

TE KOWHAI EAST CIVIL INFRASTRUCTURE REPORT

PROJECT INFORMATION

CLIENT: TE KOWHAI EAST LTD PARTNERSHIP

PROJECT: 180010

DOCUMENT CONTROL

DATE OF ISSUE: 28 August 2026

REVISION: A

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EXECUTIVE SUMMARY

This Civil Infrastructure Report assesses the proposed civil infrastructure required to service the Te Kōwhai East Development and support Te Kōwhai East Limited Partnership's substantive application under the Fast-track Approvals Act 2024 (FTAA).

The Te Kōwhai East Development is described in detail in the Substantive Application Report (SAR). In summary, the development comprises a large-scale industrial development on the north-western edge of Hamilton, adjoining the established Te Rapa industrial area and the wider Horotiu / Te Rapa / Rotokauri growth corridor.

The development is proposed to be delivered in stages, which are integrated with the civil infrastructure strategy. This report addresses earthworks, roading, stormwater, wastewater, water supply and network utilities. The key infrastructure conclusions and findings are summarised as follows:

Earthworks

Bulk earthworks are proposed to establish the primary development landform, including industrial development platforms, public road corridors, stormwater management areas, service corridors and the modified Mangaheka Stream corridor. The earthworks strategy comprises large-scale cut-to-fill works, primarily involving cutting of the northern and north-eastern rolling hill landform and placement of engineered fill within selected low-lying areas to achieve appropriate development grades, drainage falls, flood management outcomes and serviceability for staged industrial development. Earthworks within the Mangaheka Stream corridor will support flood conveyance, stormwater management and downstream transition works through widening and deepening of the channel.

Roading / Transportation

The transport strategy is based on a connected public road network comprising a primary internal industrial collector loop road, local industrial roads, intersections, active mode facilities, service corridors and maintenance accessways. The primary site access is proposed from Te Kowhai Road / SH39 via a new roundabout, with secondary access from Onion Road via a T-intersection. The road network has been developed to support industrial vehicle movements, pedestrian and cycling connectivity, emergency access, utility servicing and future public transport opportunities. Access arrangements, intersection forms, external upgrades, walking and cycling provision, public transport integration and transport effects are addressed in the CKL Integrated Transportation Assessment. Streetscape elements, including lighting, landscaping, street trees, parking bays, vehicle crossings and provision for future bus stops, will be refined through detailed design.

Stormwater and Flooding

Stormwater management is a key component of the development strategy. The site is primarily located within the upper Mangaheka Stream catchment, with a smaller eastern portion of the development interfacing with the Te Rapa catchment. The Te Otamanui catchment also passes through the southern portion of the site; however, the development does not rely on altering the operation of this catchment, and the existing Te Otamanui catchment function is intended to be maintained. The stormwater strategy has therefore been developed with reference to the Mangaheka Integrated Catchment Management Plan, the Te Rapa North Integrated

Catchment Management Plan, the Waikato Regional Council Land Drainage Management Plan, RITS, and the Waikato Regional Council stormwater guidelines. The proposed stormwater system will include reticulation, raingardens, wetlands, conveyance channels, overland flow paths, flood management areas and Mangaheka Stream corridor works.

Flood modelling has been undertaken and demonstrates that peak post-development flows, velocities and peak flooding elevations from TKE have been appropriately mitigated back to the pre-development level. Flooding internally within the TKE has been mapped via hydraulic modelling and shows flood waters are safely conveyed away from the future building platforms via the public roading corridors and stormwater network. The hazard assessment of the master-planned flow paths demonstrates low risk to TKE up to the 100-Year ARI storm event, and where deviations to normal standards apply, have been assessed to be minor and resolvable at detailed design. Refer to the separate Stormwater Management Plan provided in the substantive application appendices for the full stormwater management strategy and assessment of on- and off-site flooding effects.

While the proposed attenuation measures have managed to mitigate post-development peak flows (i.e. the rate at which stormwater leaves the site), it cannot reduce the volume, and the receiving watercourses carry elevated flows for a longer duration and longer drain down times after rainfall events. This additional duration has the potential to increase the risk of stream erosion over time. TKE proposes an 'adaptive residual management' approach which aims to offset the retention of the 2-Year ARI volume to the industrial Lots. As individual Building Consents are approved, the volume of soakage and reuse for each industrial Lot is tracked. Where a shortfall remains, further on-Lot and site-wide measures are applied first in accordance with the mitigation hierarchy in the Stormwater Management Plan. Any residual effect remaining at catchment closure is then assessed, and that remaining effect is addressed through mitigation within the receiving environment, which may include a financial contribution toward downstream erosion prevention works. Further details of the 'adaptive' approach is outlined in the Stormwater Management Plan and ensures that any contribution reflects the residual effect remaining after the soakage and re-use achieved on the industrial Lots.

Wastewater

Wastewater servicing is proposed through a new internal gravity reticulation network draining to a proposed wastewater pump station (located in Stage 1), with the network progressively extended to support staged development. The pump station will discharge via a rising main to an approved downstream connection, with interim servicing proposed through the Far Western Interceptor / existing downstream wastewater network subject to confirmed capacity, flow limits, staging triggers and connection requirements agreed with IAWAI / HCC. The ultimate long-term servicing solution is connection to the planned Northern Interceptor, with the transition from the interim arrangement to be confirmed through detailed design, infrastructure provider approval and development agreement processes. Wastewater demand has been assessed on the basis of light industrial use and domestic-strength discharge.

Water Supply

Water supply servicing is proposed through a new internal public reticulation network connected to an approved external bulk water supply source, with potable supply, operational

flexibility and FW3 firefighting capability to be provided for each serviced stage. The strategy comprises staged interim public network connections and upgrades to enable development in advance of the ultimate strategic solution comprising a reservoir and bulk main connection servicing the development. The confirmed interim servicing capacity, residual pressure, trigger points and preferred network arrangement are subject to confirmation through Hamilton City Council hydraulic modelling, IAWAI engagement, infrastructure provider approval and development agreement processes. The ultimate solution will transition the development to the strategic reservoir and associated bulk infrastructure, with final pipe sizing, pressure management, storage, firefighting, metering, backflow prevention, asset ownership and handover requirements confirmed through detailed design and engineering approval.

Utilities

Power, telecommunications and other utility services will be provided in coordination with the relevant network utility providers. Utility correspondence confirms that fibre servicing can be made available subject to a Tuatahi First Fibre installation agreement, and that electricity reticulation can be provided subject to capacity, staging and network upgrade requirements.

This report confirms that the proposed development is able to be serviced in a manner that is practical, efficient and consistent with relevant local guidelines and industry standards. Detailed design and engineering approval will be required following consent approval and prior to construction commencing on the site.

The actual and potential effects associated with the proposed civil infrastructure have been considered in this report and the supporting specialist assessments. These effects are appropriately avoided, remedied or mitigated through identified design measures, management plans, staging controls, monitoring and recommended consent conditions.

QUALIFICATIONS, EXPERIENCE AND CODE OF CONDUCT

The qualifications and experience of the authors and reviewer of this report are summarised below. We confirm that we have read the Code of Conduct for Expert Witnesses contained in the Environment Court of New Zealand Practice Note 2023 and we agree to comply with it. We confirm that the issues addressed in this report are within our area of expertise, except where we state that we are relying on the evidence or advice of another person. We have not omitted to consider material facts known to us that might alter or detract from the opinions expressed in this report.

Report Co-Author and Approver - Ryan Wyllie:

Ryan Wyllie is a Principal at Maven Associates Limited. He holds a Bachelor of Engineering with Honours from the University of Auckland and is a Chartered Professional Engineer practising in the field of civil engineering.

Ryan has over 14 years of professional experience working as a civil engineer and project manager in land development and infrastructure projects across New Zealand, Australia, the United Kingdom and Canada. His experience includes three waters infrastructure, stormwater management, roading and highways, bulk earthworks, urban infrastructure planning, development staging, and coordination of multidisciplinary land development projects.

In New Zealand, Ryan has led major land development projects including Richmond Residences in Mt Wellington, Kauri Green at Westgate, Lockerbie Estate in Morrinsville, East Coast Heights in Silverdale, and Rangitootuni Retirement village in Riverhead. These projects have involved complex civil infrastructure planning, geotechnical constraints, stormwater and flooding constraints, roading design, wastewater and water supply servicing, utility coordination, and engagement with councils and infrastructure providers.

Internationally, Ryan served as Project Manager for Transport for London's Elephant and Castle Northern Roundabout redevelopment, where he managed the construction phase of a major intersection upgrade with significant public and stakeholder engagement responsibilities. More recently, with WSP in Canada, he managed major highway rehabilitation and safety upgrade projects on the Alaska Highway, and undertook preliminary design for a highway interchange and access road associated with the Garibaldi and Squamish mountain resort project.

Ryan's qualifications include a Bachelor of Engineering with Honours from the University of Auckland. He is also a Chartered Professional Engineer.

Signed:  _____ Date: 28 August 2026

Report Co-Author - Kevin Yip:

Kevin Yip is a Team Leader at Maven Associates Limited. Kevin has been employed with Maven for more than three years. He holds a Bachelor of Engineering with Honours from the University of Auckland.

Kevin has over 10 years of professional experience working in land development shared between 3 years working as a land development project manager and 7 years working as a civil engineer. As a project manager working for a private builder developer, Kevin has informed land acquisitions, managed development feasibilities/budgets, consenting delivery and construction contracts throughout various regenerated areas throughout Auckland. As civil engineer, Kevin has experience in stormwater design and management, three waters and roading infrastructure.

Kevin has led Maven's involvement in Te Kukūnga Waka (~4,000 medium-to-high density homes) in Auckland, delivering separable portions of the backbone infrastructure and open space projects led by the three Tāmaki Makaurau Rōpū of Marutūāhu, Ngāti Whātua and Waiohua-Tāmaki as well as their project partners.

Signed:  Date: 28 August 2026

Report Reviewer - David Cloete:

David Cloete is a Chartered Professional Engineer registered with Engineering New Zealand. David has been employed with Maven for more than four years and is the Regional Director of the South office. David graduated in 2012 while working in the industry and has more than 15 years of industry experience. David has worked on land development projects across many regions and has experience in complex stormwater management, site grading, environmental controls and roading matters.

David's qualifications include the New Zealand Diploma in Engineering (Civil) and the Bachelor of Engineering Technology (Civil). David has also completed certificates of proficiency in Highway Geometric Design, Highway and Transport Design, and Urban Stormwater Management. David has been a Chartered Professional Engineer for more than five years.

Signed:  Date: 28 August 2026

1. INTRODUCTION

This Civil Infrastructure Report has been prepared by Maven Associates Limited to support Te Kōwhai East Limited Partnership's substantive application for approvals under the Fast-track Approvals Act 2024 (FTAA).

This technical report assesses the civil infrastructure required to service the Te Kōwhai East Development (the Project), including earthworks, roading, stormwater, overland flow and flooding, wastewater, water supply, power, telecommunications and other utility services.

The Project is a large-scale industrial development located on the north-western edge of Hamilton, adjacent to the established Te Rapa industrial area. The application provides for the creation of an integrated industrial precinct, including approximately 210 industrial lots, land for neighbourhood centre activities, reserves for recreation, stormwater management and ecological mitigation, and associated roading and infrastructure services. The application also seeks approval for the ultimate land use outcomes anticipated across the site, being predominantly industrial activities, along with a limited range of compatible non-industrial activities suited to an industrial environment.

A comprehensive description of the Project is provided in the Substantive Application Report (SAR).

The infrastructure design and supporting material have been prepared to a level appropriate to support the FTAA substantive application and to confirm the proposed development has a viable servicing strategy. Further design development, modelling, infrastructure provider engagement, and engineering approvals will be required prior to construction and will be undertaken through the Engineering Plan Approval process.

1.1 SITE DESCRIPTION / IDENTIFICATION

The Project site comprises a large, contiguous landholding located on the north-western edge of Hamilton, located within Waikato District. It is generally bounded by Te Kowhai Road to the south, Koura Drive and State Highway 1C to the east, and Onion Road to the north. The site is made up of multiple land titles, legal descriptions, and associated physical addresses.

For identification purposes, the site is commonly referred to by the primary addresses of 270 Te Kowhai Road and 28 Mathers Road. Full details of the landholdings are provided in the Substantive Application Report (SAR), and the overall site layout is shown on the Scheme Plan (Appendix A). The site is identified in Figure 1 below.

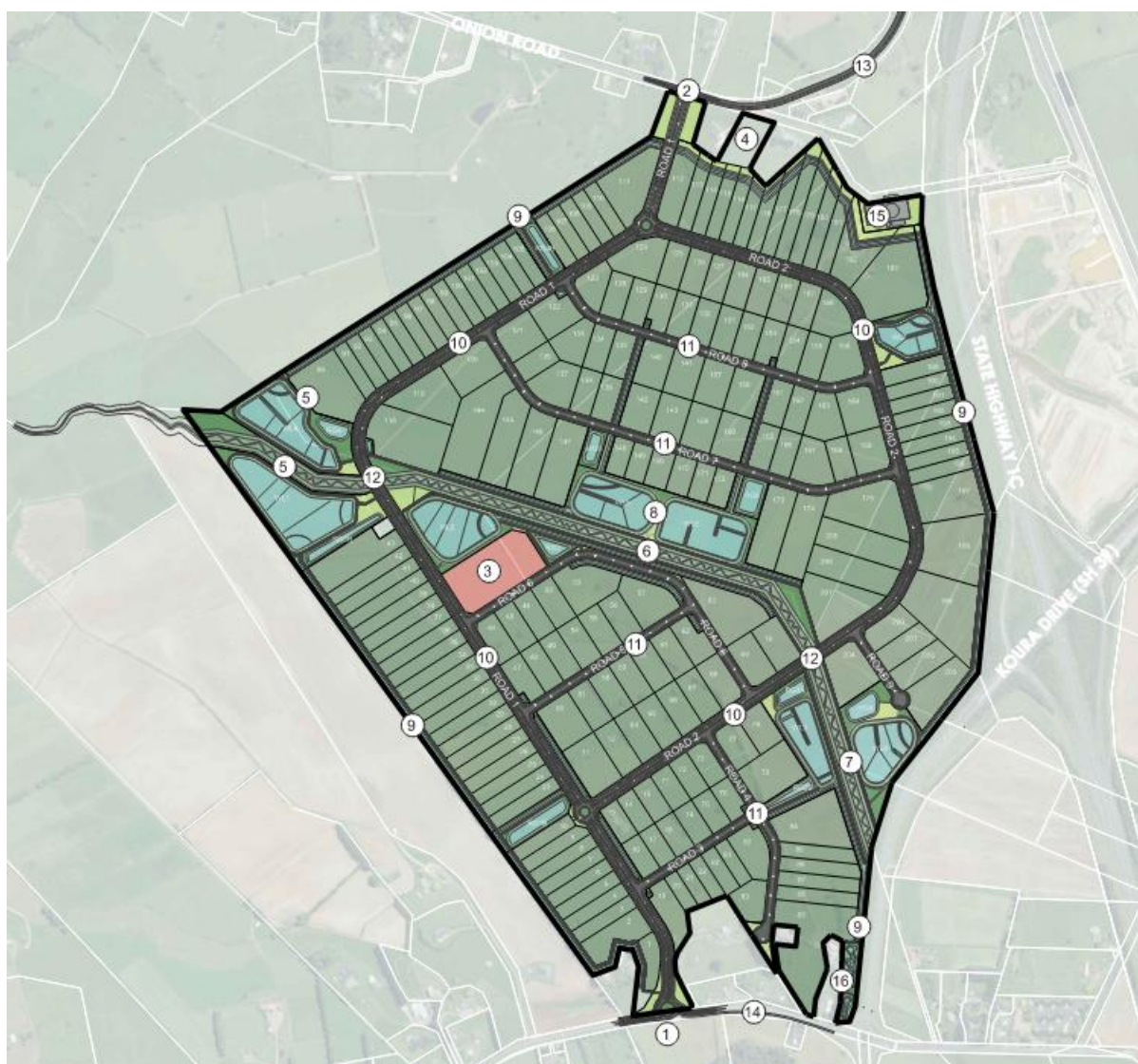


Figure 1: Adapt Site Plan

1.2 SUMMARY OF PROPOSED DEVELOPMENT AND STAGING

The Project site comprises approximately 187.5 ha of industrial land, together with a supporting Neighbourhood Centre, open space areas, stormwater management areas, public roads, and associated three waters and network utility infrastructure.

The Project is being progressed under the Fast-track Approvals Act framework, seeking a comprehensive suite of subdivision, land use consents and regional consents. This report provides the civil engineering assessment supporting that framework.

