in good condition	No immediate action required—Standard programmed condition assessment	<5	0 – 1.0
2	Minor deterioration of the sewer has occurred. Minor defects are present	No immediate action required—Standard programmed condition assessment	5 – 9	>1 – 3.0
3	Moderate deterioration of the sewer has occurred. Developed defects are present but not affecting short term structural integrity	Monitor with programmed condition assessment for rehabilitation and/or renewal in medium term	10 – 39	>3 – 5.0
4	Serious deterioration of the sewer has occurred. Significant defects are present affecting structural integrity	Take immediate action as appropriate to the defects e.g. temporary supports Immediately undertake risk assessment and further investigate as required. As appropriate to outcomes of above, schedule appropriate action which may include rehabilitation and/or renewal in the short term	40 – 59	>5 – 10.0
5	Failure of the sewer has occurred or is imminent	Take immediate action as appropriate Immediately undertake risk assessment and further investigation, and, as necessary, take appropriate action which may include immediate rehabilitation and/or renewal	≥60	>10

NOTES:

1. The actual action to be taken for any sewer system will depend on the asset management policies and procedures of the asset owner/operator.
2. Rounded to the nearest whole number.

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TABLE C6
SERVICE GRADING OF SEWERS

Grading	Description	Appropriate response in normal circumstances[1]	Peak score[2]	Mean score
1	No or insignificant loss of hydraulic performance has occurred. Appears to be in good condition and there is little likelihood of sewage pollution[3]	No immediate action required—Standard programmed condition assessment	<5	0 – 1.0
2	Minor defects are present causing minor loss of hydraulic performance and/or minor likelihood of sewage pollution	No immediate action required—Standard programmed condition assessment	5 – 9	>1.0 – 3.0
3	Developed defects are present causing moderate loss of hydraulic performance and/or moderate likelihood of sewage pollution	Take immediate action as appropriate to the defect e.g. cleaning, root cutting, point repair Monitor with programmed condition assessment for rehabilitation and/or renewal in medium term	10 – 39	>3 – 5.0
4	Significant defects are present causing serious loss of hydraulic performance and/or significant likelihood of sewage pollution	Take immediate action as appropriate to the defect e.g. root cutting, point repair, vermin treatment Immediately undertake risk assessment and further investigate as required As appropriate to outcomes of above, schedule appropriate action which may include rehabilitation and/or renewal in the short term	40 – 59	>5 – 10.0
5	Failure of the sewer or pollution of the environment has occurred or is imminent	Take immediate action as appropriate e.g. temporary support Immediately undertake risk assessment and further investigation, and, as necessary, take appropriate action which may include immediate rehabilitation and/or renewal	≥60	>10

NOTES:

1. The actual action to be taken for any sewer system will depend on the asset management policies and procedures of the asset owner/operator.
2. Rounded to the nearest whole number.
3. Pollution may occur through a sewage spill (overflow), exfiltration through a defect or cross-connection between the sewerage and stormwater systems.

APPENDIX B EXAMPLES OF DEFECT APPEARANCES

<table>
<tr><th>Segment: 10099260</th><th>Date of Inspection: 12/02/2019</th></tr>
<tr><td>


</td><td>


</td></tr>
<tr><td>Surface damage, spalling (only at obvert) and aggregates visible, full circumference above sewage, for the full length of main, from CH:2.85m to 17.6m</td><td>Longitudinal joint at 2o'clock of CH: 5.97m</td></tr>
<tr><td>


</td><td>


</td></tr>
<tr><td>Roughening seen throughout the length of the pipe</td><td>Grease deposition at 2 o'clock and at the spring line of CH: 8.99m</td></tr>
</table>

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Segment: 10007049	Date of Inspection: 11/02/2019

Infiltration seen at 11 o'clock of CH: 1.39m

Infiltration at 1 o'clock of CH:5.13m

Infiltration staining at 9 o'clock of CH: 8.14m

Calcification encrustation observed at the obvert of CH: 15.17m

Longitudinal loss of mortar found at 1 o'clock from CH: 17.15m to 20.38m, allowing ingress

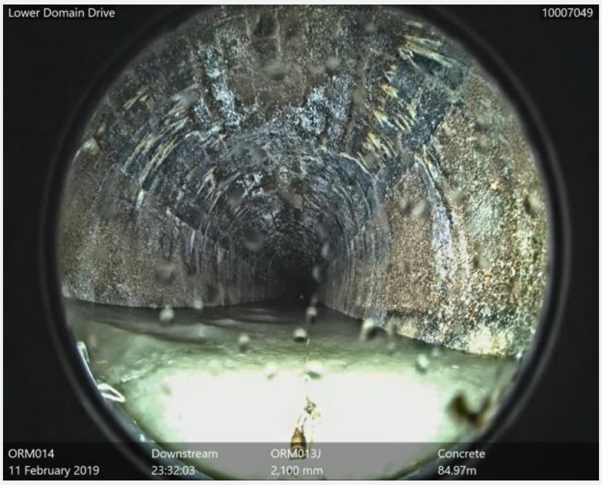
Longitudinal loss of mortar, deformation, and brick separation and found at 12 o'clock from CH: 21.92m to 25.06m

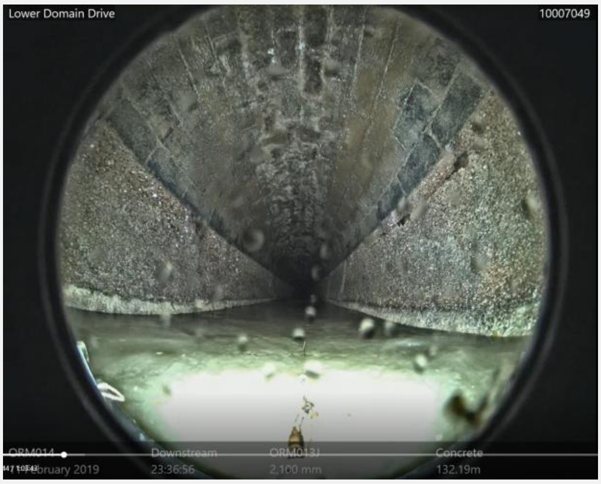
Longitudinal loss of mortar seen at 2 o'clock of CH: 25.06m, allowing ingress

Hole/opening in blocks resulting in mass infiltration/ingress at 3 o'clock of CH: 54.19m

Hole/opening in blocks resulting in mass infiltration/ ingress at 3 o'clock of CH: 52.35m

Hole/opening in blocks resulting in mass infiltration/ ingress at 3 o'clock of CH: 61.95m



Hole/opening in blocks resulting in mass infiltration/ ingress at 3 o'clock of CH: 62.86m	Calcification and infiltration staining is seen at the obvert of CH: 84.97m
	
Mortar missing at 3 and 9 o'clock at CH:132.19m	Possible reinforcement is visible at 4 o'clock at CH: 139.42m
	
Encrustation at the obvert at CH:143.02m	Mortar missing at 3 and 9 o'clock (interface between the concrete and blocks) at CH: 160.79m
	
Mortar is missing at 3 and 9 o'clock of CH: 166m	Mortar missing at 9 o'clock of CH: 169.93m

Mortar missing at 3 and 9 o'clock of CH: 222.94m

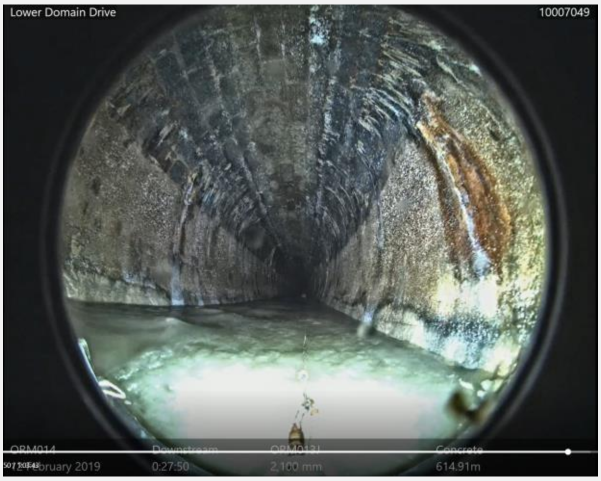
Mortar missing at 3 o'clock of CH: 243.15m

Mortar missing at 3 o'clock of CH:256.11m

Mortar missing at 3 o'clock of CH: 312.28m

Hole/opening in blocks resulting in mass infiltration/ ingress at 2 o'clock of CH: 382.57m

Mortar missing at 10 o'clock of CH: 488.53m

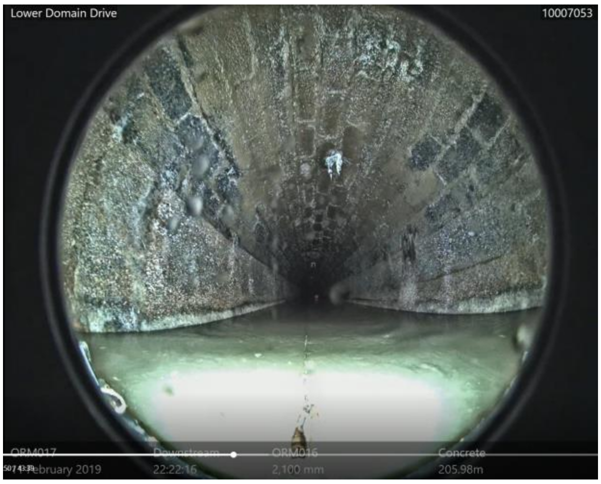
	
Mortar missing at at 4 o'clock of CH: 511.92m	Dripping infiltration seen at the obvert of the pipe at CH: 596.13m and 598.23m
	
Hole/opening in blocks resulting in mass infiltration/ ingress at 3 o'clock of CH: 614.91m	Tap root intrusion from 10 to 3 o'clock of CH: 709.85m

Segment: 10007050	**Date of Inspection: 11/02/2019**
	
Mass ingress at block/concrete joint at 9 o'clock of CH: 20.90m	Mass ingress at block/concrete joint at 9 o'clock of CH: 36.20m
	
Loss of mortar at 3 o'clock of CH: 44.78m	Mass ingress at block/concrete joint at 9 o'clock of CH: 48.18m
	
Loss of mortar at at 3 o'clock from CH: 51.62m to 55.42m	Mass ingress at block/concrete joint at 9 o'clock of CH: 58.02m