The development is proposed to be delivered progressively across nine stages, generally in sequential order. Please refer to the Scheme Plans included in Appendix A for the details of lot layout and development staging and as depicted in overview in Figure 2: below.

Detailed design and final servicing solutions, including staging-specific engineering design and any external infrastructure upgrades, will be confirmed through Engineering Plan Approval and detailed

sites, water and wastewater trunk infrastructure and the Mangaheka Stream corridor are treated as fixed or controlled infrastructure elements. Lot service laterals, individual vehicle crossings, streetscape layout elements (including street trees, street lighting and parking bays) and secondary lot-specific overland flow paths may be refined through detailed design and subdivision (223/224c approvals), provided the approved infrastructure outcomes are demonstrated as being maintained.

Any amended lot layout will need to demonstrate that all resulting lots retain compliant legal and physical access, can connect to the reticulated water, wastewater and stormwater networks without requiring upgrades beyond those authorised, and maintain equivalent or better 100-year ARI overland flow conveyance. These requirements are reflected in the draft consent conditions.

1.4 EXTERNAL INFRASTRUCTURE UPGRADES AND SERVICING TRIGGERS

Serviceability of each stage depends on the availability of sufficient network capacity and completion of any internal and external network upgrades required to service that stage. The draft consent conditions link section 224(c) certification to specified infrastructure triggers, such that a stage cannot proceed where it would exceed an identified threshold or confirmed interim capacity unless the corresponding infrastructure requirement has been satisfied.

Transport

- The SH39 / Te Kowhai Road roundabout and associated Te Kowhai Road urbanisation and active-mode works must be completed before section 224(c) certification for the first subdivision stage
- The Onion Road / TKE intersection and associated corridor works must be constructed and operational before section 224(c) certification for the first stage or allotment relying on Onion Road access.
- Further off-site transport mitigation is required where the cumulative thresholds in the CKL Integrated Transportation Assessment are exceeded.
- Detail is set out in Section 3.

Stormwater

- Each stage must have the stormwater infrastructure required to service that stage constructed and operational, or appropriate interim measures certified, before section 224(c) certification.
- Where a stage is within or drains to the Mangaheka Stream realignment corridor or the Te Rapa outlet, the relevant stream, wetland and strategic stormwater works required to service that stage must also be completed and operational.
- Detail is set out in Sections 4 and 5.

Wastewater

- Each stage must have an available wastewater servicing solution with sufficient network and treatment capacity. Any required upgrades, staging requirements and timing or development triggers are to be confirmed with the relevant network operator.
- Interim solution relies on an interim connection to the Far Western Interceptor.
- The strategic solution is connection to the Northern Interceptor once Hamilton City Council / IAWAI has constructed and commissioned this interceptor.
- Detail is set out in Section 6.

Water supply

- Interim solution relies on connection to the existing water infrastructure at Ruffell Road / Arthur Porter Drive, with the interim servicing capacity being agreed with Hamilton City Council water modellers.
- Beyond the interim capacity, the strategic reservoir and feed line to the existing bulk main on Rotokauri Road via Arthur Porter Drive must be constructed, commissioned and operational before 1 November 2033, consistent with the development agreement.
- Detail is set out in Section 7.

Power and telecommunications

- Interim electrical capacity of approximately 9 MVA is available, with the balance of the development load from the planned New Exelby zone substation or earlier agreed network upgrades.
- Telecommunications servicing is subject to the Tuatahi First Fibre installation agreement.
- These sit outside the consent trigger schedule and are managed through the utility connection agreements.
- Details are set out in Section 9.

2. EARTHWORKS

2.1 EARTHWORKS INVESTIGATION AND DESIGN BASIS

A Geotechnical Investigation Report (GIR) has been prepared by CMW Geosciences for the development. The GIR confirms that the site comprises a large area of low-lying plains at approximately RL27 m to RL29 m, underlain by Hinuera Formation soils of interbedded sands and silts, with rolling hill country along the northern boundary rising to approximately RL40 m and underlain by Walton Subgroup soils of weathered volcanic ash, clay, silt and sand.

The GIR identifies the following matters relevant to the earthworks and infrastructure design:

- Shallow groundwater conditions across the low-lying plains;
- Different ground conditions across the low-lying areas and northern rolling hills, including localised compressible soils;
- Sensitive silt soils that may require careful handling, moisture conditioning and blending during earthworks;
- Predicted settlement beneath new fill areas and future building loads;
- Liquefaction-related settlement and deformation under major seismic loading; and
- Variable soakage performance across the site.

The earthworks strategy and infrastructure design respond to these conditions and have been developed in accordance with the GIR and subsequent geotechnical advice. Key measures include settlement monitoring and possible preload in selected areas, reuse of suitable site-won material as engineered fill, dewatering for excavations and service trenches, road subgrade improvement, and the design of cut batters, fill embankments and retaining structures. The GIR provides preliminary foundation and liquefaction design recommendations, with further site-specific geotechnical investigation and design to be undertaken for future buildings at building consent stage.

The GIR concludes that the site is suitable for the proposed development, provided its recommendations are followed, and that the further investigation and design required at detailed design and construction stage do not represent unresolved geotechnical constraints to the development. Earthworks will be undertaken in accordance with the earthworks specification, NZS 4431:2022, council requirements, consent conditions and the final Earthworks Management Plan. On completion, the geotechnical engineer will prepare a Geotechnical Completion Report confirming the works and identifying any ongoing geotechnical requirements for future development. Refer to the GIR for further detail.

A Hydrogeological Assessment has also been prepared by WGA to assess the potential temporary and long-term groundwater effects of the development. The civil infrastructure and earthworks design responds to the groundwater constraints it identifies, including shallow groundwater across the plains, proposed wetland and swale levels, the lowered and widened Mangaheka Stream corridor, the wastewater pump station excavation and pipeline trenching, temporary dewatering requirements and potential drawdown effects.

These constraints influence the earthworks strategy, the stormwater management system, the wastewater pump station and pipeline construction, settlement risk, construction dewatering, stream corridor works and the long-term operation of the wetlands and swales. Both the WGA and CMW assessments confirm that the associated effects can be appropriately managed.

2.2 PROPOSED EARTHWORKS AND MANAGEMENT

2.2.1 Proposed Earthworks Overview

The proposed earthworks have been developed to support the masterplan by regrading the site to establish suitable industrial development platforms, public road corridors, stormwater management areas, stream corridor works, drainage conveyance paths and other supporting infrastructure. The earthworks strategy has been integrated with the proposed development staging and has been designed to achieve the required development levels while responding to the site's existing topography, geotechnical constraints, stormwater management requirements and flood management objectives.

The development will require a large-scale cut-to-fill operation across the site. Earthworks will primarily involve cutting the elevated rolling hill terrain in the northern and north-eastern portions of the site and placing engineered fill within the lower-lying and flatter areas to achieve the finished subgrade levels required for development platforms, road corridors and stormwater management infrastructure. Cuts and fills up to approximately 5.0 m in depth will also be required across the plains of the site to establish the final development grades and provide the required falls for drainage. A retaining wall up to 6.0 m high is located along the northern boundary of the site, which is discussed further in Section 8 and within the CMW report.

The Mangaheka Stream corridor forms an integral part of the wider stormwater and flood management for the development. Earthworks within the corridor are proposed to facilitate flood conveyance and integrate with the stormwater management system. The proposed stream corridor works extend approximately 400 m downstream beyond the site boundary, where the modified channel transitions back into the existing stream alignment.

The proposed earthworks plans are included within Appendix B of this report; refer to C200 (Earthworks) series.

2.2.2 Proposed Earthworks Volumes

The following volumes summarise the scale of bulk earthworks required to form the development platforms, road corridors, stormwater management areas and stream corridor works described above. The volumes are based on a comparison of the existing subgrade surface with the proposed subgrade surface and exclude any allowance for service trenches. Volumes are unfactored and in-situ.

- Bulk subgrade earthworks, existing subgrade to proposed subgrade:
- Total earthworks area: approximately 1,859,563 m² / 185.9 ha;
- Total cut volume: approximately 1,150,860 m³;
- Total fill volume: approximately 1,000,780 m³;
- Net cut volume: approximately 150,080 m³; and
- Topsoil stripping, based on 300 mm depth: 557,869 m³;
- Topsoil respread 219,026 m³.

The above estimate includes a surplus cut allowance to account for a compaction factor of approximately 15% to achieve a general balance without the need to import fill. Residual surplus (if any) will be managed through minor modifications to final levels. Excess topsoil that is surplus to the respread requirements is to be removed from site through an approved means. The subgrade assumptions adopted for the estimate are:

- Existing subgrade finished ground level less 300 mm topsoil;
- Lot subgrade proposed ground level less 100 mm topsoil;
- Stormwater reserve proposed ground level less 200 mm topsoil; and
- Road subgrade proposed ground level less pavement depth (1m pavement, 300mm berm).

Final earthworks volumes will be refined through detailed design but are expected to remain generally consistent with the earthworks strategy and scale of works described in this report.

2.2.3 Earthworks Management and Erosion and Sediment Control

Construction of the proposed earthworks and stream works will be undertaken in accordance with an approved Erosion and Sediment Control Plan (ESCP), Earthworks Management Plan (EWMP) and Stream Works Management Plan (STMP). These plans provide the environmental and engineering controls for construction, which is expected to be delivered over approximately four main earthworks phases.

Erosion and Sediment Control Plan (ESCP)

Erosion and sediment control measures will apply to all earthworks and stream works. A draft ESCP (Appendix B – 200 Series Earthworks Drawings) has been prepared for the current methodology and staging in accordance with Waikato Regional Council requirements and relevant guidelines. It will be finalised before construction to reflect the confirmed methodology, consent conditions and final EWMP, then implemented progressively as works advance across the site.

The erosion and sediment control measures are expected to include, where appropriate:

- Clean water diversions;
- Dirty water diversion channels;
- Sediment retention ponds;
- Decanting earth bunds;

- Stabilised construction entrances;
- Construction staging and catchment separation;
- Progressive stabilisation of completed earthworks areas;
- Dust control;
- Erosion protection to channels, swales and exposed batters;
- Construction monitoring and maintenance; and
- Protection of downstream receiving environments.

Earthworks Management Plan (EWMP)

A draft EWMP is included in Appendix D and will be finalised before construction. It provides the framework for bulk earthworks across the site and addresses:

- Construction staging and sequencing;
- Earthworks methodology and material management;
- Cut and fill operations;
- Topsoil stripping, stockpiling and reinstatement;
- Engineered fill placement and quality assurance;
- Temporary drainage and groundwater management; and
- Integration with stormwater infrastructure, stream works and other civil infrastructure.

The EWMP will be implemented alongside the CMW Geotechnical Investigation Report, project earthworks specifications, engineering drawings and relevant consent conditions.

The earthworks are arranged in four main phases, supported by three stream works phases (S1–S3). The sequence is intended to allow cut material to be hauled directly from cut areas to fill areas, avoiding intermediate stockpiling where practicable. The southern phase and stream works phases are net cut-generating, while the northern phases are net fill-consuming. Significant stockpiling is expected to be limited to topsoil, in locations approved by the project geotechnical engineer.

The programme is sequenced to align with the intended stage releases and is expected to span approximately four to five earthworks seasons. Cut and fill volumes will be recalibrated before each stage and reviewed monthly so the direct cut-to-fill approach can be maintained as ground conditions and material yields are confirmed. Within each stage, drainage and stream works will generally precede bulk earthworks to provide platform drainage. Stream reaches will be constructed from upstream to downstream, with each reach reinstated and stabilised before the next begins.

Further detail on phasing, programme durations, staged volumes and construction methodology is provided in the EWMP (Appendix D) and STMP (Appendix F).

Stream Works Management Plan (STMP)

A draft STMP is included in Appendix F to guide works within and adjacent to the Mangaheka Stream corridor. The STMP sets out the methodology and environmental controls for stream modification works, including:

- Construction sequencing within the stream corridor;
- Temporary flow diversion and dewatering where required;
- Sediment and erosion control measures specific to stream works;
- Fish passage and ecological management measures;

- Channel reconstruction and erosion protection;
- Riparian reinstatement and planting;
- Construction monitoring and environmental compliance; and
- Integration with the wider stormwater management and earthworks programme.

The STMP should be read with the Stormwater Management Plan (Appendix E), EWMP, ecological assessments and engineering drawings.

2.3 FUTURE LOT DEVELOPMENT EARTHWORKS

The bulk earthworks described in this report are intended to establish the primary development landform, public road corridors, stormwater management areas, stream corridor works, service corridors and general industrial development platforms.

Future lot-specific earthworks may be required as individual sites are developed. These works may include building platform trimming, yard formation, service trenching, retaining walls, pavement construction, localised ground improvement, foundation preparation, and final tie-ins to road frontages, stormwater devices and adjacent lots.

Future lot-level earthworks will be subject to detailed design and will need to respond to the recommendations of the Geotechnical Completion Report, building consent requirements, any engineering approval requirements, and any relevant consent conditions.

Where future industrial users propose activities with unusual loading, high pavement demand, heavy vehicle movements, large building footprints, high storage loads, vibration-sensitive uses, or specific foundation requirements, further site-specific geotechnical assessment may be required.

Future earthworks will also need to maintain the integrity of the approved stormwater management system, overland flow paths, flood management areas, road corridors, public services, retaining structures, boundary interfaces and any completed stream corridor works.

3. ROADING

3.1 ROADING OVERVIEW AND DESIGN BASIS

3.1.1 Existing Roading Network

The site is located directly adjacent to Te Kowhai Road / State Highway 39, Onion Road, Mathers Road, Koura Drive and State Highway 1C. These roads currently provide access to the surrounding rural, rural-residential and industrial areas and connect the site to the wider Hamilton and Waikato transport network.

3.1.2 Roading Design Basis and Overview

The proposed road layout included within the engineering plans (Appendix B – C300 Roading) and depicted in Figure 3 below is based on a connected public road network comprising a primary internal industrial collector loop road that provides connections to the existing off-site network (at SH39 and Onion Road), with all other internal roads generally formed as local industrial roads. The network is intended to provide legible movement through the site, support efficient industrial access, and allow for staged delivery whilst maintaining flexibility for future lot configuration. The internal roads are

proposed to vest as public roads. A 50 km/h design speed has been adopted for the internal collector and local industrial roads.

An Integrated Transportation Assessment has been prepared by CKL to assess the existing transport environment and the transport effects of the proposed development.

The proposed roading network has been designed having regard to the applicable road controlling authority requirements, Waikato District Council standards, the Waikato Regional Infrastructure Technical Specification (RITS), and relevant Waka Kotahi / NZTA requirements where works interface with the surrounding state highway network. The proposed road layouts are generally consistent with the applicable Waikato District Council standards, with identified departures addressed in the ITA.