	
Mass ingress at block/concrete joint at 3 and 9 o'clock of CH: 64.42m	Mass ingress at block/concrete joint at 3 o'clock of CH: 102.45m

Segment: 10007051	Date of Inspection: 11/02/2019
	
Fine root intrusion from 10 to 2 o’clock of CH: 5.42m	Infiltration/ingress, loss of mortar and staining at 9 o’clock of CH: 25.80m
	
Infiltration/ingress, loss of mortar and staining at 9 o’clock of CH: 50.87m	Fine root intrusion from CH: 50 to 61.73m

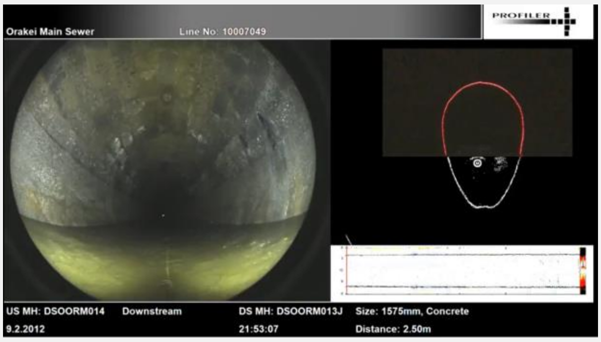
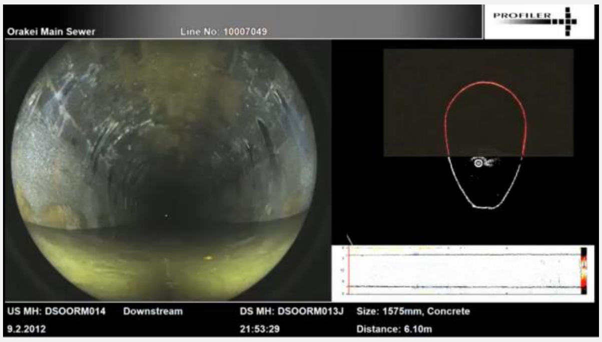
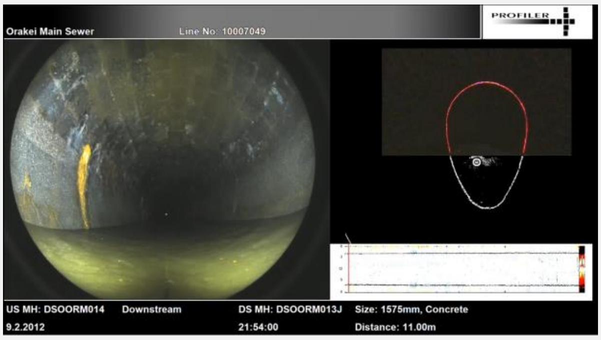
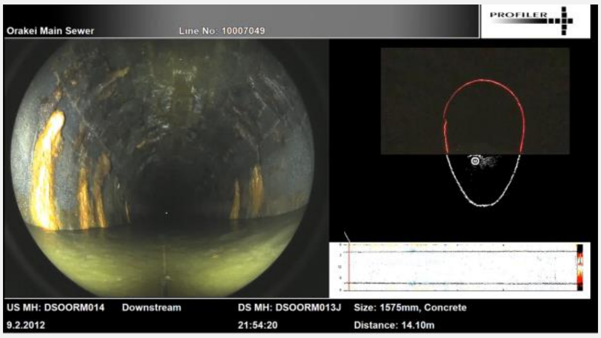
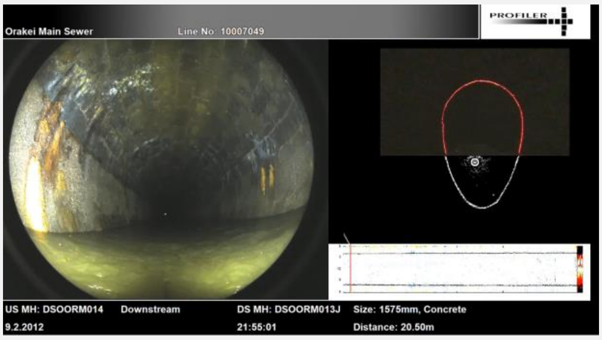
Segment: 10007052	Date of Inspection: 11/02/2019
	
Aggregate visible throughout the length of the pipe with exposed aggregates and spalling at the concrete obvert	Infiltration/ingress, loss of mortar and ingress staining at 3 o'clock of CH: 23.09m
	
Fine root intrusion with loss of mortar at 11 and 1 o'clock, ingress at 3 and 9 o'clock at CH: 55.30m	Fine root intrusion with loss of mortar at 11 and 1 o'clock, ingress at 3 and 9 o'clock from CH: 55.30m to 98.36m
	
Fine root intrusion, ingress with loss of mortar at 9 and 1 o'clock of CH: 107.76m	

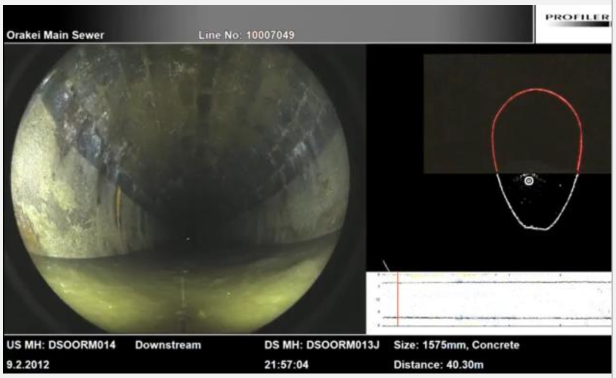
Segement: 10007053	Date of Inspection: 11/02/2019
	
Infiltration and loss of mortar at the obvert at CH: 102.48m, visible aggregate across full length of concrete base	Root intrusion at the obvert of CH:151.75m
	
Root intrusion at the obvert of CH: 205.98m	Infiltration staining, loss of mortar at 3 o'clock of CH: 305.10m
	
Mortar missing at 9 o'clock of CH:315.73m	Root intrusions with infiltrations at missing mortar between blocks at 3 o'clock of CH: 428.11m

 	
Dripping infiltration at 1 o'clock of CH: 432.03m	Spalling and rebars exposed at the obvert from CH:444.48m to 487.68m

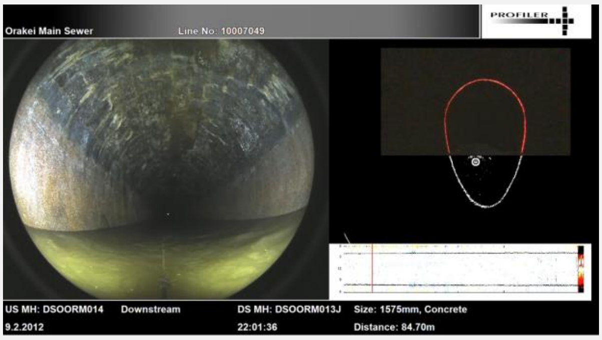
Segement: 10007054	**Date of Inspection: 11/02/2019**
	
Spalling at the obvert throughout the length of the pipe	

APPENDIX C EXAMPLES OF DEFECT APPEARANCES (2012)

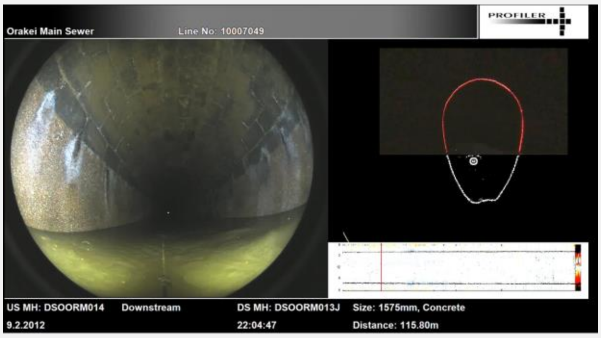
Segement: 10007049	**Date of Inspection: 09/02/2012**
	
Infiltration at 3 o'clock of CH: 2.5m	Infiltration (evident lime leaching) at 3 o'clock of CH: 6.1m
	
Infiltration and loss of mortar at 9 o'clock of CH: 11m	Infiltration at 9 o'clock of CH: 12m, resulting from loss of mortar
	
Infiltration and loss of mortar at 9 and 3 o'clock of CH: 14.1m. Encrustation seen at the obvert of the conduit, however no erosion of the block is seen.	Infiltration at 9 o'clock of CH: 20.5m, resulting from loss of mortar



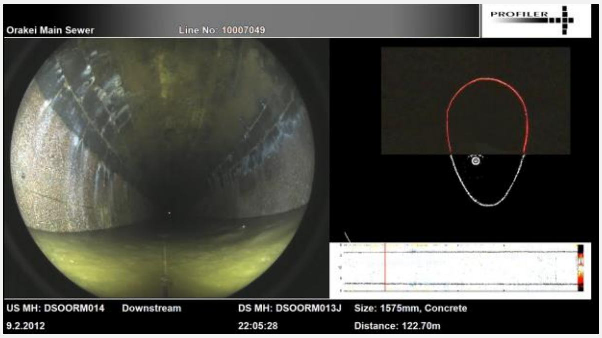
Encrustation seen at the obvert of the conduit, however no erosion of the block is seen



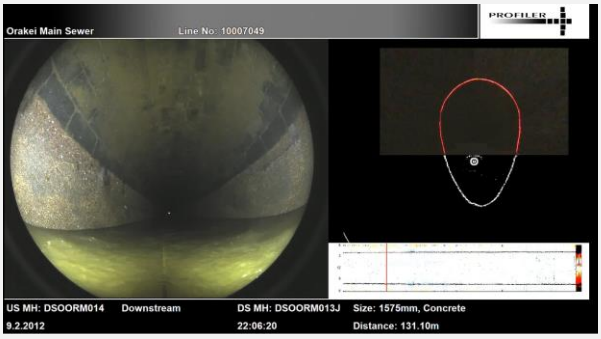
Infiltration and loss of mortar from 9 to 3 o'clock of CH: 84.7m.