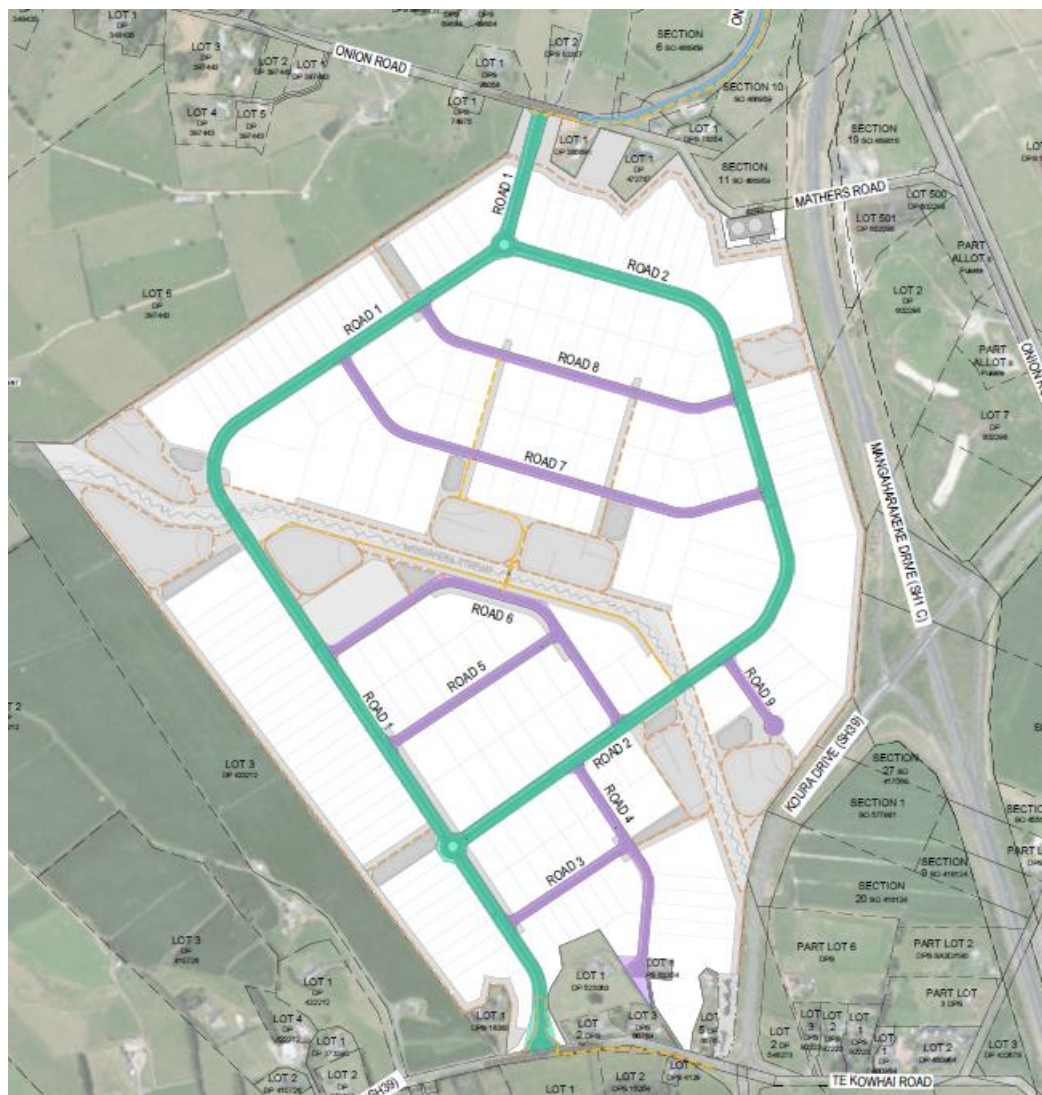


Figure 3: Roading Network Overview

3.2 ROADING NETWORK

3.2.1 Road Typologies and Cross-Sections

The internal network comprises a hierarchy of industrial collector and local roads designed to accommodate the anticipated traffic demands of the development. The access arrangements and associated external upgrades are described in Section 3.2.3.

Typical cross-sections for each road typology are summarised in Table 1 and illustrated in Figure 4 and Figure 5. The collector roads provide the primary movement corridors through the development and accommodate the principal utility and stormwater service corridors, while local roads provide access to individual development lots. The proposed road reserve and carriageway widths have been assessed in the ITA and form the basis of the current infrastructure design.

Collector road / collector road intersections are proposed as roundabouts, with other intersection forms reflecting the function and hierarchy of the intersecting roads. The road cross sections accommodate water supply, wastewater, stormwater, power, telecommunications and street lighting infrastructure within the service berms. Stormwater reticulation within the road reserve is integrated with the wider stormwater management system and, where required, contributes to the secondary overland flow network.

The streetscape layout shown on the typical cross-sections, including street trees, street lighting, parking bays and vehicle crossings, is indicative and subject to refinement at detailed design stage, consistent with the subdivision flexibility and servicing approach set out in Section 1.3, provided the approved road function and servicing requirements are maintained.

Table 1: Typical Road Cross-Section Summary

Industrial collector road	28.0 m	• Two-way carriageway • 11.0m trafficable width • 2 x 4.5 m lanes • 2.0 m Median provision	• 1.5m Footpaths both sides • 2.0m Off-road cycle paths both sides	• Indented parking bays
Industrial local road	20.0 m	• Two-way carriageway • 9.0 m trafficable width • 2 x 4.5 m lanes • No median proposed	• 1.5m Footpaths on both sides • No cycle facility	• Parking within traffic lane

Figure 4: Industrial Collector Road Typical Cross-Section

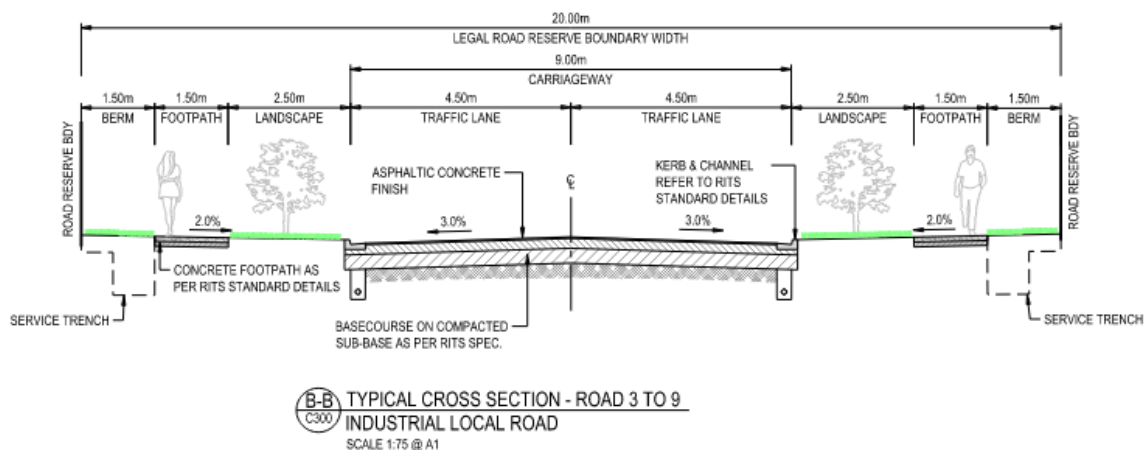


Figure 5: Industrial Local Road Typical Cross-Section

3.2.2 Walking, Cycling and Public Transport

The proposed roading network includes walking and cycling infrastructure to support safe movement within the development and to connect with the surrounding active mode network.

The internal collector roads will include segregated off-road footpaths and cycle facilities. The local industrial roads will include footpaths on both sides, with cyclists expected to share the road space due to the lower traffic volume and local access function of these roads.

Pedestrian refuge islands are provided at collector road / local road intersections where required to support safe crossing movements. Additional pedestrian and cycling connections are provided through reserves, stormwater corridors and open space areas where these improve connectivity across the development. A footbridge is also proposed across the Mangaheka Stream to provide a direct walking connection between the northern part of the development and the neighbourhood centre.

A shared path is proposed along the southern side of State Highway 39 between the site access and the SH39 / Burbush Road roundabout, together with a grade-separated crossing of SH39. The crossing may be provided as either an underpass or overbridge, with the final form to be confirmed through detailed design. These facilities have been agreed with NZTA. There are currently no committed public transport services within the site. However, the collector road loop has been designed to be capable of accommodating buses/bus stops in the future, should Waikato Regional Council introduce services to the development in the future.

3.2.3 Site Access and External Upgrades

As introduced above, primary access is proposed from Te Kowhai Road / State Highway 39 via a new single-lane roundabout west of the Koura Drive interchange, forming the principal connection to the wider strategic transport network. A shared path is also proposed along State Highway 39 to connect the development with the existing walking and cycling network. A secondary access is proposed from Onion Road via a priority-controlled intersection providing through site connectivity and network resilience. The section of Onion Road from the Site entrance through to Holmes Road will also provide for an urbanisation of the existing rural roadway to support future demands.

Any external transport improvements required to support the staged development, including intersection upgrades, pedestrian and cycle facilities, and associated transport infrastructure, will be delivered in accordance with the Integrated Transportation Assessment.

3.3 ENGINEERING EXCEPTIONS AND DEPARTURES

The roading design will generally seek to comply with the applicable road controlling authority standards and the Waikato Regional Infrastructure Technical Specification.

The ITA identifies areas where the proposed design differs from the Waikato District Plan requirements, including reduced footpath widths and reduced parking widths on selected typologies. These matters are assessed in the ITA and are considered acceptable in the context of the wider road corridor design, active mode provision, heavy vehicle requirements and industrial land use context.

Any other departures will be confirmed through the detailed civil design and engineering approval process.

4. STORMWATER

4.1 STORMWATER OVERVIEW AND DESIGN BASIS

4.1.1 Existing Stormwater and Catchment Context

The site is predominantly located within the upper Mangaheka Stream catchment, which forms part of a low-gradient rural land drainage network ultimately discharging toward the Waipā River system. The Te Otamanui catchment lies alongside the Mangaheka Stream catchment on the north side of Hamilton City and joins into the Waipā River at a higher confluence. A portion of the site also contributes to the Te Rapa catchment, which drains toward the Waikato River via downstream urbanised networks. The catchments are depicted in Figure 6 below.

The existing catchment is characterised by gently sloping rural land with limited fall, resulting in slow drainage response times and reliance on modified land drainage channels. Groundwater is also shallow across much of the site, limiting the ability to rely on soakage for stormwater management. The Mangaheka Stream corridor currently functions as a modified agricultural drainage system with constrained conveyance capacity and sensitivity to increased runoff volumes and velocities.

Existing stormwater flow paths generally follow natural topography, with overland flow dispersed across shallow gradients before converging into defined drainage channels and low points. The existing catchments are discussed in more detail in the Stormwater Management Plan (SWMP) included in Appendix E.

Key constraints influencing the proposed development include:

- Low-gradient terrain limiting hydraulic fall and discharge efficiency;
- Shallow groundwater limiting opportunities for stormwater soakage;
- Existing channel capacity constraints within the Mangaheka Stream system;
- Sensitivity of downstream rural land drainage infrastructure (LDMP-managed system);
- The need to accommodate external upstream catchment flows through the site;
- Localised floodplain ponding under high rainfall events; and
- Limited formalised stormwater infrastructure within the site.

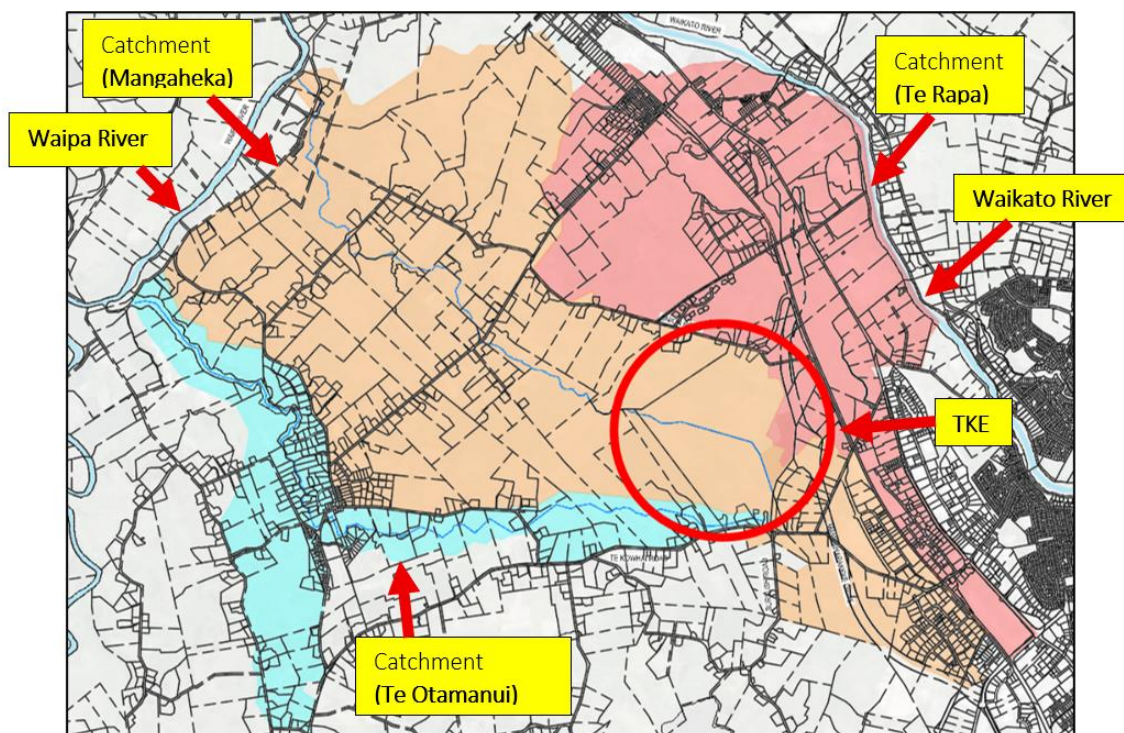


Figure 6: TKE relative to regional catchments – Mangaheka (Orange), Te Rapa (Red), Te Otamanui (Blue)

4.1.2 Design Criteria

The stormwater design for the Project has been developed in accordance with the project-specific Basis of Design contained within the SWMP. The Basis of Design draws together the relevant requirements of Stormwater Management Plan (SWMP) and Basis of Design (BoD);

- Mangaheka and Te Rapa Integrated Catchment Management Plans (ICMP);
- Regional Infrastructure Technical Specifications (RITS v2);
- NZTA / Waka Kotahi TR20-06 and TR20-07 guidelines (where applicable); and
- Waikato Regional Council Land Drainage Management Plan (LDMP).

The key design outcomes include stormwater quality treatment, retention and extended detention, attenuation of peak flows, safe conveyance of major storm events, allowance for climate change and maintenance of the required performance of the downstream drainage network. Detailed criteria are set out in the SWMP and its Basis of Design.

4.1.3 Proposed Stormwater Strategy

The stormwater strategy for the Project adopts an integrated catchment-wide treatment train approach incorporating source control, conveyance, attenuation, and discharge management. The strategy has been developed in response to the low-gradient site, shallow groundwater, existing rural drainage network, external catchment flows and sensitivity of the downstream receiving environment.

Key components of the system include:

- Piped reticulation supported by open conveyance channels and overland flow paths;

- Eight centralised wetlands providing treatment, extended detention and peak-flow attenuation;
- Reconstruction of the Mangaheka Stream corridor to improve conveyance, attenuation and ecological function;
- Raingardens and other retention measures to manage the initial abstraction volume;
- On-lot source control and treatment requirements for future industrial development; and
- Progressive implementation aligned with staged development.

The stormwater system will be delivered progressively in alignment with development staging.

Each stage will be hydraulically self-sufficient and will include:

- Fully functional stormwater conveyance systems;
- Appropriate treatment and attenuation capacity;
- Temporary works where required to manage interim conditions; and
- Transition pathways to ultimate infrastructure configuration.

Infrastructure triggers will be linked to catchment activation and impervious area thresholds.

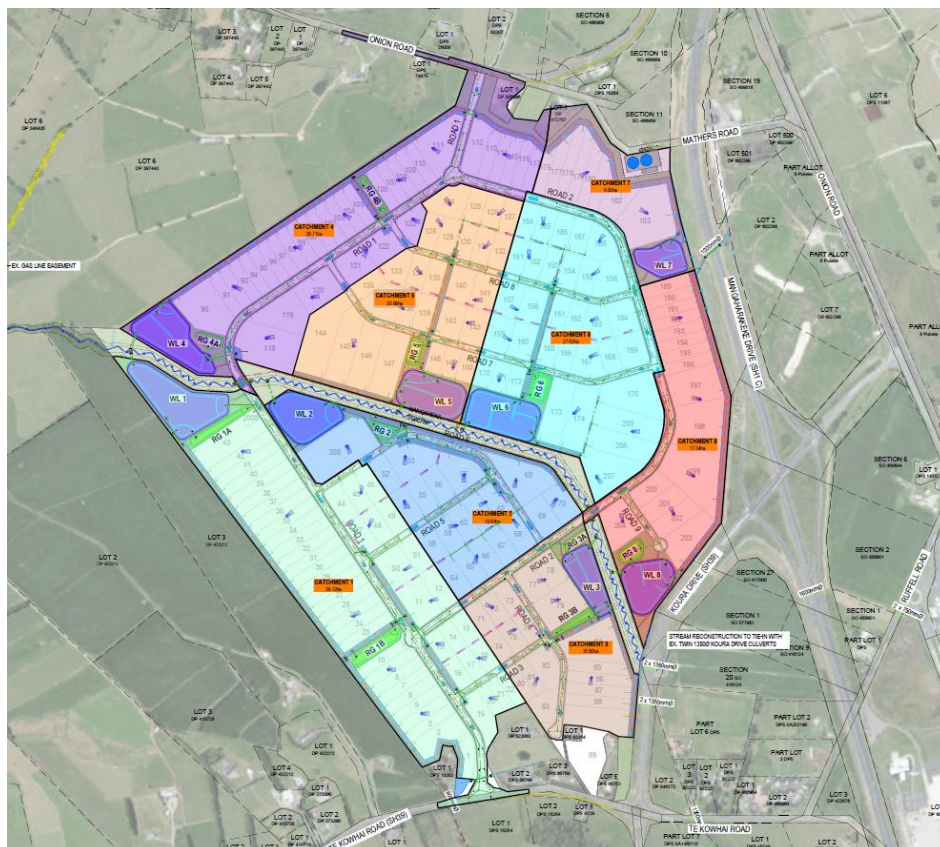


Figure 7: Proposed Catchment Boundaries

The stormwater management strategy has been developed based on the on-site stormwater catchments (illustrated in Figure 7: above), development cells and staged infrastructure delivery, rather than relying solely on the exact lot boundaries shown on the scheme plan.

The stormwater reticulation and conveyance network will be designed to service each stormwater catchment, block or development cell within a stage. Pipework, conveyance channels and outlet

structures are sized to provide servicing flexibility for future lot configuration changes, provided those changes remain within the approved catchment and staging framework. Individual lot connections shown on the engineering plans are indicative and are subject to confirmation at future detailed design stage, when the final lot arrangement is confirmed.

This approach allows the final subdivision lot layout within each stage to be refined based on market demand and purchaser requirements, before 223 / 224c certification, while maintaining the required stormwater treatment, attenuation, conveyance, overland flow and discharge outcomes.

In addition to providing stormwater treatment, attenuation, conveyance and flood management, the wetlands, conveyance channels and Mangaheka Stream corridor are intended to contribute to ecological enhancement, landscape integration, amenity and walking/cycling connectivity, where compatible with the primary stormwater function.

4.1.4 Hydraulic Modelling and Key Outcomes

Te Miro Water has undertaken hydraulic modelling of the site and downstream receiving environment for existing and post-development conditions. The modelling confirms that pre-development flows and velocities are maintained for the 2, 10 and 100-yr rainfall events. As a result, pre-development peak water elevations have also been maintained downstream of the site in all rainfall events.

While the proposed attenuation measures have managed to mitigate post-development peak flows (i.e. the rate at which stormwater leaves the site), they cannot reduce the volume, and the receiving watercourses carry elevated flows for a longer duration and longer drain down times after rainfall events. This additional duration has the potential to increase the risk of stream erosion over time. Key hydraulic outcomes are further discussed in Section 5 of this report and residual volume effects, and the proposed framework for mitigation are discussed in depth separately in the SWMP.

4.2 STORMWATER CONVEYANCE

The stormwater conveyance system comprises a hierarchical network of piped reticulation, open channels, and overland flow pathways designed to convey runoff efficiently through the development.

The network includes:

- Primary conveyance systems via pipes and open channels;
- Secondary flowpaths within road corridors supported by open channels;
- Local lot-level connections and drainage pathways.

Pipe reticulation forms a key component of the stormwater network and is primarily located within road corridors, providing collection and transfer of runoff to downstream treatment and attenuation systems.

Open channel systems are used where topography and hydraulic efficiency support surface conveyance. The channels have been proposed where a piped network becomes impractical given the flat gradients across the site and reduces the requirement. These channels provide conveyance for both the primary network and to also safely contain flood waters as part of the secondary network. The provision of these conveyance channels limits the requirement of providing for large diameter pipe network and the associated ongoing maintenance burden.

These channels will be vested as local purpose stormwater drainage reserves. These channels are typically 12.5m wide and will have planted batters. Channels where they are not located adjacent to a public road reserve, have been provided with a dedicated connection to the road and a minimum 3m wide access track alongside to allow periodic maintenance. These are predominantly located along the perimeter of the site and a typical section is shown via Figure 8 below.

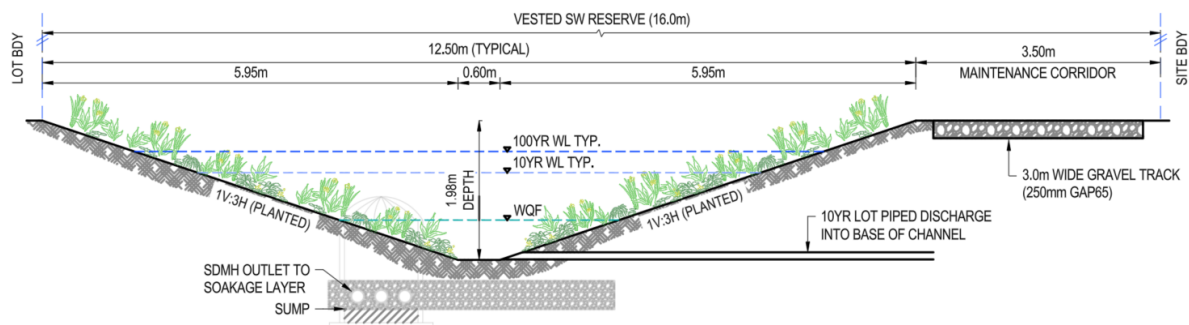


Figure 8: Conveyance Channel Typical Section

4.2.1 Mangaheka Stream Corridor Works

The Mangaheka Stream corridor forms a key hydraulic and ecological feature of the stormwater system. Proposed works include widening, lowering, and reconstruction of the channel to improve conveyance capacity, provide additional flood storage and attenuation, and reduce erosion risk under post-development conditions. The corridor functions as both a stormwater conveyance system and a controlled overflow pathway. Key elements include:

- Enhanced channel geometry and capacity improvements;
- Integration of restricted culverts and overflow spillways;
- Erosion management for frequent storm events (2-year ARI); and
- Staged construction aligned with development sequencing.

The Mangaheka Stream is proposed to be lowered within the site to improve flood conveyance and reduce flood risk. The lowered stream corridor has, where practicable, been integrated with stormwater management, and ecological enhancement. The lowered channel is currently anticipated to extend approximately 400 m downstream of the site and into the properties at 302, 360 Te Kowhai Road, and 196 Onion Road, to provide a transition into the existing stream alignment. Typical details for the proposed reconstruction of the Mangaheka Stream are shown via Figure 9.

The reconstructed stream cross-section will include a meandering baseflow channel that is connected to newly created permanent pools to provide a natural refuge for native fauna. Batters will form the embankment of the new stream cross-section and maintenance access will be provided at the top of these embankments. The stream will be vested to Council as a stormwater drainage reserve.



Figure 9: Proposed Mangaheka Stream Corridor (Cross-Section)

Meandering of the low flow channel within the stream will be detailed for approval during EPA. The design will aim to reflect the natural forms and character of a stream and be composed with ‘mosaic’ features such as riffles, runs and/or pools where practical as illustrated in Figure 10 below.

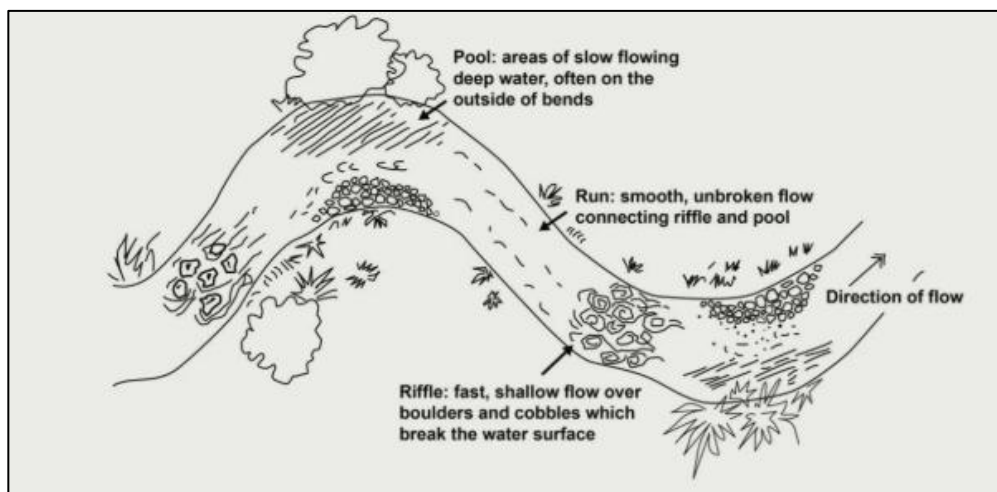


Figure 10: Riffle, run and pool habitats (from Clapcott et al, 2011)

4.2.2 Industrial Lot 89 – Specific Discharge

The southern portion of Lot 89 is bisected by the existing Te Otamanui Stream, therefore is physically separated from Catchment 3 in the post-development model. The rest of Lot 89 (its northern portion) per Figure 11 below is located within Catchment 3 and provisions to connect into the stormwater network has been provided in the design. The existing ground levels within the southern portion of the lot are not proposed to be changed, and therefore this area of land retains its existing discharge directly to the Te Otamanui Stream.

This application does not seek to develop within the southern portion of this Lot and therefore no stormwater outlet or mitigation is proposed for this area. As the specific development outcome for this Lot is currently unknown, the application proposes a consent notice condition on the subdivision consent requiring:

“Lot 89 shall be subject to site-specific stormwater design at the time of development, demonstrating that stormwater runoff is directed to the stormwater network serving Catchment 3, or where that is impracticable, is appropriately managed before discharge to the Te Otamanui Stream.”

If the southern portion of Lot 89 is to be developed in future, then prior to any discharge into the Te Otamanui Stream, attenuation of the 2, 10 and 100 Year ARI peak flows will be required, and a new consent shall be sought.

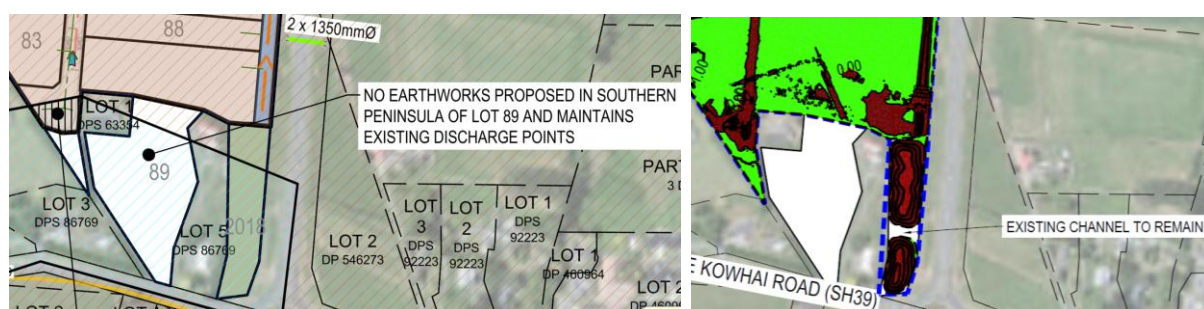


Figure 11: Lot 89 Peninsula – Existing Discharge Strategy

4.3 STORMWATER QUANTITY MANAGEMENT

Stormwater quantity is a key design consideration for the Project given the change from predominantly rural land to a large-scale industrial development, the low-gradient receiving environment and limited opportunities for stormwater soakage.

Stormwater quantity is proposed to be managed through a combination of flow control, attenuation, and storage systems designed to manage peak flows and runoff volumes. The proposed attenuation measures have been developed to ensure pre-development flows are maintained for all major storm events. The hydraulic modelling confirms that peak-flow and flood-level effects can be appropriately managed.

However, while peak flows (i.e. the rate at which stormwater leaves the site) have been mitigated, the proposal has not reduced the volume, and the receiving watercourses carry elevated flows for a longer duration and longer drain down times after rainfall events. This additional duration has the potential to increase the risk of stream erosion over time. TKE have proposed an ‘adaptive residual management’ approach to mitigate the effects from the increased volume and is discussed in detail in Section 1.6 of the SWMP.

Key components of the quantity management system include:

- Flow attenuation for 2-year, 10-year, and 100-year ARI events;
- Wetland and basin storage systems providing detention capacity;
- Flow control devices regulating discharge rates; and
- Restricted culverts and overflow spillways located within the central Mangaheka reconstructed corridor, designed to safely convey exceedance flows during extreme events.

4.3.1 Wetlands

Wetland bathymetry has been designed in general accordance with the permanent storage zone parameters provided in RITS v2. Shallow and deep marshes have been distributed within the footprint to support emergent macrophyte species. Intermediate deep pools have also been provided to provide habitat diversity. A deep outlet pool is provided where the wetland outlets to the stream to allow cooler waters to be drawn in between larger storm events.

The bottom of the wetland will be made impermeable to at least the PWL to prevent water losses. This impermeable lining will either be composed of compacted clay or be underlain with geo-synthetic clay liners (GCL) or HDPE. Selection for the base of the wetlands will be detailed for approval during EPA.

Forebays have been preliminarily sized to meet a minimum area of 10% of the artificial wetlands and minimum 15% of the WQV per RITS. Inflows into the wetlands have been amalgamated as much as possible, but where impractical due to gradients, additional forebays have been provided to provide the pre-treatment of medium-coarse sediments. Examples of where additional forebays are required are shown via Figure 12 below. All forebays proposed in the design are proposed to be separated from the main wetland body with an impermeable compacted earth bund set to the PWL.



Figure 12: Typical Additional Wetland Forebay Configuration

Wetland outlet structures have been located within or adjacent to safety benches in the main body of the wetland. Vehicle access, for a ute, has been provided to at least 50% of the wetland perimeter and to all hydraulic structures. Pedestrian access for maintenance staff is also available around the entire perimeter of the wetland. Maintenance vehicle tracking has been undertaken within Catchment 1 as a proof of concept per Maven DWG C445. The maintenance accessway for the remaining Catchments will be detailed for approval at the EPA stage.

The TKE wetlands have a large footprint and as such their forebays all exceed the minimum 10m reach from the maintenance track. To ensure the forebay can be cleaned out without the use of special equipment, an extension of the maintenance track will have to be provided into the base of the forebay. The proposed concept is indicated in Figure 13 below. The hardstand will be designed to support the loading of a suitable excavator and will be detailed for approval during EPA.

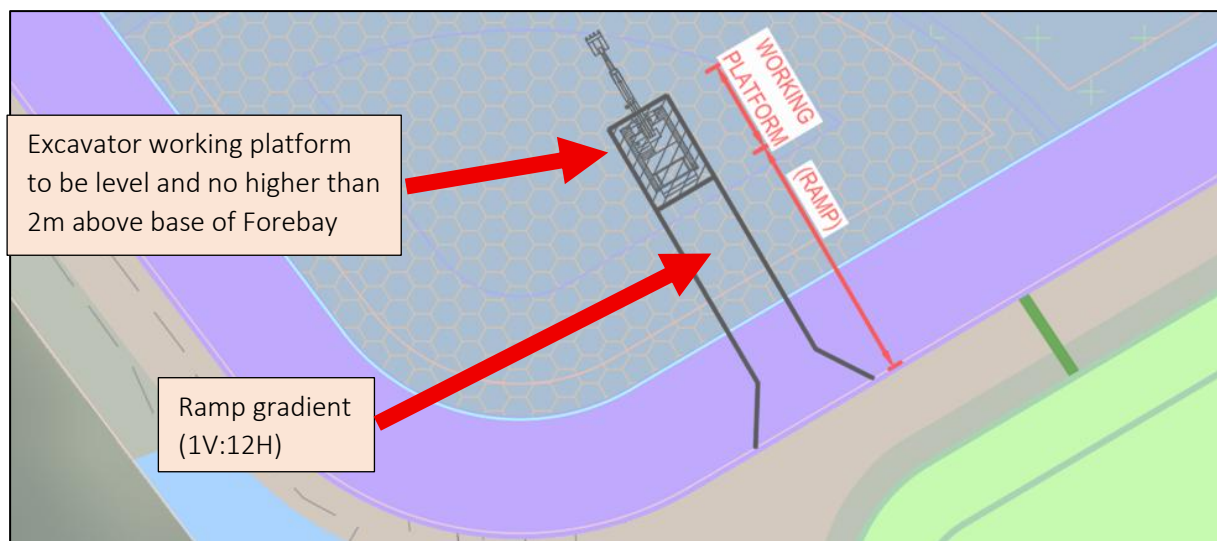


Figure 13: Extension of maintenance track into base of forebay (Concept)

A 2m wide safety bench with a maximum slope of 1V:8H is provided below the PWL around both the main body of the wetland and the forebay. A stabilised emergency spillway will also be integrated into the top of the wetland to allow for safe and controlled discharge in the event of full blockage of the wetland outlets. Permeable fencing will be also provided within the proposed landscaping design and will be detailed for approval during EPA.

4.3.2 Hydraulic Structures and Culverts

Hydraulic structures include culverts and outlet structures required to manage stormwater conveyance and transitions across the network. These include:

- Road crossing culverts and piped transitions;
- Stream and channel crossings via culverts and bridges;
- Outlet control structures regulating discharge rates;
- Energy dissipation structures at discharge points; and
- Overflow and bypass structures within the Manganahaka corridor system.

All outlet structures under the proposed stormwater system are proposed with energy dissipation devices to mitigate erosion and local scour. High-flow bypasses where appropriate have been provided into the body of the wetland and will be provided with the same riprap aprons to mitigate the effects of scour for up to the 100-Year event. The locations for these devices have been indicatively shown in the engineering drawings and will be detailed for approval during EPA.