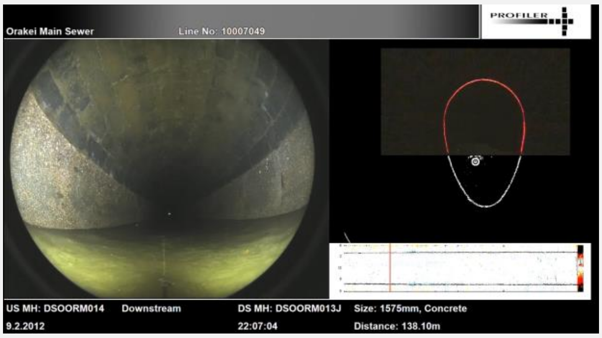
Roughening seen from visual spectrum of 3 to 9 o'clock, throughout the conduit, (evident lime leaching)



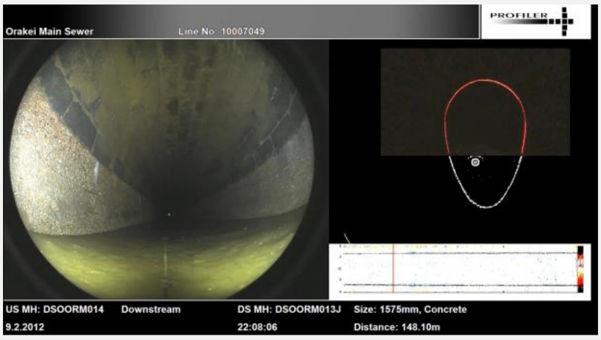
Lime leaching through the mortar is evident throughout the conduit form 9 to 3 o'clock



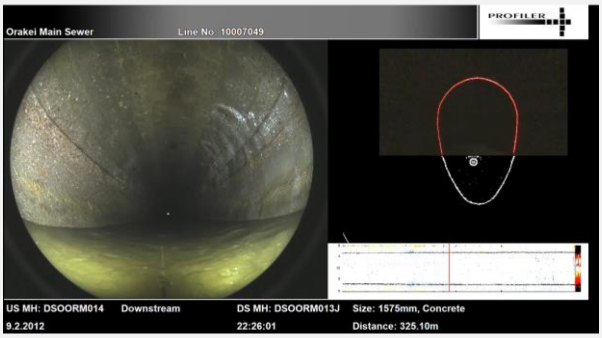
Minor spalling seen at 3 o'clock of CH: 131.10m



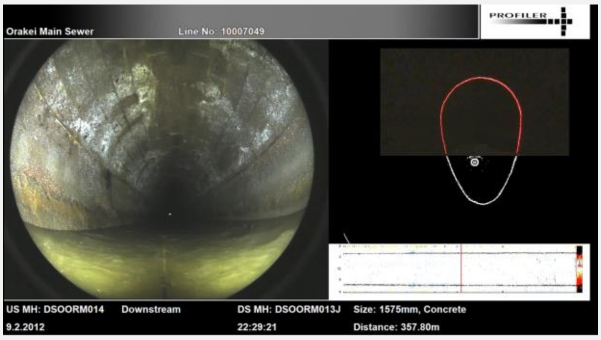
Spalling/reinforcing at 3 o'clock of CH:138.1m



Spalling/reinforcing appears exposed through corroded concrete at 3 and 9 o'clock of CH:148.1m



Lime leaching through the mortar is evident throughout the conduit at 3 o'clock

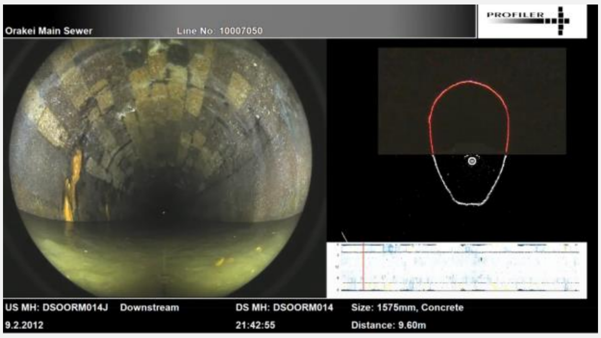
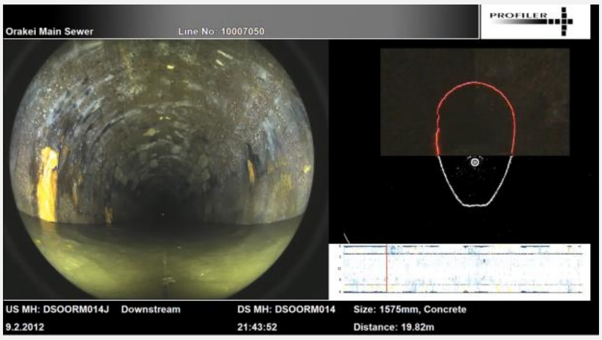
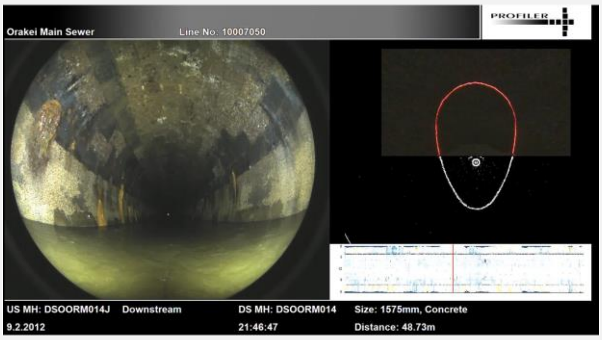
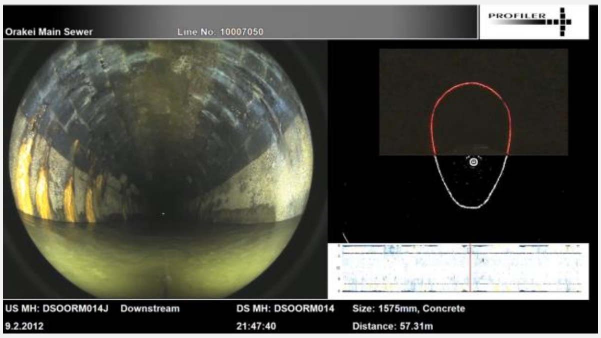
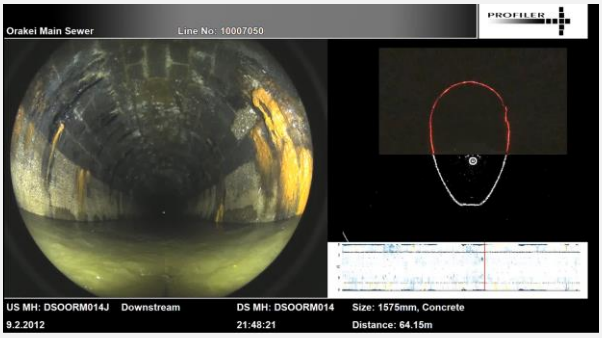


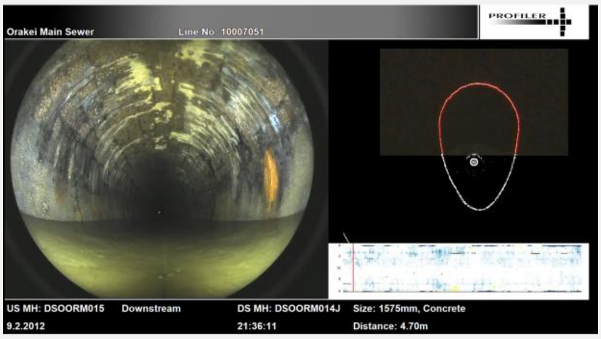
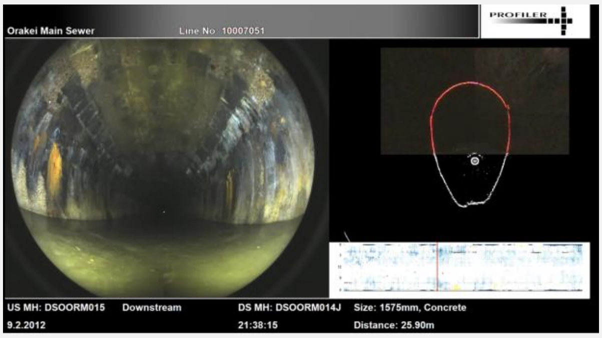
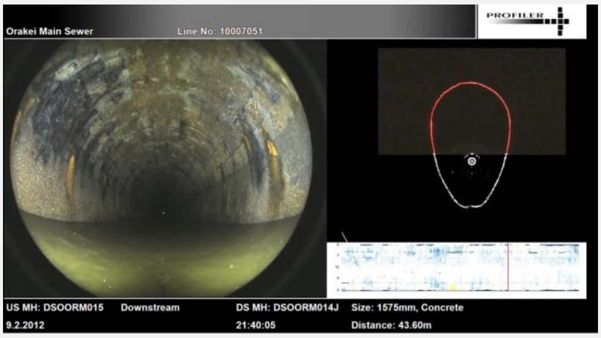
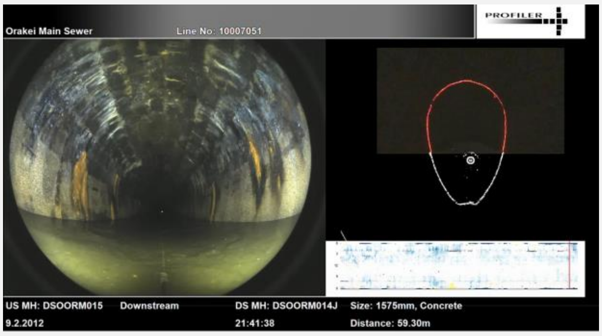
Lime leaching through the mortar is evident throughout the conduit form 9 to 3 o'clock

Lime leaching through the mortar is evident throughout the conduit form 9 to 3 o'clock

Root intrusion from 9 to 3 o'clock from CH: 704.6m to 705.6m

Longitudinal crack, assumed at mortar joint, observed at 1 o'clock from CH: 712m to 717.6m

Segement: 10007050	**Date of Inspection: 09/02/2012**
	
Infiltration and loss of mortar seen at 9 o'clock of CH:9.6m	Infiltration and loss of mortar at 9 and 3 o'clock of CH: 19.82m. Encrustation seen at the obvert of the conduit, however no erosion of the block is observed.
	
Infiltration and loss of mortar seen at 9 o'clock of CH:35.14m	Infiltration and loss of mortar at 9 and 3 o'clock of CH: 48.73m. Encrustation seen at the obvert of the conduit, however no erosion of the block is seen.
	
Infiltration and loss of mortar seen at 9 o'clock of CH:57.31m	Infiltration and loss of mortar seen at 3 o'clock of CH:64.15m

Segement: 10007051	**Date of Inspection: 09/02/2012**
	
Lime leaching through the mortar is evident throughout the conduit form 9 to 3 o’clock	Lime leaching through the mortar is evident throughout the conduit form 9 to 3 o’clock
	
Efflourecence observed at 9 o’clock of CH: 43.60m	Lime leaching through the mortar is evident throughout the conduit form 9 to 3 o’clock

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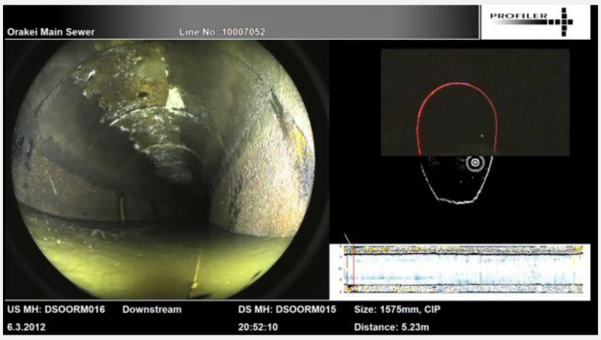
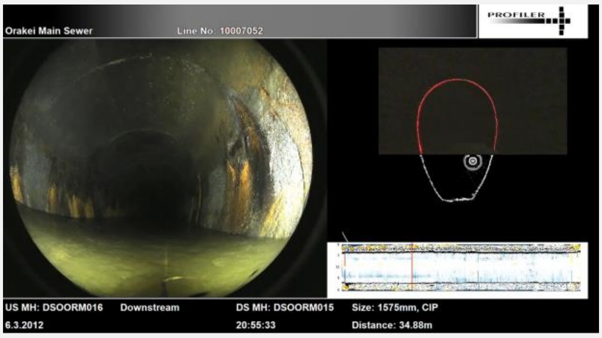
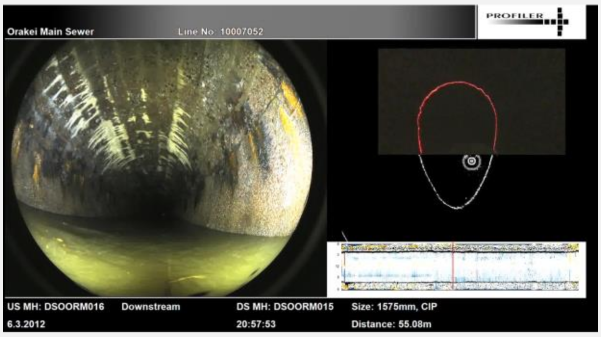
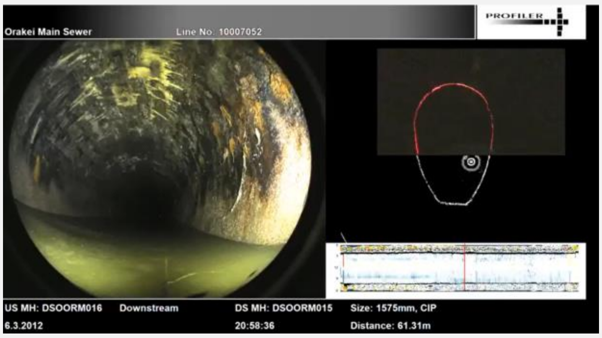
<tr><td>
</td><td>
</td></tr>
<tr><td>Surface damage at the obvert is seen throughout the precast section. However, the surface has not breached until the visibility of aggregates. Hence this defect can be considered as roughening (CH:0 to CH:46.34m)</td><td>Infiltration between blocks at 3 o'clock of CH: 34.88m</td></tr>
<tr><td>
</td><td>
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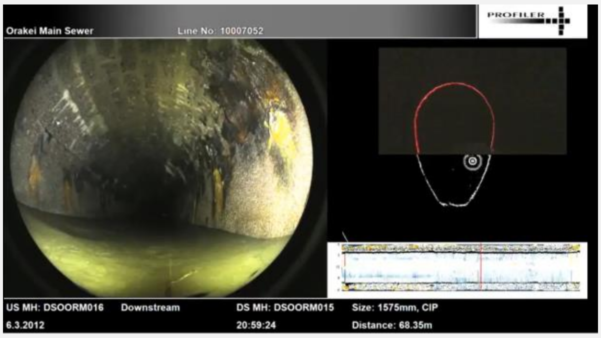
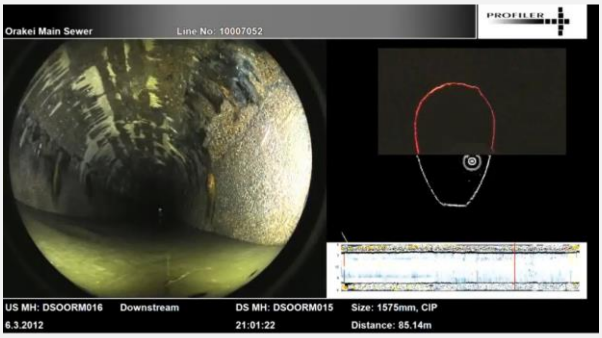
<tr><td>Lime leaching through the mortar is evident throughout the conduit form 9 to 1 o'clock</td><td>Mortar loss observed at 3 o'clock of CH: 61.31m</td></tr>
<tr><td>
</td><td>
</td></tr>
<tr><td>Mortar loss observed at 3 o'clock of CH: 68.35m</td><td>Lime leaching through the mortar is evident throughout the conduit form 9 to 12 o'clock</td></tr>
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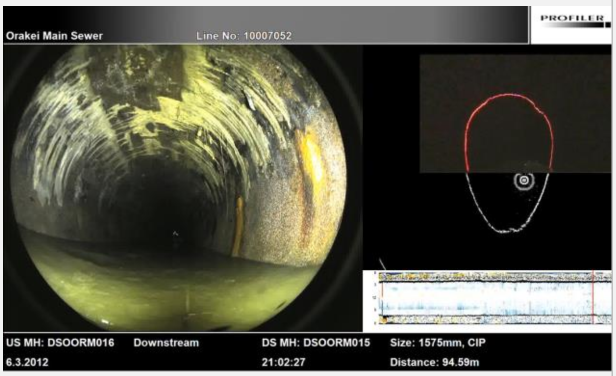
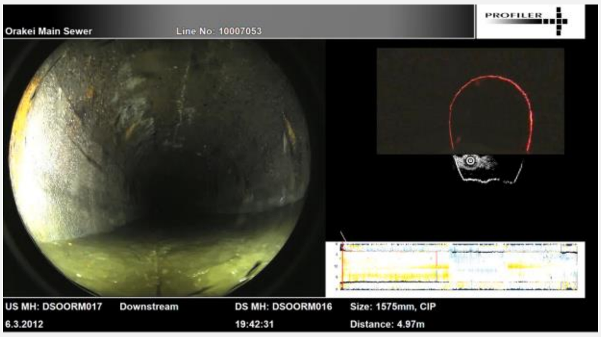
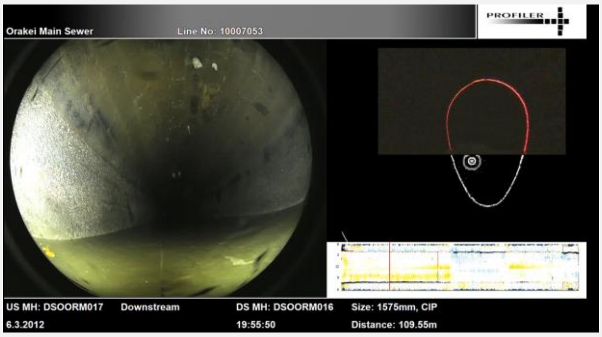
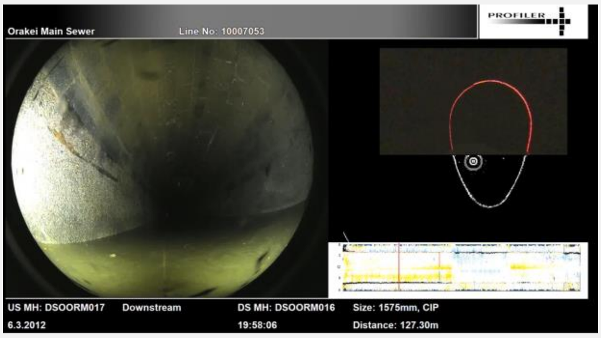
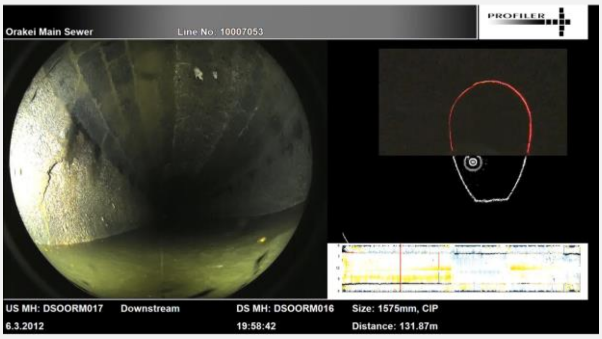
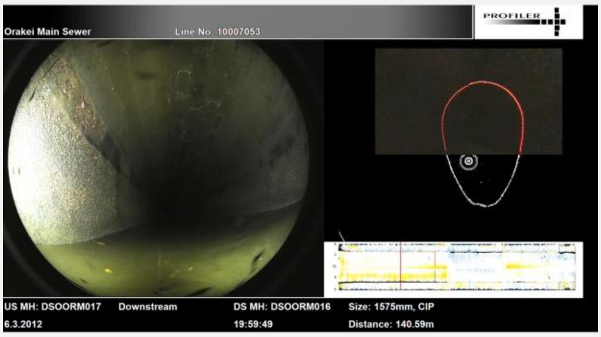
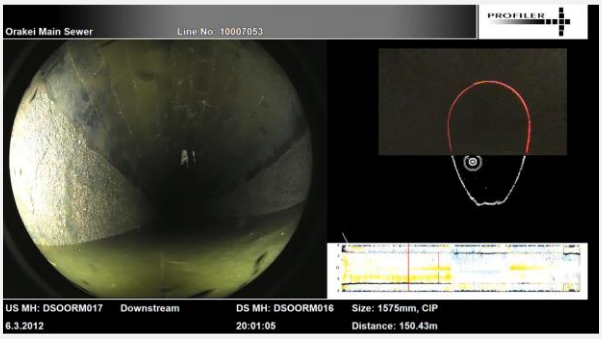
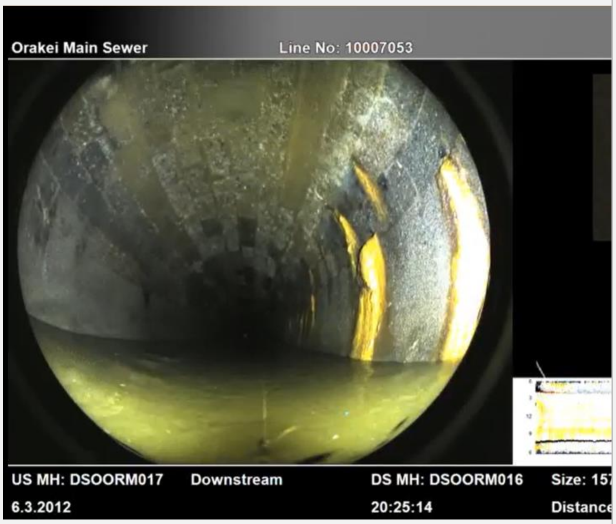
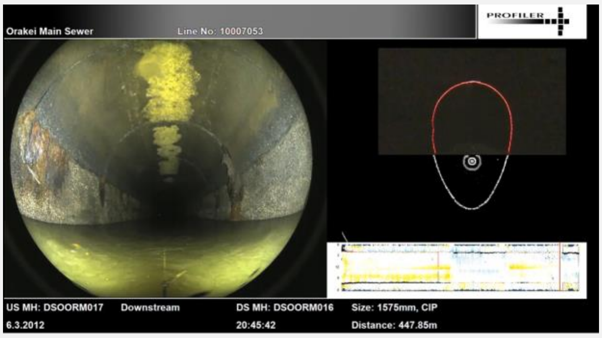
	