Scruffy dome outlet(s) with orifice and weir controls, specific for each wetland, have been designed to provide attenuation of the EDV, 2-Year, 10-Year and 100-Year storm events. A submerged pipe outlet below the PWL will draw off cooler water from the outlet pool, thereby reducing temperature-related effects on the receiving environment. A typical wetland outlet configuration is shown via Figure 14 below.

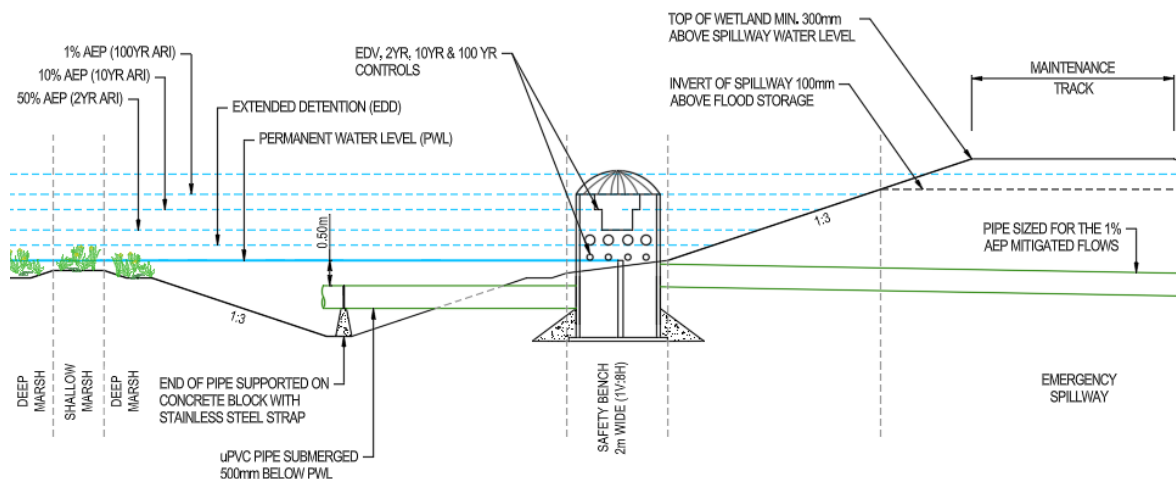


Figure 14: Typical Wetland Outlet Configuration

The development requires several culverts to ensure connectivity of the proposed conveyance channels under the proposed roading corridors. There are two culverts in the Mangaheka Stream (M.1 & M.2) and one within the Te Otamanui Stream (O.1), and these have been designed in accordance with the requirements of the National Environmental Standards for Freshwater (NES-F), to provide for fish passage. A typical culvert long-section allowing for fish passage is shown via Figure 15 below. These will be further detailed at the Engineering Approval stage via HY-8 or similar hydraulic modelling software.

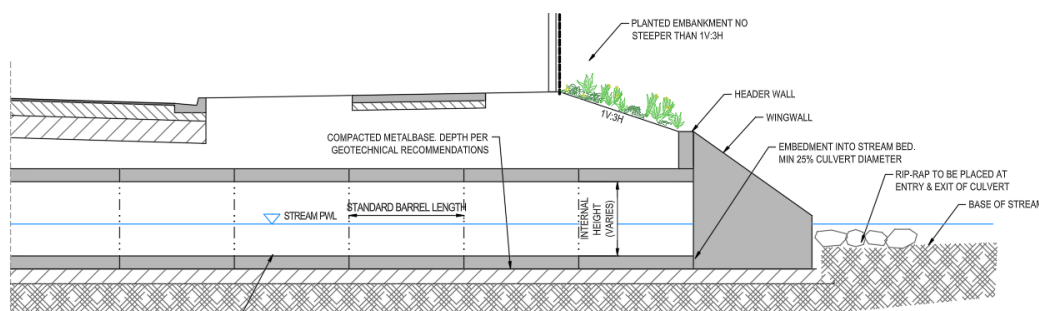


Figure 15: Typical Culvert Long-Section with Fish Passage

The reconstruction of the Mangaheka Stream improves tailwater conditions downstream of Koura Drive and the conveyance capacity of the channel which correspondingly increases peak flows within the stream in a post-development scenario. Restricted culverts have therefore been proposed within the stream to mitigate peak flows which provide a level of online attenuation with the additional storage capacity provided by the widened and deepened channel.

An embankment is proposed to contain the culvert with the top level to correspond with the 100-Year ARI water level. A bridge will be constructed above the embankment to provide a spillway which will activate if a blockage scenario occurs which ensure that maximum water level within the stream will not flow back within the site in an emergency scenario providing overall resilience to the design solution. A typical attenuation culvert arrangement is shown in Figure 16 below.

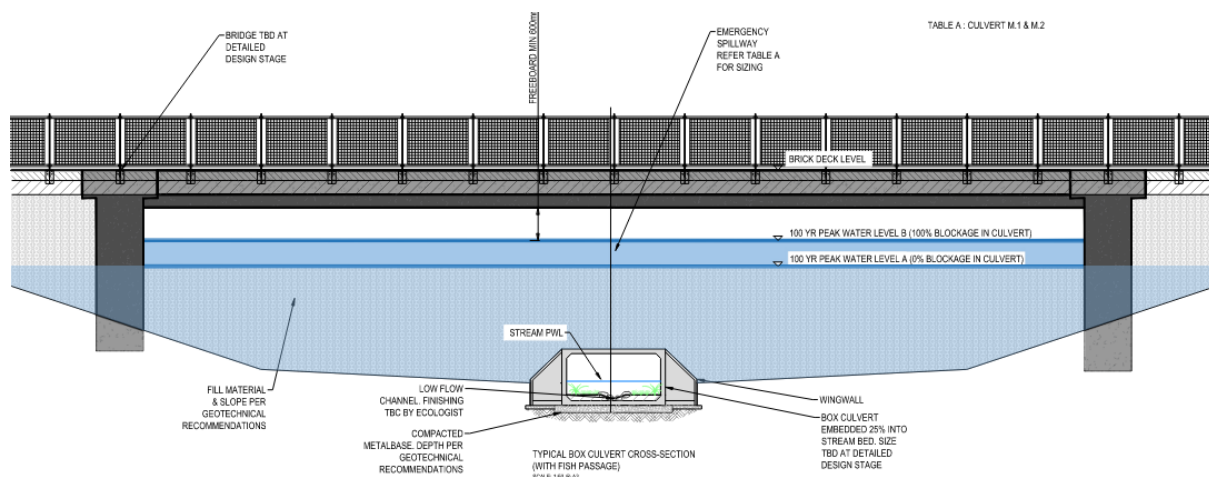


Figure 16: Mangaheka Attenuating Bridge-Culvert Typical Configuration

4.4 STORMWATER QUALITY TREATMENT

4.4.1 Treatment Train

Stormwater quality treatment is provided through a treatment train approach consistent with SWMP and BoD requirements. The treatment system includes:

- Lot-scale source control measures where appropriate, to be constructed at a later stage when lots are developed by future purchasers;
- Primary treatment via planted conveyance channels;
- Integration of bioretention of the initial abstraction via raingardens where appropriate; and
- End-of-line treatment via constructed wetlands.

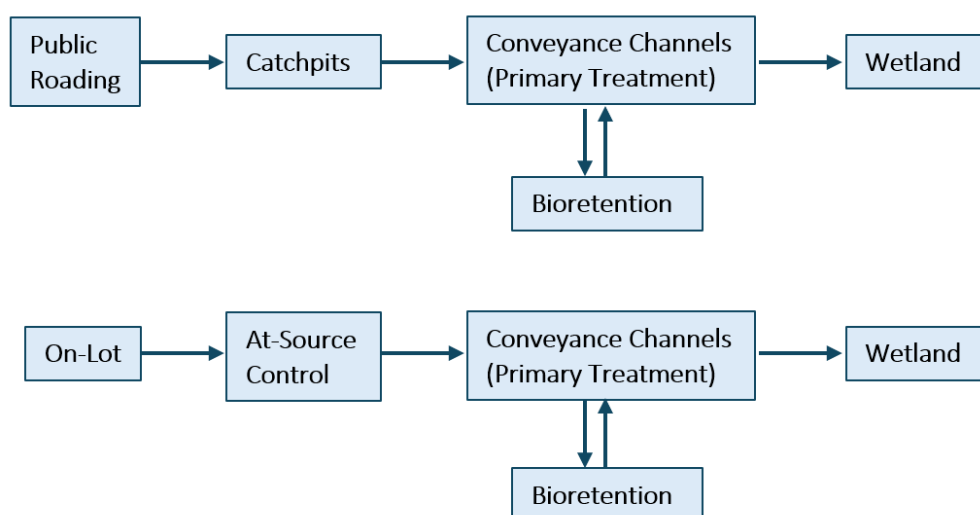


Figure 17: Treatment Train for Public Roading and Lots

The system is designed to achieve long-term water quality performance for key contaminants including suspended solids, hydrocarbons, and nutrients.

4.4.2 Conveyance Channels

The proposed stormwater channels described in Section 4.2 provides conveyance of the primary and secondary flow paths but also provide a primary treatment function before discharging into their respective wetlands for final treatment. These channels will be planted and have been designed to provide an average 9-minute or longer hydraulic residence time in accordance with RITS v2 and TR20/07.

4.4.3 Raingardens

Amalgamated raingardens are proposed to provide bioretention of the initial abstraction volume from public road catchments. The raingardens have been consolidated where practicable to reduce the number of vested stormwater assets and simplify ongoing maintenance per consultation with Council stakeholders.

Stormwater runoff captured by the raingardens has not been proposed to discharge directly to ground as dual treatment is first required for industrial zones in accordance with RITS v2 and TR20-07. All raingardens are therefore proposed to be lined and bioretention will be achieved through evapotranspiration per the TR20-07 guidelines. A typical section is shown via Figure 18 below.

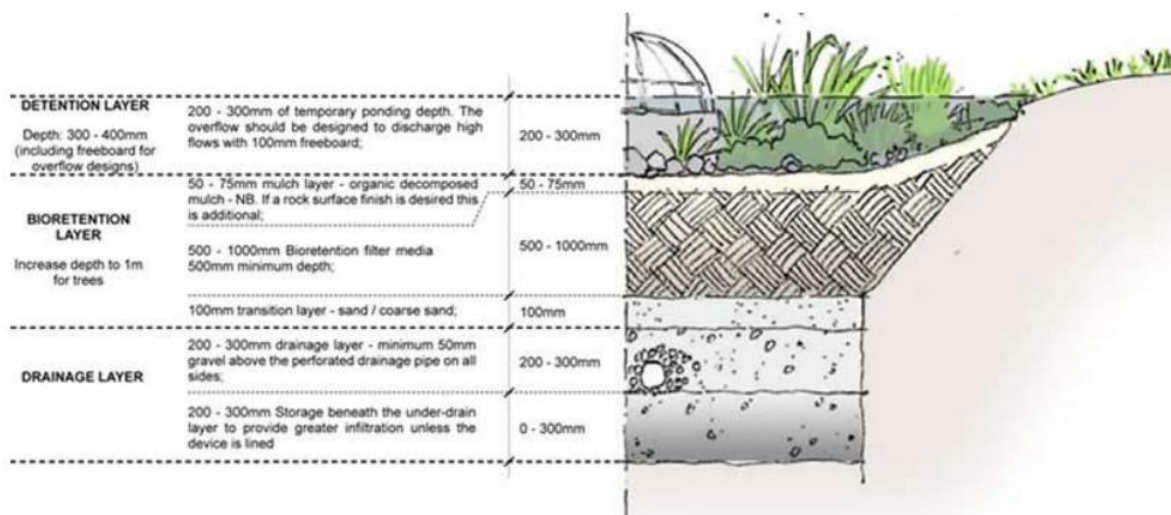


Figure 18: Amalgamated Raingarden Typical Section

Due to the limited gradient available within the site, runoff from the roading stormwater network has been designed to discharge to the open conveyance channels as soon as practicable to minimise overly large pipe diameters. The raingardens will be hydraulically connected to the conveyance network through low-flow diversions, with bypass and overflow arrangements for larger events. The diversion will use an earth bund and auxiliary pipe that connects the invert of the conveyance channel to the amalgamated raingarden. A typical arrangement of the diversion is shown via Figure 19 below.

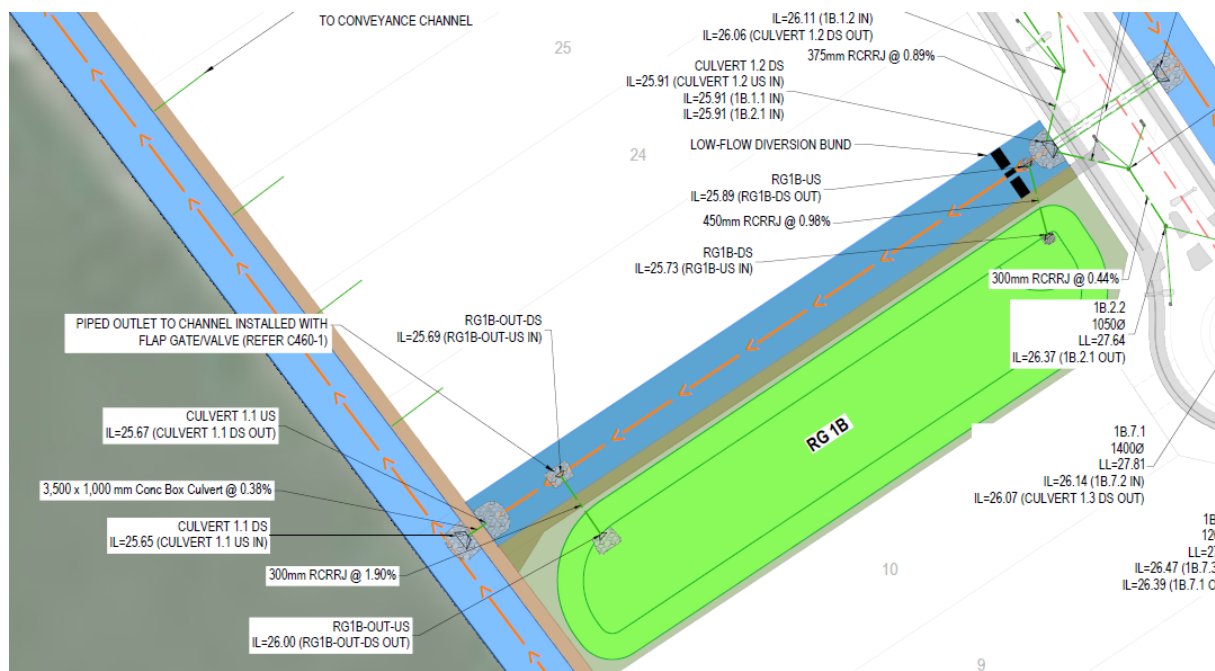


Figure 19: Low-Flow Diversion Typical Configuration

The auxiliary pipe will be sized with the WQF as its capacity and earth bund built to the soffit of the pipe. Runoff generated from events exceeding the WQF will overtop the bund and bypass the raingarden.

Because this diversion naturally needs to happen at the invert of the conveyance channel, the raingarden is set to a similar or slightly lower datum for water to flow. The limited gradients in TKE mean that ponded water within the raingarden cannot discharge out conventionally to a lower landform with an above-ground outlet. Post storm event, conveyance in the low-flow pipe is expected to reverse to aid with the drain-down as water in the conveyance channel subsides.

To facilitate drain-down, a secondary piped outlet downstream of the low-flow diversion bund will be provided 300mm above the raingarden media to assist with draining any ponding within the raingarden within 24-hours. The secondary outlet is set 300mm above the media to simulate similar ponding depths expected within road-side raingardens. A flap/gate valve will be installed over the secondary outlet to prevent the backflow of attenuated waters within the conveyance channels from entering the raingarden.

An emergency spillway from the raingarden into the conveyance channel will be provided to provide a controlled means of overtopping. The spillway will be built into the maintenance track and will be detailed for approval during EPA.

For completeness, Maven has also investigated and provided an option for roadside modular raingardens as an alternative for Council's consideration via Maven Engineering Drawing C410. These would be elevated higher than amalgamated raingardens and would be more readily accessible. These higher modular raingardens also provide a higher potential for providing a soakage layer by having an increased clearance to groundwater levels and would help contribute to post-development volume neutrality for the 2-Year ARI event.