Lime leaching through the mortar is evident throughout the conduit form 9 to 3 o'clock	

Segement: 10007053	**Date of Inspection: 06/02/2012**
	
Longitudinal crack (between blocks) observed at the obvert from CH: 4.97m to CH:7.51m	Encrustation seen at the obvert of CH: 109.55 m
	
Minor spalling/reinforcing expsoure at 9 o'clock of CH: 127.3 m	Spalling observed at 9 o'clock of CH: 131.87m
	
Blocks roughening at 9 o'clock of CH: 140.59m	Circumferential cracks from 7 to 9 o'clock of CH: 150.43m

	
Infiltration and loss of mortar seen at 3 o'clock of CH:303.4m	Surface damage at the obvert is seen throughout the precast section. However, the surface has not breached until the visibility of aggregates. Hence this defect can be considered as roughening (From CH:447.85m to 455m)

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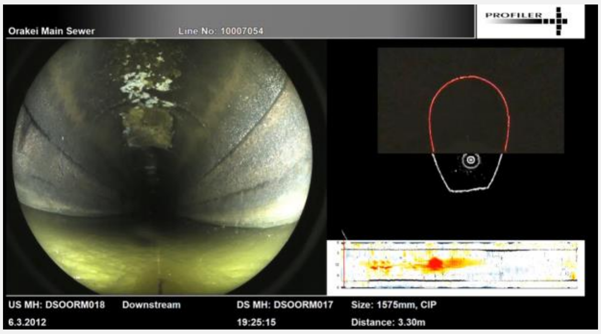
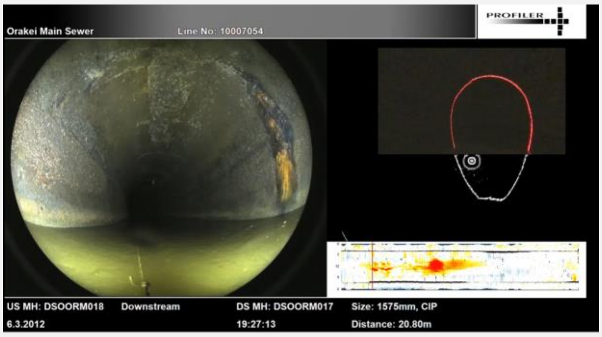
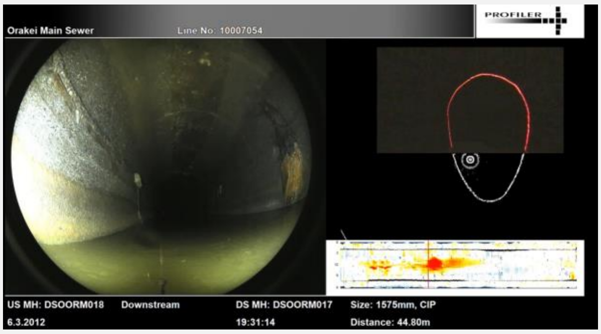
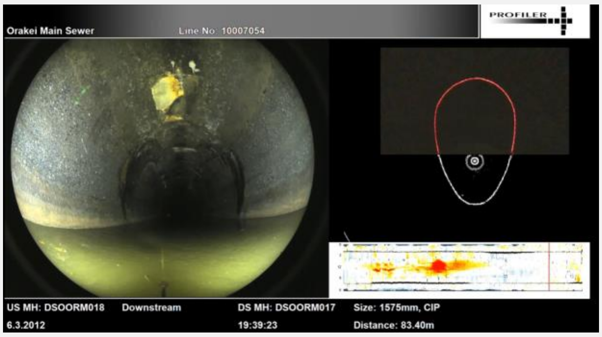
<tr><td>
</td><td>
</td></tr>
<tr><td>Surface damage at the obvert is seen throughout the precast section. However, the surface has not breached until the visibility of aggregates. Hence this defect can be considered as roughening (From CH:0m to CH:20m)</td><td>Localised aggregates and mortar missing with infiltration observed from 11 to 4 o'clock of CH: 20.8m</td></tr>
<tr><td>
</td><td>
</td></tr>
<tr><td>Localised aggregates and mortar missing from 2 to 4 o'clock of CH:44.8m</td><td>Hole or partial or full loss of block at the obvert at CH: 83.4m</td></tr>
</table>

APPENDIX G: POST-COLLAPSE DRONE FOOTAGE MEMO

Memorandum

To	Appendix to Main Report
Copy	
From	Philip McFarlane
Office	Auckland
Date	9 January 2024
File/Ref	W-SL021.02
Subject	Orakei Main Sewer - Observations from Drone Inspection

1 Background

WSP has been requested to undertake a review to identify the likely cause of the failure of the Ōrākei Main Sewer (OMS) that occurred in the vicinity of 79 St Georges Bay Road in September 2023.

A drone was flown through the pipe on 9 October 2023 entering at manhole 15 proceeding up to the collapsed section and then downstream to manhole 14.

This memo describes observations from that inspection.

2 Observations

The section between manhole 15 and the collapse was in noticeably worse condition that observed in from the 2019 CCTV inspection.

Blocks were missing adjacent to manhole 15, refer Figure 1. There was also a block missing part way between manhole 15 and the collapsed section, refer Figure 2. These were not observed during the 2019 CCTV.

Exposed aggregate is clearly visible in the block arch and the concrete base, refer Figure 3.

There appears that two different types of blocks have been used, refer Figure 4. Most are brown with exposed aggregate, but others are grey. The grey blocks appear to have sustained less erosion.

The pipe between manhole 15 & 14 appears to be in better condition. The condition appears to be broadly similar to that observed in the 2019 CCTV.

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Figure 1 – Missing blocks adjacent to Manhole 15 (view is looking upstream)

Figure 2 – Missing block between Manhole 15 and collapse (view is looking upstream, missing block is at 10 o'clock)

Figure 3 – View of Exposed Aggregate observed between Manhole 15 and the Collapsed Section

Figure 4 – View showing different blocks i.e., grey and brown blocks

Figure 5 – General Condition of Sewer Between Manhole 15 & Manhole 14

APPENDIX H: POST-COLLAPSE VISUAL OBSERVATION MEMO

Memorandum

To	Appendix to Main Report
Copy	
From	Philip McFarlane
Office	Auckland
Date	9 January 2024
File/Ref	W-SL021.02
Subject	Ōrākei Main Sewer - Observations from Inspection of the OMS

1 Background

WSP has been requested to undertake a review to identify the likely cause of the failure of the Ōrākei Main Sewer (OMS) that occurred in the vicinity of 79 St Georges Bay Road in September 2023.

This memo describes observations of the collapsed section of the OMS undertaken in November and December 2023.

2 Exposure of the OMS

The collapsed section of the OMS was exposed during November 2023, and it was made safe for personnel to undertake inspection and repair works.

This involved placing a large trench shield over the hole that had formed above the OMS, refer Figure 1. The material under the shield was then progressively excavated so that the shield settled down to the level of the OMS.

Reinforced concrete corbels were constructed to support the edges of the collapsed section and the block arch within the corbels was removed to provide a rectangular opening, refer Figure 2.

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Figure 1 Trench Shield Installed over Collapsed Section of OMS

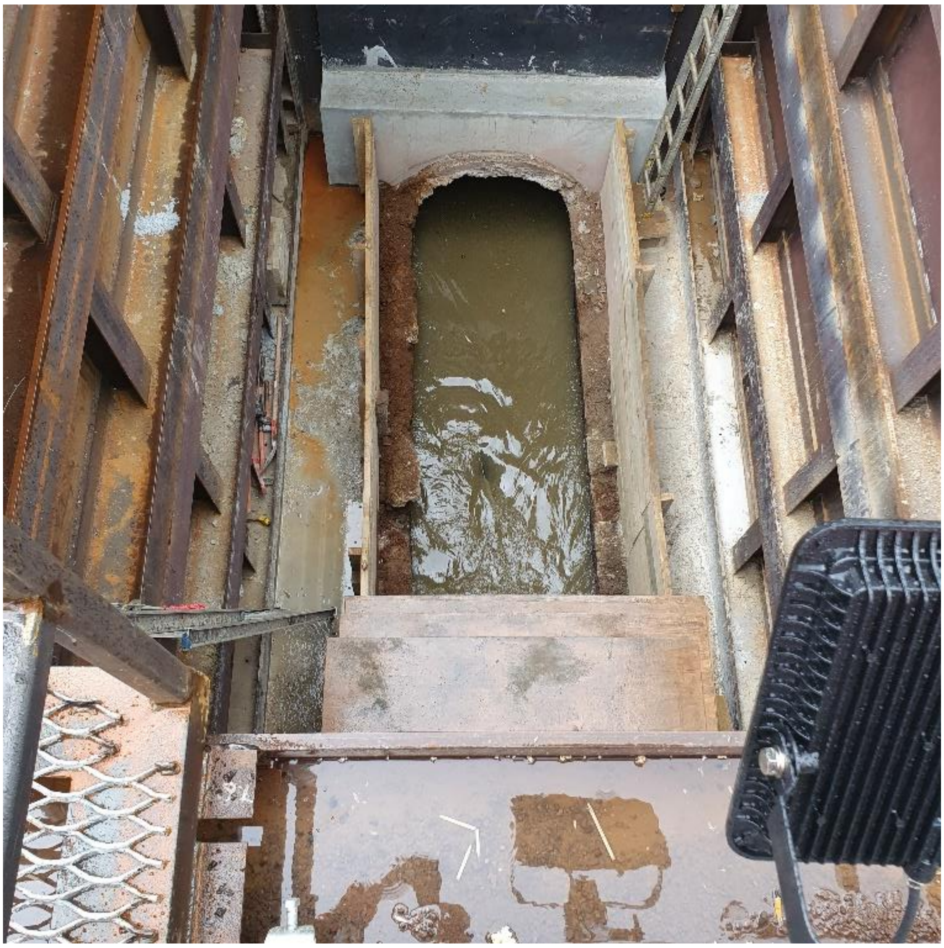

Figure 2 Looking down at OMS after installation of corbels and removal of block arch

3 Observations

3.1 General

It was confirmed that the OMS was constructed from two parts, i.e., a cast in place concrete base and a single layer block arch.

The block arch had collapsed between 9 o'clock and 1 o'clock over a length of 3.5m, refer Figure 3.

Figure 3 Collapse section looking upstream

3.2 Concrete Layer

The block arch was covered by a layer of concrete which was not shown on the as-builts. It was not possible to confirm how far this layer extended along the OMS. It was assumed however that the concrete was localised and did not extend over the full length of the OMS.

The concrete layer was between 50mm and 120mm thick at the centre of the arch, refer Figure 4, Figure 5 and Figure 8. But at the edges where the arch joined onto the cast in-situ concrete base, the concrete layer was less than 30mm thick, refer Figure 6.

3.3 Blocks

Two types of blocks were observed.

Many of the blocks appeared to have been constructed from competent concrete, albeit the concrete is likely to be weaker than what would be used for modern blockwork. The blockwork appeared to be more competent than that reported in the Preliminary Design Report for the Stanley St rehabilitation, which was undertaken in 2014, e.g. there were less voids.

These blocks were approximately 130mm thick, refer Figure 4 & Figure 5. This is approximately 100mm less than the 9inch thick blocks shown on the as-built drawing.

There was a layer of weak corroded concrete on the inside of the blocks about 30mm thick. The brown line that can be seen on Figure 5 indicates the extent of corrosion.

However, in other cases, as shown in Figure 7 and Figure 8, the blocks were very weak and crumbled easily when disturbed. It wasn't possible to determine the extent of these weak blocks.

Figure 4 Typical block arch section in-situ

Figure 5 Typical block arch section after removal. Note the brown line indicating the extent of corrosion on the inside face (see red arrow).

Figure 6 Measurement of concrete layer at edge of block arch

Figure 7 Weak blockwork in-situ

Figure 8 Weak blockwork after removal

4 Commentary

It is unclear why some of the blockwork was constructed from very weak concrete.

One explanation is that when the OMS was constructed this location was used an access point for building the tunnel. When construction was completed, equipment was brought out through the opening and the arch formed from the outside. The weak section might have been a make-up piece constructed onsite to enable the arch to be closed.

This has not been able to be confirmed, but it is plausible given that at the time the OMS was constructed the area was a natural low point.

APPENDIX I: GROUNDWATER MEMO

Memorandum

To	Philip McFarlane
Copy	Eric van Nieuwkerk
From	Louise Soltau
Office	Auckland
Date	14 December 2023
File/Ref	W-SL021.02
Subject	Groundwater Perspective on the Collapse of the Ōrākei Sewer

1 Introduction.

This memorandum discusses the likely influence of groundwater on the collapse of the Ōrākei Mains Sewer (OMS) at 79 St Georges Bay Rd, Parnell on 26 September 2023 (Figure 2). It is likely that the infiltration of rainwater causing erosion of soil above the OMS sewer played a role in the collapse of the sewer. The assessment includes a review of available groundwater information for the area and relevant information obtained from other nearby projects, as well as a review on the formation of sinkholes due to soil erosion around pipe defects.

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2 Erosion Processes and Sinkhole Formation

The key process responsible for formation of sinkholes is the localised erosion and collapse of soils. Soil erosion can occur where there are defective pipes, with soil material caving into an opening into a corroded pipe. This is called 'soil erosion due to defective pipes' (SEDP) and several studies have been conducted on SEDP. Dastpak, Sousa and Dias (2023) provided a comprehensive review of available research on the subject.

Initial studies on SEDP originated from studies on internal erosion of dam embankments, which requires two criteria to be satisfied:

1. the eroded soil must have the ability to flow downstream and,
2. there must be sufficient flow (driving force) to initiate and progress the erosion.

These two factors are integral to SEDP, but additional factors affecting the mechanics of SEDP are the hydraulic condition (Section 2.1), the soil type (particularly soil gradation) (Section 2.2) and the pipe defect characteristics (Section 2.3). It is noteworthy that the driving force in SEDP is gravity which is not necessarily so for embankment dams.

Hydraulic loading mainly and cohesion of soils to a lesser extent affect the process of SEDP and are defined as follows:

Hydraulic loading

Hydraulic head is the pressure that water exerts above a datum (for e.g., above the buried sewer pipe). This hydraulic head can change, which is called hydraulic loading. In natural groundwater processes, hydraulic loading is caused when rainfall infiltration and raises the groundwater level, which will then cause increased pressure on the buried pipe. The groundwater subsequently flows downgradient as the rainwater dissipates laterally and the water level lowers, which reduces the pressure on the buried pipe. A defect in a pipe affects the hydraulic loading as follows:

- Water pressure in the pipe (full pipe) can result in water flowing from the pipe into the surrounding soils (called exfiltration).
- The water in the pipe recedes, leaving an airgap between the water in the pipe and the water in the formation above the pipe defect. The groundwater will now seep into the pipe (called infiltration) through the defect because of the hydraulic loading from the increased groundwater level above the pipe.
- Groundwater flow will reduce the groundwater level over time, which in turn reduces the hydraulic loading on the pipe.

Cohesive soils

Cohesive soils are comprised of fine-grained particles that stick together, like clay and silt. The effects of hydraulic loading and cohesive soils on SEDP formation are explained in more detail in Sections 2.1 and 2.2.

A simplified schematic of the formation of SEDP and subsequent formation of a sinkhole are shown in Figure 1 and summarised as follows:

1. The hydraulic loading around the sewer pipe is caused by rainfall events introducing a higher groundwater head. In some cases, water pressure in the pipe also increases (when the pipe is full) and may cause water leakage through the pipe defect into the formation, resulting in softening of the soils around the defect. When the water in the pipe recedes the increased hydraulic loading around the pipe will cause infiltration of the groundwater and some of the surrounding soils into the pipe through the defect. This erosion of soil around the defect in the pipe causes the development of a cavity.

2 When the groundwater level recedes when the groundwater drains away after rainfall, the hydraulic loading above the pipe reduces, and the cavity stabilises, partly due to the cohesiveness of the soil.
3 Subsequent increases in groundwater level due to rainfall infiltration increases the hydraulic loading again, resulting in an increase in the cavity size through the same process as 1.
4 As the cavity expands, the cohesion of the soil surrounding the cavity is compromised and the soil above the cavity collapses.
5 The repeat of this cycle eventually results in a sinkhole forming.

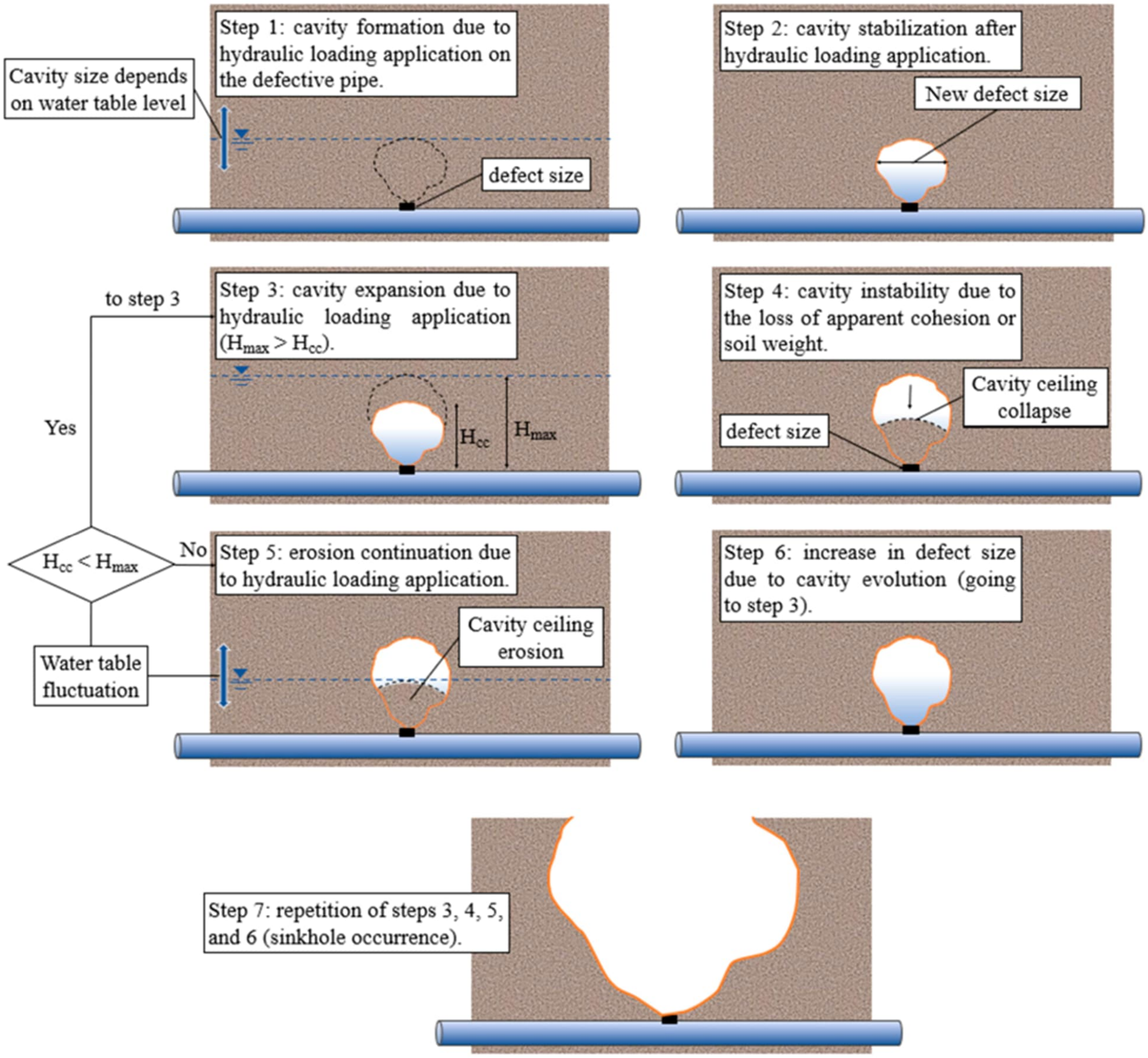

Figure 1. SEDP progression to sinkhole formation from Dastpak et al. (2023)