4.4.4 Wetlands

End-of-line of the treatment train is proposed via one of the eight new centralised wetland devices. These provide the greatest potential for treatment of the anticipated contaminants. The shallow and deep marshes in the main body of the wetland, together with their deeper pools will provide treatment of the WQV and also extended detention. Refer to the landscape plans provided in the application for the proposed wetland planting species.

4.4.5 Private On-Lot

Stormwater discharges from industrial sites must be managed in accordance with the design criteria in TR20-07 and must be supported by Pollution Control Plans. TR20-07 Table 13-1 sets out various industrial activities, their contaminants of concern and acceptable on-lot treatment processes. The private treatment device must be fit for purpose based on the proposed industrial activity and submitted for approval under future Building Consent applications. A template for a Pollution Control Plan is provided in the SWMP. A typical on-Lot (at source) treatment system is shown in Figure 20 below. Consent notices will be provided for the industrial Lots to ensure future owners comply with this requirement.

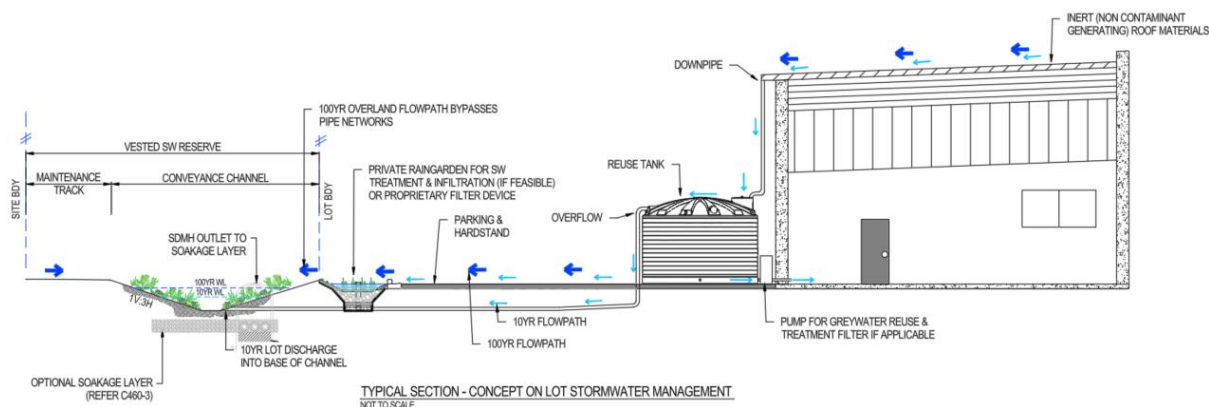


Figure 20: Typical On-Lot Private Treatment Configuration

4.5 OPERATION, MAINTENANCE AND OWNERSHIP

Stormwater assets will be managed through a combination of vested public infrastructure and privately maintained systems, depending on final ownership arrangements. Maintenance requirements include:

- Regular inspection and maintenance of treatment systems;
- Sediment removal from wetlands and basins;
- Maintenance of conveyance channels and outlets;
- Vegetation management within stormwater corridors; and
- Provision of maintenance access to all key infrastructure.

An adaptive management framework will be applied to ensure long-term system performance.

4.6 ENGINEERING EXCEPTIONS AND DEPARTURES

Any departures from RITS v2, TR20, ICMP, LDMP or SWMP requirements are identified and assessed in this section.

Each departure includes:

- engineering justification;
- effects-based assessment under the FTAA framework;
- alternative solutions considered;
- mitigation measures; and
- stakeholder acceptance pathway.

4.6.1 Increase in Post-Development Stormwater Volumes

While the proposed attenuation measures manage post-development peak flows, full volume neutrality for the 2-year ARI event is not achieved. Approximately 44,000 m³ of additional runoff is modelled to discharge to the Mangaheka catchment and approximately 4,000 m³ to the Te Rapa catchment.

The increased runoff volume results in elevated water levels persisting for longer within the receiving watercourses and has the potential to increase stream erosion over time. This is the principal residual stormwater effect identified by the SWMP.

The Project therefore adopts an adaptive residual management approach. The approach seeks to progressively reduce the residual volume through on-lot rainwater reuse, soakage where demonstrated to be practicable, and other retention measures as individual industrial lots are developed. The residual effect will be monitored and reassessed as development proceeds, with downstream mitigation and financial contribution requirements applied in accordance with the consent conditions.

Further detail on the assessment of alternatives, monitoring, geomorphological assessment and residual mitigation framework is provided in the SWMP.

5. FLOODING AND OVERLAND FLOW MANAGEMENT

5.1 FLOODING OVERVIEW

Te Miro Water has undertaken integrated hydraulic modelling of the site and downstream receiving environment for existing and post-development conditions, including allowance for climate change, development staging and relevant blockage and sensitivity scenarios.

The modelling has been used to assess flood extents, levels, velocities and hazard; overland flow paths; road and stream conveyance; downstream flood effects and drainage performance; and the flood-management requirements for the proposed development. Further detail on the modelling methodology and results is provided in the SWMP and Te Miro Water Model Build Report.

This section assesses flood hazard, overland flow, and flood risk effects associated with the development for the 2-, 10- and 100-Year ARI storm events.

Flood modelling and flood hazard assessment for the development have been carried out by Te Miro Water using the hydraulic model described in the SWMP. The modelling assesses existing and post-

development flood extents, levels, velocities, hazard and overland flow behaviour across the site and downstream receiving environment.

5.1.1 Existing Flood Environment

The existing site is generally characterised by low-relief rural terrain with shallow overland flow paths and limited hydraulic gradients, including low-lying land associated with the Mangaheka Stream and the existing rural drainage network.

The Mangaheka Stream currently provides the primary flood conveyance corridor for the site, with additional overland flow dispersal across adjacent rural land. Pre-development peak flood flows are not able to be contained within the current stream banks, and flood flows currently spread across the low-lying portions of the site. This means the site already experiences flooding under existing, pre-development conditions.

The existing flooding and drainage environment has informed the proposed landform strategy, including the need to provide improved conveyance through the Mangaheka Stream corridor, coordinated stormwater reserve levels, appropriate development platform levels, and dedicated overland flow routes.

5.2 FLOOD MANAGEMENT FOR THE DEVELOPMENT

5.2.1 Staging and Implementation

Flood behaviour is assessed on a staged basis consistent with the development sequencing.

Consistent with the overall staging approach described in Section 1.3, flood management infrastructure will be delivered progressively with the staged development of the site, with each stage providing a functional stormwater and overland flow system that safely manages runoff and maintains appropriate downstream discharge conditions; shared or downstream flood management infrastructure may need to be delivered ahead of the stage that physically contains it where required to support upstream development or provide a complete flood conveyance route.

Temporary flood management measures may be required during construction or interim stages, addressed through detailed design and construction methodology.

5.2.2 Overland Flow Path (OLFP)

The OLFP system provides defined pathways for the safe conveyance of flood flows across the site.

The system includes:

- Principal OLFP corridors (fixed and protected);
- Secondary OLFP routes (flexible within development blocks); and
- Emergency exceedance pathways for extreme events.

The proposed overland flow strategy generally separates road-generated overland flow from lot-generated overland flow where practicable. Public road corridors will generally convey road-generated overland flow with hump and hollow profiles and outlet connections designed to direct flows toward stormwater reserves, conveyance channels or other approved discharge locations. Lot-generated overland flow will be directed towards conveyance channels, stormwater reserves, nominated overland

flow corridors or an approved discharge location, where road corridors do not have sufficient capacity to reliably convey the full lot-generated flow from the development.

Section 1.3 of this report outlines the flexible subdivision strategy sought with this application. For an industrial development of this scale, future purchaser or tenant requirements are likely to influence final lot sizes and boundaries. Each future lot will be provided with an approved overland flow discharge location or discharge corridor, identifying where lot-generated overland flow is required to leave the site and connect to the downstream overland flow or stormwater conveyance system. Future lot development will need to maintain this approved location and avoid blocking or redirecting flows in a way that adversely affects adjoining lots, public roads, stormwater reserves or downstream infrastructure.

Public conveyance channels have been proposed catchment wide and provide 'principal' overland flow paths for TKE. Principal overland flow paths and drainage corridors that perform a catchment-scale function will be protected as a public drainage reserve. In situations where a Lot does not have direct frontage to these public conveyance channels, a 'secondary' overland flow path has been designed into the proposed terrain to ensure Lot generated sheet flows can be appropriately discharged to the public channels without negatively impacting downstream Lots.

The 'secondary' flow paths presented in this application are provided as a proof-of-concept based on the current Lot layouts. Figure 21 below shows a typical 'principal' and 'secondary' overland flow path configuration based on the proposed masterplan. These could be subject to change pending the ultimate Lot and boundary configuration based on future purchaser requirements. At such time, an engineering assessment, based on the ultimate Lot configuration, must be submitted with the future Building Consent demonstrating that the proposed built form does not negatively impact the neighbours and retains a connection to the public channels. Certification by a suitably qualified engineer must accompany the Building Consent application and should confirm that the flooding design intent prepared under this application has been maintained.

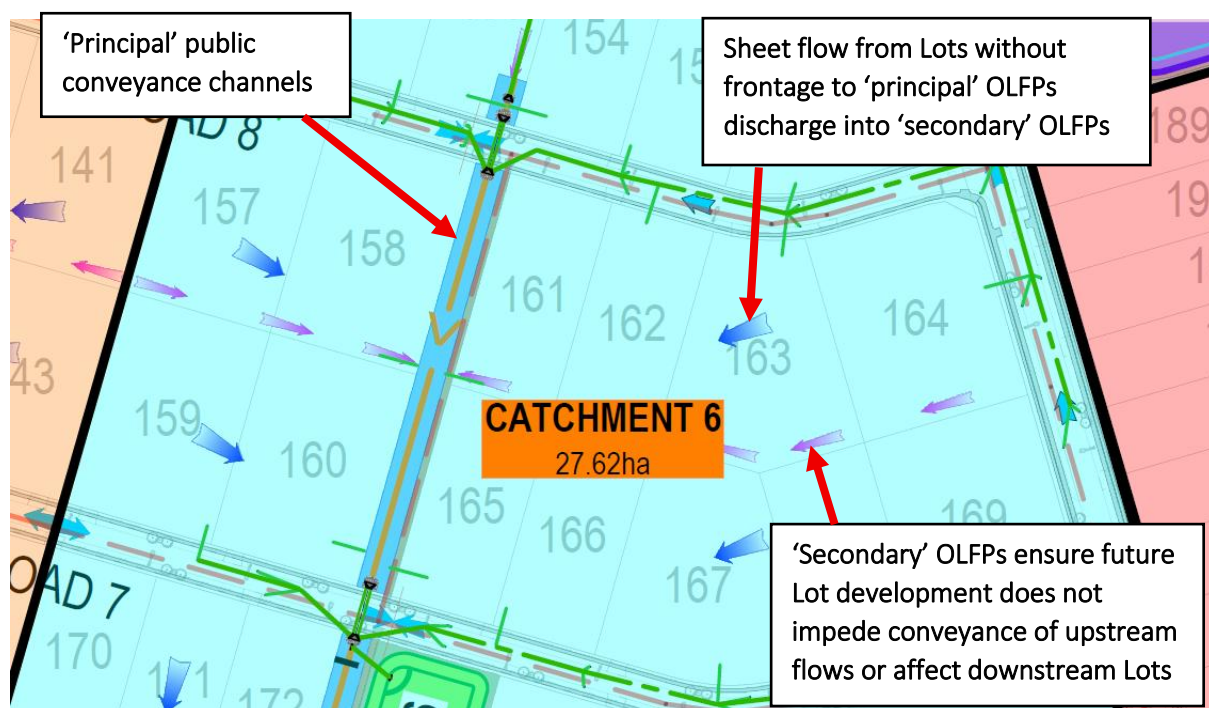


Figure 21: Typical 'Principal' and 'Secondary' OLFP configuration

A consent notice is proposed in accordance with Recommendation 9 'Overland Flow Path Protection' outlined in Section 10 of this report to ensure future development adheres to this requirement. The final secondary flow path locations within internal lots shall be secured or protected via easements provided at 223 to Council's satisfaction.

5.2.3 Mangaheka Stream Flood Conveyance

The primary flood conveyance system is the Mangaheka Stream corridor, supported by secondary conveyance through roads and open stormwater channel networks.

The Mangaheka Stream corridor that bisects the site also receives flows from external HCC catchments upstream of the site. As previously discussed in Section 5.1.1, the Mangaheka Stream which currently provides the primary flood conveyance corridor, does not adequately contain pre-development peak flows within its banks, and flooding spills across the site. Reconstruction of the Mangaheka Stream includes widening and lowering the corridor to provide improved conveyance capacity for upstream flows and internal stormwater discharges. The reconstructed corridor will include a low-flow channel within a wider flood conveyance corridor, integrated with stormwater reserves, ecological planting, maintenance access and erosion protection.

Reconstruction of the Mangaheka Stream improves the capacity for flood conveyance within its new corridor and in doing so effectively improves tailwater conditions downstream of the Koura Drive culverts. The attenuation culverts within the two road crossings over the stream manages this tailwater improvement by temporarily storing and releasing flood waters gradually. This ensures downstream peak flow rates are maintained while making use of the additional storage created by the reconstructed channel.

The reconstructed corridor will provide a more defined and resilient flood conveyance route through the site, while also supporting improved drainage gradients, stormwater device performance, developable platform levels and building freeboard. The system ensures hydraulic continuity across all development stages and maintains downstream drainage function.

The final stream levels, corridor width, floodplain shape, erosion protection, planting, maintenance access and downstream transition works have been confirmed through flood modelling, stream works design, ecological assessment, the Streamworks Management Plan, and will be subject to approval at future detailed design stages.

5.3 FLOOD EFFECTS AND RISK ASSESSMENT

Key features of the proposed flood management system include:

- Modification to the central Mangaheka Stream to provide increase of hydraulic grades laterally across the development, increase of flood conveyance capacity from external flows and online mitigation to avoid adverse flooding effects.
- Eight new artificial wetlands, one for each sub-catchment, to attenuate their respective post-development runoffs from the 2-Year, 10-Year and 100-Year ARI storm events back to or below their existing pre-development flow rates. Notably, the level of attenuation has also considered the effects of climate change within the development and mitigates for this future increase in rainfall as well.

5.3.1 Existing Vs Post-Development

The site discharges most of its runoff into the Mangaheka catchment, with a smaller portion along the north-eastern portion of the site discharging to the Te Rapa catchment. Table 2 and Table 3 below summarise the reduction in post-development peak flows achieved with the proposed wetlands and demonstrates that the effects from the development will be appropriately mitigated within its boundary.

Table 2: Peak Flow Mitigation Results – TKE Site Only Discharge to Mangaheka

TKE Site Only – Discharge to Mangaheka (Catchments 1 – 6 & 8) – Measured at Point D				
ARI Event	Pre-Development (No CC) (m ³ /s)	Post-Development (With CC) (m ³ /s)	Peak Flow Reduction (m ³ /s)	Post % of Pre
2-Year	2.30	1.51	- 0.79	66%
10-Year	4.93	3.39	- 1.54	69%
100-Year	9.93	9.34	- 0.59	94%

Table 3: Peak Flow Mitigation Results – TKE Site Only Discharge to Te Rapa

TKE Site Only – Discharge to Te Rapa (Catchment 7) – Measured at Point ‘Te Rapa’				
ARI Event	Pre-Development (No CC) (m ³ /s)	Post-Development (With CC) (m ³ /s)	Peak Flow Reduction (m ³ /s)	Post % of Pre
2-Year	0.35	0.25	- 0.10	71%
10-Year	0.79	0.55	- 0.24	70%
100-Year	1.37	1.10	- 0.27	80%

An assessment at a wider catchment scale, beyond the boundaries of the site is provided in the SWMP. The assessment of effects, both mitigated and residual, is discussed in detail separately in the SWMP with a summary provided via Table 4 below.

Table 4: SWMP Summary of Effects

Assessment	Results
TKE Site Only Post Development Peak Flows	Post development peak flows are mitigated back to 80% of pre-development (no CC) of the 2- & 10-Year ARI events and 100% of pre-development (no CC) of the 100-Year ARI peak flows. Wetlands also provide 24-hour release of the EDV.
TKE + External Upstream Catchments Post Development Peak Flows	Post development peak flows are mitigated back to 100% of pre-development (CC) of the 2-,10- & 100-Year ARI peak flows
Downstream flood levels (10 and 100-year ARI)	Post developed water levels are all lower than pre-development levels including at all hydraulic restrictions i.e. Horotiu Road and Crawford Road bridge.