2.1 Hydraulic Loading

Hydraulic loading is considered the most influential factor in the rate and volume of soil erosion because the particle movement starts when water seepage occurs through a defect as a result of the water pressure difference between the outside and inside of the pipe. Hydraulic loading is only of concern near a defect in the pipe. Where there is no defect in the pipe, the hydraulic pressure is balanced by the pipe and no seepage into the pipe occurs. Higher head differences will result in increased volume of flow and associated soil erosion, because the erosion time is longer, even though the sand and water discharge rate changes little.

Cyclic infiltration/exfiltration tests were conducted by Mukunoki et al. (referenced by Dastpak, et al. 2023) to study the cavity expansion due to cyclic loading. They found that the void ceiling would collapse if the groundwater table rose above the ceiling height of the void. This finding highlights the likely adverse effects of climate change on SEDP, because flooding frequency will increase cyclic loading, which may lead to the unexpected occurrence of catastrophic sinkhole formation.

A similar experiment by Kwak et al. (referenced by Dastpak, et al. 2023) considered the impact of rainfall intensity and found that subsidence only occurred after extremely high rainfall intensity and that there may be a threshold rainfall intensity associated with subsidence. After the rainfall stopped, the soil erosion occurred resulting in the cavity development.

2.2 Soil Properties

The following information and findings from experiments on the effects of particle size distribution, and the ratio between the defect size and the maximum grain size on the SEDP are considered applicable to this study:

- An increased ratio of defect size in the pipe to maximum grain size increases the cavity size – i.e., with a bigger defect size and smaller grain size, the cavity is bigger. The bigger cavity size allows for sediment to erode unhindered into the pipe.
- The particle size distribution (PSD) of the soils used for backfilling around the pipe affects the cavity size. Soils with greater uniformity (i.e., less variation in grain size) have a higher susceptibility to SEDP than well-graded soils with a good distribution of grain sizes. This is because graded soils have larger particles that can form bridging behind the defect and reduce the erosion into the pipe.
- Experiments comparing the SEDP potential of uniform sand with three different particle sizes (Guo et al. from Dastpak et al) found that the finer sand had a lower discharge flow rate, but the eroded zone was almost equal for the different sands, i.e., eroded volume is similar. The cavity formation for a coarser sand will be quicker than for a finer sand.
- Kwak et al. (from Dastpak et al, 2023) investigated the effect of compaction of backfill material on the formation of SEDP. They found that the use of loosely compacted backfill material can lead to sinkhole formation much sooner during infiltration-exfiltration cycles.
- Indiketiya et al. (from Dastpak et al., 2023) also examined the effect of the PSD to defect width ratio. They found that the risk of SEDP for grain sizes smaller than 0.3 mm is significant, but significantly lower than when the grain sizes are larger than 1.18 mm.
- Kwak et al (from Dastpak et al., 2023) found the amount of fine particles in the soil also impacted SEDP, with cohesive soils like clay showing large settlements, but small eroded mass, whereas the eroded mass for sandy soil was much higher.

2.3 Pipe Defect Characteristics

The most critical parameter of the defect characteristic in SEDP is the defect size, in particular in relation to the soil particle size, as mentioned above. The defect size in particular was important during cyclic infiltration-exfiltration loadings, where a large slot size resulted in subsidence during the early cycles. Defect location on the top of a pipe resulted in higher SEDP rates.

The pipe depth is directly proportional to the eroded zone volume, because if the backfill height above the pipe is higher, the eroded zone will be bigger and when the eroded void collapses, the sinkhole diameter will be larger.

2.4 Other Factors

The location of other nearby existing pipes changes the position and orientation of the cavity due to the localised change in hydraulic gradient near the defect.

Fluidization occurs when the granular soils transfer into a fluid-like state after being pressurised by water, e.g., when a pressurised buried pipe leaks into the granular backfill.

3 Hydrogeological Site Description in Terms of Erosion and Sinkhole Formation

The site conditions and hydrogeological information for the site around the Ōrākei Sewer are described and defined in terms of the SEDP factors mentioned in Section 2.

3.1 The Construction and Condition of the Ōrākei Sewer

The OMS was either constructed within a hand dug tunnel or a trench depending on the depth of the pipeline. It is uncertain which method was used for construction of the section in St George Bay Rd that collapsed. The typical construction sequence would be:

1. Dig tunnel or trench.
2. Cast concrete base in place.
3. Install block arch.
4. Backfill tunnel or trench above arch.

It is likely that both the hand dug tunnel or trench construction may have imperfect backfill condition. In the case of tunnel construction, it is possible that a void remained between the top of the arch and the tunnel, or the void was filled with poorly compacted material. Alternatively, in the case of a trench, poor quality backfill may have been used and or this may not have been compacted properly. As indicated under soil properties above, the use of loosely compacted backfill material can lead to sinkhole formation much sooner during infiltration-exfiltration cycles.

The OMS sewer has shown signs of degradation along other sections of the sewer (2023 drone condition survey), mainly comprising the degradation of the mortar between the concrete blocks, but also showing isolated missing concrete blocks, which would have resulted from the loss of the mortar between the blocks. It is considered likely that similar degradation (both missing mortar and missing blocks) has occurred at the collapsed section of the OMS, however this has not been confirmed to date. Furthermore, groundwater seepage has been observed through the degraded mortar joints, which was likely to further degrade the mortar. A missing block or two is considered to be the defect in the pipe.

3.2 Regional Geology

The published geological map information (GNS, 1992) indicates the sinkhole area is underlain by the East Coast Bays Formation (ECBF), comprising alternating sandstone and mudstone with variable volcanic content and interbedded volcaniclastic grits. The residual (in-situ) weathered ECBF typically comprises extremely weak rock that is often described as sandy and silty soil. Puketoka Formation, Tauranga Group alluvium outcrops to the east of the site, comprising mostly pumiceous mud, sand and gravel. The volcanic sediments were sourced from the eruption of the Pukekawa Volcano at Auckland Domain. It is also highly likely that the historic gully that ran across the area has been infilled with fill. The fill material can vary from poorly sorted construction debris with boulders, gravel and fines or well sorted and well compacted silty and clayey material.

3.3 Site Specific Geology

Limited lithological and groundwater data were obtained from two hand auger logs drilled at 96 St Georges Bay Road (indicated on the map in Figure 2). The logs indicate fill comprising mostly clayey silt, silty clay and silt, underlain by residual soils East Coast Bays Formation (ECBF) comprising clayey silt, sandy silt and silt, grading into highly to moderately weathered ECBF. The information from the different lithological units is summarised in Table 1

Table 1. Available lithological information around OMS site – NZGD

Lithology unit	Thickness (m)	Description
Fill	1.2-2.5	Clayey silt, sandy silt, silt and some inclusions of scoria gravel
Residual soils ECBF	3.4-3.5	Clayey silt, sandy silt and silt
Highly to moderately weathered ECBF	-	Sandy silt, silt and clayey silt

The regional and site-specific geology indicate that the material overlying the Ōrākei Sewer is likely to be fine grained and cohesive. The ratio of the fine-grained nature of the overlying material with a cement brick size defect in the sewer pipe is likely to provide a large ratio of defect size in the pipe to maximum grain size, hence increasing the potential for SEDP.

3.4 Groundwater Levels

The groundwater levels measured in the two hand augers near the OMS sinkhole indicated deep water levels (5.3 and 4.7 m bgl for HA_117371 and HA_117372 respectively), measured in March 2015. This is considered representative of low water levels for the summer.

Groundwater level varied considerably as a result of the big storm events in late January and early February 2023. This fluctuation is very relevant to consider to better understand how groundwater contributed to the OMS collapse. Groundwater levels are monitored for several sites around the Auckland area. Typically, groundwater levels vary 0.5-1 m seasonally, however sharp increases of at least 0.5 m were observed over a very short period of time after the Auckland Anniversary Weekend storm (28 January 2023). This can be seen in the plot of groundwater level variations for a monitoring piezometer PZ07_D in the Auckland CBD on Queen Street, 1.6 km east of the Ōrākei Sewer sinkhole (Figure 3). This monitoring piezometer was screened at a depth of 6-9 m bgl in the Puketoka Formation, comprising silty clay and clayey silt at this location. The formation is considered similar to that of the Ōrākei Sewer site. A similar increase in groundwater level can be seen after the storm event associated with cyclone Gabrielle on the weekend of 3 February 2023. This trend of significant increases in groundwater level over a short time period as a result of the storm events were observed in other monitoring piezometers further afield in the Auckland area – Drury, Takanini, Wiri and Māngere areas.

ATTRIBUTION

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SCALE

0 20 40 m

PROJECT

Watercare Services Limited
Auckland
Orakei Mains Sewer Failure

FIGURE

Figure 2. Overview map of the OMS failure site and surrounds.