WRC managed drains - Level of Service (10-year drawdown)	Drawdown increases of 9.5 hours (Horotiu Road) and 2.7 hours (between TKE and Horotiu Road).
Volume neutrality	Neutrality for the post-development increase in 2-Year ARI volume not achieved. Circa 44,000 m ³ and 4,000 m ³ additional volume discharged to Mangaheka & Te Rapa catchments respectively.
Flooding – TKE Internal	There is no flooding within the proposed Lots. Velocity within the roading corridors comply with the maximum velocity limit per RITS. Depths within the roading corridors are broadly complying with the maximum depth limit, except at the roading low points. H1 Hazard Vulnerability (the lowest hazard) is achieved throughout TKE except at two locations that align with the increased depth at the roading low points.

5.3.2 Flood Risk to the Development

Commercial and industrial building platforms will be required to provide appropriate freeboard above the 100-Year ARI design flood level per the Te Miro flood model. The BoD sets a minimum 300mm freeboard requirement in accordance with RITS v2 for commercial and industrial building platforms. A consent notice over the proposed Lots will be applied to ensure that this requirement is followed through in future built form.

Final building floor levels will be confirmed at building consent or future development approval stage, once lot-specific building footprints, finished yard levels, internal drainage and overland flow arrangements are known.

The design of the proposed roading corridors aims to safely convey overland flow towards designated low points adjacent to a public conveyance channel. The design also aims to ensure flood hazards within the proposed roading corridors are within acceptable limits per RITS v2, which requires flood waters to not exceed:

- A maximum velocity limit – Less than or equal to 1m/s
- A maximum depth limit – 150mm depth, 1.5m from kerb

Supplementary to the criteria above, an assessment against Guideline 7-3 from the Australian Institute for Disaster Resilience has also been undertaken. The Te Miro flood model results show that the safest H1 Hazard Vulnerability classification, which is considered generally safe for vehicles, people and buildings, is achieved throughout the proposed Lots and roading corridors.

Flood model mapping results provided in Figure 22 to Figure 24 show that:

- Velocity within the roading corridors complies with the maximum velocity limit
- Depths within the roading corridors are broadly complying with the maximum depth limit, except at the roading low points (where flood waters are intended to spill into a public conveyance channel or wetland).

- H1 Hazard Vulnerability (the lowest hazard) is achieved throughout TKE except at two locations that align with the increased depth at the roading low points.

The flooding depth at the roading sags are a departure from RITs V2 is discussed in detail further in Section 5.4.1 below.

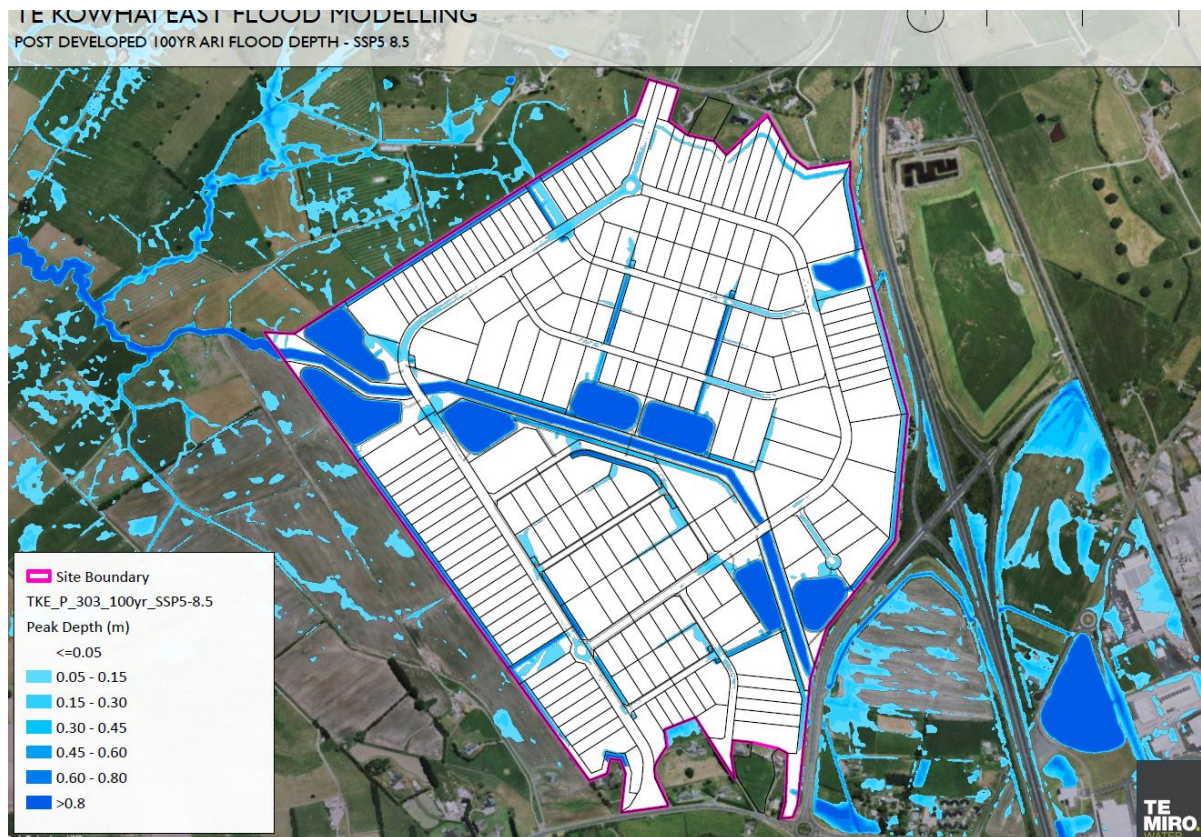


Figure 22: TKE Flood Depth Map (Source: TMW Model Build Report)

TE KOWHAI EAST FLOOD MODELLING
POST DEVELOPED 100YR ARI FLOOD VELOCITY - SSP5 8.5



Figure 23: TKE Flood Velocity Map (Source: TMW Model Build Report)

TE KOWHAI EAST FLOOD MODELLING
POST DEVELOPED 100YR ARI HAZARD - SSP5 8.5

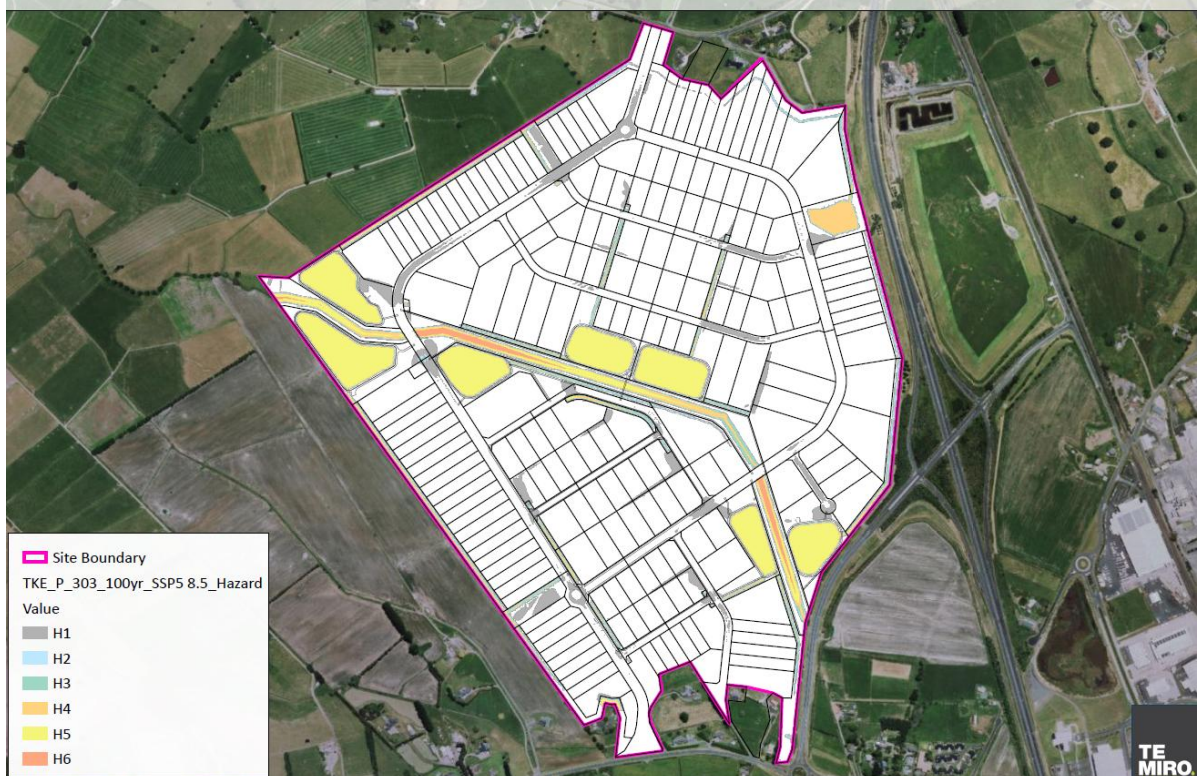


Figure 24: TKE Flood Hazard Vulnerability Map (Source: TMW Model Build Report)

5.4 ENGINEERING EXCEPTIONS AND DEPARTURES

Any departures from RITS v2, TR20, ICMP, LDMP or SWMP requirements are identified and assessed in this section.

Each departure includes:

- engineering justification;
- effects-based assessment under the FTAA framework;
- alternative solutions considered;
- mitigation measures; and
- stakeholder acceptance pathway.

5.4.1 Flow Depth Exceeding Limits at Roding Sags

Per section 5.3.2 above, the current Te Miro Water flood model indicates depths exceeding the 150mm limit (up to 400mm) at low points within the roading corridors. The flood model is conservative in its build as roading catchpits and their relative pipe networks have not been included in the flood model.

TKE will seek to comply with this requirement at detailed design and will investigate the alternative measures listed below:

- Advance the flood model by including catchpits and roading stormwater network;
- Provide sensitivity analysis assuming catchpits are fully blocked;
- Provide proprietary Megapits if required to alleviate ponding;
- Increase vertical curvature length of the roading corridor to increase the spill weir and minimise ponding depth;
- Mono camber transition in the carriageway and/or footpath berms to reduce the depth of spilling headwaters;
- Or a combination of the solutions above.

The detailed design will be required to demonstrate compliance with the applicable RITS flood-depth criterion, or an alternative design accepted through the engineering approval process, while maintaining the intended secondary-flow-path function.

6. WASTEWATER

6.1 WASTEWATER OVERVIEW

6.1.1 Existing Wastewater Network

The site is located outside the existing urban wastewater reticulation network. The site is currently rural zoned and is not serviced by a public wastewater network. The nearest existing wastewater infrastructure comprises a gravity local network of 150mm and 225mm diameter pipes along Te Kowhai Road, situated to the south/southeast of the development site. This network eventually discharges to the 1050mm diameter Far Western Interceptor (FWI) pipe on the western side of State Highway 1C.

6.1.2 Wastewater Design Basis and Servicing Strategy

The proposed wastewater network design is based on the relevant IAWAI requirements, WDC requirements, and the Waikato Regional Infrastructure Technical Specification.

The proposed wastewater servicing strategy is based on collecting wastewater from the development through an internal gravity reticulation network and conveying flows to a new wastewater pump station. The wastewater pump station will be delivered in Stage 1 so that the initial development stages can be serviced. The pump station will discharge flows via a rising main to the approved downstream wastewater network connection point. Wastewater infrastructure will be delivered progressively to support the staged development of the site. Internal gravity reticulation will be extended progressively with each stage of development. Details of the network are included in engineering drawings Section 500 contained in Appendix B.

Wastewater demand has been assessed based on light industrial use and discharge is to be limited to domestic strength only unless otherwise approved. Wet industries or other atypical users may require separate assessment and approval from IAWAI or the relevant wastewater authority at the time those activities are proposed. Where a future activity has the potential to generate trade waste, non-domestic wastewater, or flows exceeding those anticipated by the approved infrastructure strategy, the draft land use conditions require the consent holder to obtain all necessary approvals from the network operator and to demonstrate that any additional servicing requirements can be appropriately accommodated before the activity is established.

The strategic wastewater servicing approach is based on an interim servicing solution and an ultimate long-term solution. The interim solution is to discharge to the Far Western Interceptor / existing downstream wastewater network which has been agreed with IAWAI under a development agreement. The ultimate long-term solution is connection to the planned Northern Interceptor, with the current preferred alignment via the Arthur Porter Drive / Northern Interceptor Option A corridor with details also provided for within the agreement.

Consistent with the draft subdivision conditions, each stage will be supported by certification from a suitably qualified and experienced engineer that a wastewater servicing solution is available and capable of servicing the development anticipated within that stage, together with written confirmation from the network operator addressing available network and treatment capacity, any required upgrades, staging or connection approvals, and compatibility with the ultimate strategic servicing solution. The approved servicing solution for each stage will be incorporated into the detailed engineering plans for that stage.

6.2 WASTEWATER SERVICING

6.2.1 Internal Wastewater Reticulation

The internal wastewater network is to comprise a gravity pipe network located within public road and maintenance corridors. A separate wastewater connection will be provided to service each of the industrial lots, with no private wastewater drain passing between one lot and another unless protected by an easement. The pipe network is designed to be coordinated with road levels, lot levels, stormwater infrastructure, water supply, utilities and other services. The layout overview is depicted in Figure 25: below.

The internal gravity network is designed to provide compliant grades, cover, maintenance access, manhole spacing and capacity for the expected development flows. If wastewater pipes are required outside public road corridors, appropriate easements or other legal mechanisms will be provided to protect access, operation and maintenance, subject to approval.

The final layout, pipe sizes, grades, manhole locations, connection depths and easement requirements will be confirmed through detailed design and engineering approval.

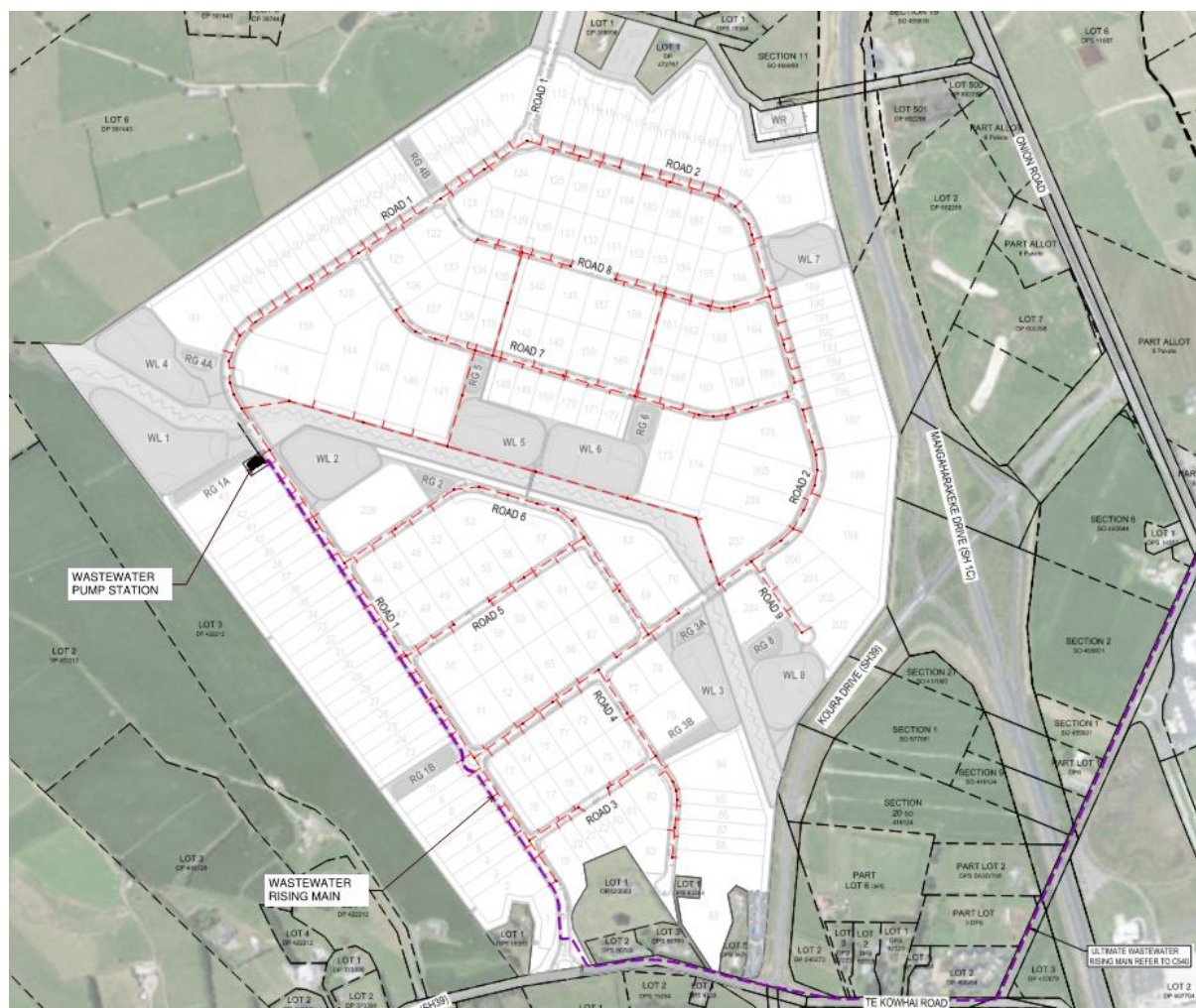


Figure 25: Wastewater Reticulation

6.2.2 Wastewater Pump Station

A new wastewater pump station will be delivered in Stage 1 to service the development. The pump station will receive wastewater flows from the internal gravity network and pump those flows to the approved downstream wastewater network. It will be sized and configured to support both the interim and ultimate servicing arrangements, with the ability to upgrade pumps as development demand increases. The rising main will be designed so that the transition between the interim and ultimate discharge locations can be managed safely and efficiently. Table 5 summarises the key preliminary design parameters for the pump station.