PROJECT NUMBER	REVISION DATE	REVISION
W-SL021.02	5 Dec 2023	R0

File Ref: C:\Users\NZLS30680\WSP O365\WSL EM - W-SL021.02\04_Disciplines\Hydrogeology\GIS\OMS Base.qgz

Original Sheet Size: A4 [W = 210, H = 297] Plot Date: 2024-01-10 07:54:55 by Soltau, Louise (NZLS30680)

The significant change in groundwater level is likely to have resulted in hydraulic loading that would have caused the start of a cavity above the Ōrākei Sewer:

- The increased water pressure in the sewer pipe after the first storm event (28 January) could possibly have caused some water flowing from the pipe into the surrounding soils (exfiltration) through the defect in the sewer pipe.
- After the water in the pipe receded, the increased hydraulic head above the pipe and saturated soils, would have allowed seepage (infiltration) of the groundwater loaded with fine sediment into the sewer.
- The groundwater level will likely have reduced somewhat before the next storm event in February, which may have allowed the cavity to stabilise.
- The next storm event would have increased the cyclic hydraulic loading again, resulting in an increase and subsequent stabilisation of the cavity above the sewer.
- There were other higher rainfall events after the two big storm events mentioned above that would likely have had a similar cyclic loading effect, though on a smaller scale,
- It is considered likely that increased rainfall on 24 and 26 September may have constituted the final cyclic loading that resulted in the collapse of the surface to create the sinkhole.

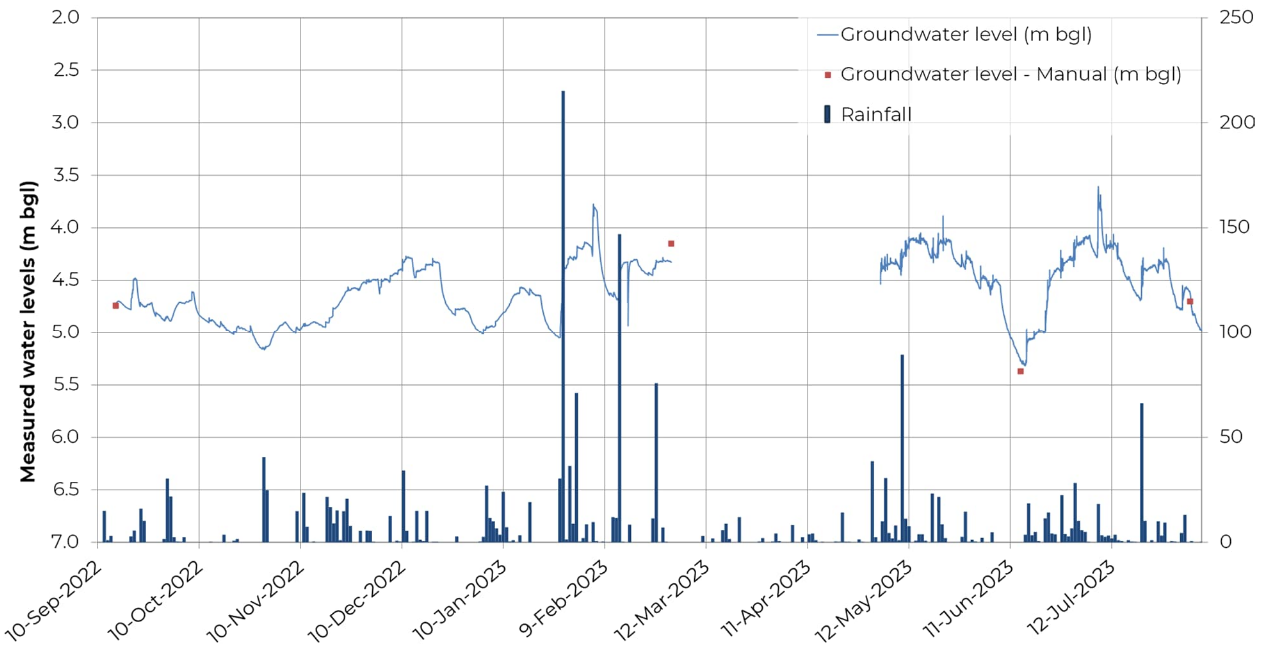

Figure 3. Groundwater level variation – Auckland CBD.

3.5 Hydraulic Conductivity Data

PDP (2016) summarised available hydraulic parameters from several studies for the City Rail Link, Aotea station site investigations. These hydraulic parameters are considered applicable for the OMS sinkhole site, because of the proximity and general similarity in geology of the two sites. The hydraulic conductivity ranges for the different lithological units are summarised in Table 2.

Table 2: Summary of Hydraulic Conductivity ranges for the relevant hydrogeologic units (from PDP, 2016)

Hydrogeology unit	Hydraulic conductivity		
	Minimum in m/day [m/s]	Maximum in m/day [m/s]	Kh:Kv
Fill	0.015 [1.7×10^{-7}]	8,640 [1.0×10^{-1}]	0.01-1
Tauranga Group alluvium and residual soil East Coast Bays Formation (ECBF)	0.0005 [6.3×10^{-9}]	0.6 [7.2×10^{-6}]	0.001-0.1
Weathered and unweathered ECBF	0.00039 [4.5×10^{-9}]	0.5 [6.0×10^{-6}]	0.001-0.1

The hydraulic conductivities are typically low, and the groundwater level would increase substantially during rainfall events because the water drains away slowly, resulting in significantly increased hydraulic loading. It is also important to note that the hydraulic conductivity of the fill material varies significantly due to differing sources and hence variable composition of the fill. Should there be zones within the fill material that comprises coarser sediments, these would form preferential groundwater flow paths. If these coincide with a defect in the sewer, the formation of the sinkhole would be accelerated.

4 Discussion and Conclusions

It is considered likely that soil erosion due to increased groundwater levels near a pipe defect in the OMS sewer resulted in the formation of the OMS sewer sinkhole. This can be explained as follows:

- The OMS sewer has shown signs of degradation along other sections of the sewer (2023 CCTV condition survey), mainly comprising the degradation of the mortar between the concrete blocks, but also showing isolated missing concrete blocks, which would have resulted from the loss of the mortar between the blocks. It is considered likely that similar degradation (both missing mortar and missing blocks) has occurred at the collapsed section of the OMS, however this has not been confirmed to date. Furthermore, groundwater seepage has been observed through the degraded mortar joints, which was likely to further degrade the mortar. A missing block or two is considered to be the defect in the pipe.

- The soils surrounding the sewer likely comprises fill material, which is considered to be mostly fine grained. The fine-grained soils would be much smaller than the defect size of a concrete block and would have easily eroded into the sewer pipe.

- The storm events on the weekends of 27 January and 3 February will have resulted in varying groundwater pressures as follows:

  The sewer pipe would have been full and may have resulted in water seeping out of the sewer into the surrounding soils (exfiltration), which would have been saturated already from the high rainfall, through the existing defect in the sewer pipe. This would have resulted in an initial physical scouring of the formation. The drop in water level within the pipe after the flood waters receded would have resulted in an increase in head difference between the ambient groundwater level (which will have risen notably in response to the heavy rainfall) and the water level in the pipe. This likely led to increased seepage of groundwater (infiltration) laden with fine sediment into the pipe through the defect. This would have constituted the start of the cavity. The groundwater level may have lowered after the first storm to below the top of this cavity, which would have allowed the stabilisation of the cavity. However, the process would have repeated during the second storm, increasing the size of the cavity.

  There were other higher rainfall events after the two big storm events mentioned above that may have had a similar cyclic loading effect, though on a smaller scale. It is considered likely that increased rainfall on 24 and 26 September may have constituted the final cyclic loading that resulted in the collapse of the surface to create the sinkhole.

It should also be noted that arches are compressive, self-supporting structures that are stabilised by the force of gravity pushing the blocks together. This makes them very stable, and they can support relatively large loads. However, arches can fail if overlying loads are not evenly balanced. The formation and subsequent enlargement of the cavity above the arch, may have resulted in an uneven load that eventually led to the catastrophic collapse of the arch.

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APPENDIX J: CONDITION ASSESSMENT TECHNIQUES

Table J:2 summaries some common techniques categories by what part of the pipe it assesses. This information was derived from the ‘Guidelines for Condition Assessment and Rehabilitation of Large Sewers’ (Zhao, McDonald, & Kleiner, 2001) and ‘New Zealand Gravity Pipe Inspection Manual 4^{th} edition’ (Water NZ, 2021).Table J:2 summaries some common techniques categories by what part of the pipe it assesses. This information was derived from the ‘Guidelines for Condition Assessment and Rehabilitation of Large Sewers’ (Zhao, McDonald, & Kleiner, 2001) and ‘New Zealand Gravity Pipe Inspection Manual 4^{th} edition’ (Water NZ, 2021).

Table J:2 Outputs and limitations of different condition assessment techniques

Type	Technique	Output	Limitations
Internal Condition	Person-entry	Qualitative and quantitative (direct and manual measurements) observation of pipe interior	Health and safety risks, pipes greater than 900mm diameter, above flow line only
	CCTV	Video footage of interior of pipe above flow line. Can be a tractor unit or a floating unit	Dependent on camera/ footage quality, low quality, difficult to quantify defect size
	Laser	3D profile and accurate dimensions of the internal pipe wall surface. Usually combined with sonar	Above flow line only
	Sonar	Pipe surface profile in submerged sections. Usually combined with laser profiling	Below water line only, low accuracy
	LiDAR (3D laser scan)	Similar to Laser scanning but usually from a fixed location	Sensitive to splashes of water, good for manholes but not for pipes
	ROV (remotely operated vehicle) e.g. drone	Video footage. Often combined with LiDAR	Varies largely depending on type of ROV.
Pipe Wall	Impact echo	Physical test to indicate the presences of void or defects in the pipe wall	Requires person-entry
	GPR	Locate voids and presence of any reinforcement	Requires person-entry [1]

Type	Technique	Output	Limitations
	Coring	Taking a sample of the pipe wall to assess the condition	Invasive. Requires either entry of pipe or excavation to expose outside of pipe.
	Insitu strength testing	Using a hand-held instrument (e.g. Schmidt hammer)to determine strength	Influenced by aggregate size and skill of user. Only works on surface layer.
Ground surrounding the pipe	Infrared thermography	Drone or fixed camera uses heat signatures to detect thermal anomalies in the ground (e.g. due to water movement or to voids).	Limited sensitivity subject to time of day of assessment. Low accuracy for estimating size or depth of thermal anomaly Needs skilled interpretation to determine cause of the anomaly
	Surface inspection	Dips and settlement of surface above pipe indicate the presence of voids or pipe deformation	Low accuracy. Cannot detect deeper sub-surface features.
	Ground penetrating radar (GPR)	Detects voids in the soil surrounding the pipe	Effectiveness dependant on soil type and depth.
	Other geophysical techniques	A family of techniques used to detect variations in properties below the surface.	Not widely available, limited sensitivity to this application.