Table 5: Preliminary wastewater pump station design basis

Parameter	Preliminary design basis
Development area (net) served by gravity / pump station	125 ha

Parameter	Preliminary design basis
Population equivalent allowance	45 persons / ha
Total population equivalent	5,625 people
Average daily flow	1,125 m ³ /day (13.02 L/s)
Peak wet weather flow	87.24 L/s
Emergency storage	421.9 m ³ (9 hours of average daily flow)
Indicative pump station lot size	Approximately 30 m x 40 m

The above parameters provide a preliminary resource-consent level basis for the pump station and rising main. Final pump selection, wet well arrangement, emergency storage configuration, odour control, power supply, telemetry, geotechnical design, access, easements and asset handover requirements will be confirmed through detailed design, IAWAI engagement and engineering approval. The Aquate pump station design details and relevant calculations are in Appendix C.

6.2.3 Interim Wastewater Servicing

Wastewater servicing is supported by a staged and agreed infrastructure pathway between the developer and IAWAI. The initial development stages will be serviced through a controlled interim discharge to the Far Western Interceptor adjacent to State Highway 1C. The interim servicing capacity has been agreed with IAWAI and is recorded in the consent conditions. This provides a practical interim solution that enables early stages to proceed while maintaining the developer's commitment to the ultimate strategic wastewater infrastructure. The interim connection must be constructed, commissioned and operational before section 224(c) certification for Stage 1, and the rising main to the agreed connection point on the Northern Interceptor is required within 24 months of the infrastructure provider confirming the connection point, consistent with the development agreement.

6.2.4 Ultimate Wastewater Servicing

Ultimate wastewater servicing is proposed via rising main connection to the Northern Interceptor once constructed and commissioned by IAWAI, the timing of which is subject to the infrastructure provider's delivery programme but aligning with the overall development strategy. Interim servicing arrangements detailed within Section 6.2.3 enable development to proceed ahead of its availability.

6.3 OPERATION, MAINTENANCE AND OWNERSHIP

The proposed wastewater infrastructure will provide safe, practical, all-weather access for routine operation, inspection, maintenance and emergency response, including pump removal and future upgrades where required.

Maintenance access has been incorporated into the concept design and will be further refined with IAWAI during detailed design. The wastewater infrastructure is expected to be vested in, and ultimately owned, operated and maintained by, IAWAI.

Final ownership, operation and maintenance responsibilities will be confirmed through detailed design, IAWAI requirements, development agreements, engineering approval and asset handover processes.

6.4 ENGINEERING EXCEPTIONS AND DEPARTURES

No specific wastewater engineering exceptions have been identified.

Final pipe sizes, grades, pump station and rising main configuration, and asset handover requirements will be confirmed through detailed design, modelling, IAWAI engagement and engineering approval.

7. WATER SUPPLY

7.1 WATER SUPPLY OVERVIEW

7.1.1 Existing Water Supply Network

The site is located outside the existing urban water supply reticulation network and outside the Hamilton City Council territorial boundary. The site is currently rural in character, with the only existing public water supply infrastructure in the vicinity comprising a 100 mm uPVC watermain within Te Kowhai Road adjacent to the site, together with existing out-of-district supply connections. This infrastructure was not designed to service large-scale industrial development.

The site was not included within the growth assumptions underpinning HCC's Water Master Plan V4, and no long-term servicing solution was identified for the development. The wider strategic water supply solution for the northern growth area is the future delivery of a second municipal water treatment plant (Northern WTP), associated trunk water infrastructure, storage reservoirs and strategic bulk watermains. Delivery of that infrastructure sits with the network operator and is not relied upon to service the development.

Accordingly, the proposed water supply strategy has been developed as a staged approach that enables development to proceed ahead of the ultimate strategic network while maintaining an orderly transition to the long-term public water supply solution.

7.1.2 Water Supply Design Basis and Servicing Strategy

The water supply network design is based on relevant IAWAI requirements, Waikato District Council requirements, Hamilton City Council requirements where connection to Hamilton infrastructure is required, RITS, Fire and Emergency New Zealand requirements, and will continue to be refined through detailed design.

Water demands adopted for hydraulic modelling have been calculated in accordance with the RITS industrial demand assumptions, with FW3 firefighting capability required across all serviced areas at each stage of development.

The water supply strategy assumes typical industrial water demand appropriate for the proposed land use. Where a future activity has water demand characteristics materially exceeding RITS industrial allowances, site-specific assessment and network upgrades may be required before connection to the public water supply network.

The proposed water supply servicing strategy is based on providing a new internal reticulation network connected to an approved external bulk water supply source.

The servicing strategy comprises two components:

- Interim water supply arrangements to enable staged development in advance of the ultimate strategic infrastructure; and

- Ultimate water supply infrastructure comprising a strategic reservoir and bulk main connection to service the full development.

The interim arrangement supports the initial development stages, with the confirmed interim capacity to be established through hydraulic modelling currently being undertaken with Hamilton City Council and IAWAI. The ultimate infrastructure will be required by the trigger set out in the proposed consent conditions, or earlier if the confirmed interim capacity is exceeded. Water supply infrastructure is expected to be provided progressively in response to development demand, rather than through a single upfront investment, enabling an orderly transition from interim servicing to the ultimate solution while maintaining appropriate levels of service throughout the development. Details of the internal network and proposed servicing feed alignments are included in the C600 Water Supply drawings in Appendix B.

7.2 WATER SUPPLY NETWORK AND SERVICING

7.2.1 Internal Water Supply Network and Demand

The internal water supply network is to comprise public watermains located within public road corridors. Watermains are designed to provide potable water supply, firefighting capability and operational flexibility across the development.

The internal network has been designed to provide firefighting supply in accordance with SNZ PAS 4509 and Fire and Emergency New Zealand requirements. Maven's hydraulic modelling demonstrates that the proposed reticulation can achieve FW3 requirements where the required supply pressure is available. The internal water supply network includes fire hydrants located to support firefighting access across the development.

Where future industrial users require firefighting demand beyond the standard network allowance, additional site-specific fire engineering measures, private storage, sprinkler supply arrangements, pumps or other mitigation may be required at the time of building consent or future development approval.

Based on Maven's internal hydraulic modelling undertaken for approximately 125 ha of net proposed development area, a minimum residual pressure of 250 kPa is required at the point of supply to achieve the required level of service across the internal water supply network. At this supply pressure, the proposed internal reticulation is capable of meeting the FW3 firefighting water supply requirements.

The hydraulic modelling and pressure requirements have been determined using an EPANET model, with the modelling methodology, assumptions and detailed results presented in the EPANET Report included in Appendix H. Accordingly, no servicing constraints are anticipated for the ultimate development, provided the planned future infrastructure is delivered. The modelling provides an interim assessment of the feasibility and will be updated throughout detailed infrastructure design to ensure ongoing conformance.

7.2.2 Interim Water Supply Servicing

The interim water supply arrangement is a connection to the existing water infrastructure at Ruffell Road / Arthur Porter Drive. This enables staged development in advance of the strategic solution. Council has advised that some upgrades to the existing external network may be required around the Ruffell Road connection, including replacement of a smaller section of reticulation with larger capacity

pipework to support the interim model. Hydraulic modelling has been commissioned by Hamilton City Council based on an agreed modelling scope established in May 2026, assessed against the 2041 and 2061 planning horizons and incorporating strategic network upgrades. The confirmed interim servicing capacity, and the scope of the associated external upgrades, will be established through that modelling and confirmed with Hamilton City Council and IAWAI prior to engineering approval.

The interim connection must be established before section 224(c) certification for Stage 1, and the strategic reservoir and filling main are required by 1 November 2033, consistent with the development agreement, or the earlier exceedance of the confirmed interim capacity.

7.2.3 Ultimate Water Supply Servicing

The strategic water supply solution for the development is a reservoir with a feed line to the existing bulk main on Rotokauri Road via Arthur Porter Drive. This infrastructure provides the long-term servicing arrangement for the site, servicing the development's water demand from existing Council water infrastructure.

Strategic network modelling undertaken for Hamilton City Council and IAWAI has confirmed the existing network, including the high lift pump station and bulk ring main, can fill a reservoir located on TKE development land. A 250 mm internal diameter filling main meets the network operator's headloss requirements. The proposed reservoir platform (finished ground level RL 40.0) sits within the development on land controlled by the developer, with a reservoir sized for the demand. Reservoir location and elevation sensitivity, and the split between the attributable infrastructure and wider catchment requirements, are being confirmed through the strategic scenario modelling with IAWAI.

The timing for delivery of the strategic reservoir, feed line and any associated pumping, pressure management and connection infrastructure is addressed through the proposed consent conditions, which require this infrastructure to be constructed, commissioned and operational by the date specified in those conditions, or upon the earlier exceedance of the confirmed interim servicing capacity.

7.3 OPERATION, MAINTENANCE AND OWNERSHIP

The water supply infrastructure will be designed to provide safe and practical maintenance access. The public water supply infrastructure is expected to be ultimately owned and operated by IAWAI. Final ownership, operation and maintenance responsibilities will be confirmed through detailed design, IAWAI requirements, Hamilton City Council requirements where relevant, development agreements, engineering approval and asset handover processes.

7.4 ENGINEERING EXCEPTIONS AND DEPARTURES

No specific water supply engineering exceptions have been identified that would prevent the development from being serviced. The current strategy is considered a feasible, compliant pathway for water supply servicing of the development, subject to final hydraulic modelling.

Final pipe sizes, pressure zones, storage requirements, hydrant coverage, firefighting flows, network resilience, metering, backflow prevention, operational controls and asset handover requirements will be confirmed through detailed water supply design, hydraulic modelling, IAWAI engagement and engineering approval.

8. RETAINING WALLS AND STRUCTURES

8.1 RETAINING WALLS

In the northern portion of the site, a retaining wall is required to support the downcutting of the rolling hill landform. As set out in the Geotechnical Investigation Report (GIR) prepared by CMW Geosciences, the excavation of the hill slopes will necessitate retained heights of up to approximately 6m, with a 1V:3H maximum graded slope above and a 10m minimum buffer to the site boundary. Several retaining solutions are currently under consideration, with preliminary options including a cantilever pole wall and a segmental block retaining wall system. Further details are included within the GIR.

8.2 BRIDGES

The two road crossings over the Mangaheka Stream will each require a bridge approximately 20m in length. The bridge spans provide for a spillway between the attenuation embankments, allowing for emergency flows in the event of storm events exceeding the 100-year event or a 100% culvert blockage scenario.

An additional foot/cycle bridge is provided at approximately the mid-length of the Mangaheka Stream corridor to provide connectivity of the pedestrian and cycle networks. Bridge forms, spans, freeboard and levels will be confirmed at detailed design stage and will comply with the New Zealand Building Code and other relevant standards.

9. UTILITY SERVICES

9.1 OVERVIEW

The development will require power and telecommunications infrastructure to service the proposed industrial lots and associated infrastructure. Conditional acceptance has been provided by both Tuatahi First Fibre and WEL Networks.

9.2 POWER SUPPLY

Power supply will be provided to service industrial lots, non-industrial precinct, the local Neighbourhood Centre, street lighting, wastewater pump station infrastructure, stormwater assets and other public infrastructure.

Correspondence has been received confirming that electrical reticulation can be supplied to the development. The current advice identifies available capacity of approximately 9 MVA, sufficient to service approximately 70–75 hectares of light industrial land, depending on individual customer load requirements.

The remaining development load is expected to be supplied from the planned New Exelby zone substation, which is currently planned for installation by 2033. Alternatively, this infrastructure may be brought forward as development progresses, subject to network planning, development uptake and electricity network provider requirements.

9.3 TELECOMMUNICATIONS

Tuatahi First Fibre has provided acceptance as telecommunications operator for the development. Telecommunications connections are expected to be made available to each site in the development, subject to the developer entering into the required Tuatahi First Fibre Installation Agreement.

10. RECOMMENDATIONS

The following recommendations summarise the matters that should be carried through into consent conditions for the development, to be read alongside the detailed assessment in each section above.

1. **Engineering Design Flexibility:** minor amendments to engineering design, infrastructure alignments, dimensions, levels and construction methodology should be permitted through the engineering approval process, provided the assessed environmental effects are not materially increased and the infrastructure performance outcomes identified within this report continue to be achieved.
2. **Integrated Detailed Design:** detailed engineering design should continue to be undertaken in an integrated manner across civil, stormwater, transport, ecology, hydrogeology, geotechnical and urban design disciplines to ensure that modifications within one discipline do not adversely affect the performance outcomes of another.
3. **Earthworks and Development Staging:** earthworks, subdivision and development staging/sequencing should remain flexible to respond to market demand, construction sequencing and detailed engineering design, provided each stage can demonstrate it is independently serviceable and infrastructure performance outcomes are maintained.
4. **Future Earthworks and Ground Level Changes:** at the time of subdivision construction and at individual lot development stage, ground levels and earthworks extents may be refined and retaining walls may be included without requiring additional land use consent where they remain generally in accordance with the approved development, provided any amendments demonstrate that flood behaviour, overland flow paths, infrastructure performance and geotechnical outcomes are not adversely affected.
5. **Roading and Streetscape Flexibility:** streetscape elements including lighting, landscaping, street trees, parking bays, vehicle crossings, bus stops, street furniture and minor road geometry refinements may be amended during detailed engineering design, subject to approval by the relevant road controlling authority and consistency with the approved transport and urban design outcomes.
6. **Stormwater Toolbox Approach:** at the time of lot development, sites shall comply with the on-lot requirements outlined in the stormwater toolbox of the Stormwater Management Plan. Where an alternative approach is proposed, a suitably qualified professional shall demonstrate at building consent stage that the alternative achieves the required stormwater performance outcomes.
7. **Stormwater Performance Verification:** prior to each earthworks or development stage, updated flood modelling shall demonstrate that the completed and proposed development continues to achieve the approved flood management and stormwater performance outcomes, including downstream flood levels, attenuation performance, flood storage, overland flow path functionality and level of service.
8. **Overland Flow Path Protection:** principal overland flow paths should be protected through public reserve, easements or other appropriate legal mechanisms; secondary lot-specific overland flow paths may be refined through future subdivision or engineering design, provided equivalent or improved hydraulic performance is demonstrated and adequate protection mechanisms provided.
9. **Lot Development Flood Assessment:** where finished ground levels, building platforms or site layout are altered during future engineering or building consent stages, an appropriately qualified engineer should demonstrate that the development maintains the approved flood management outcomes, including overland flow path functionality, flood conveyance, building flood resilience and any required freeboard.

10. **Residual Flooding Effects:** The residual effects as a result of increased runoff volumes identified within Te Miro Water's flooding assessment shall be mitigated through one or a combination of the following:
 - Site wide stormwater management measures within public devices,
 - On lot stormwater management measures secured through consent conditions and/or consent notices,
 - A financial contribution to the relevant Regional or District authority to fund erosion protection, watercourse resilience, or other agreed offset works that mitigate the residual downstream effects.
11. **Lot 89 Stormwater Discharge:** future stormwater discharge generated within the Lot 89 peninsula shall be redirected to the proposed network within Catchment 3. Where this is proven to be impractical, discharge into the Te Otamanui catchment is acceptable provided that on-Lot attenuation of the 2-Year, 10-Year and 100-Year ARI peak flows to pre-development levels are provided and an assessment of any other stormwater related effects. Any stormwater discharge to the Te Otamanui Drain would need to be applied for under a future consent.
12. **Maximum Permissible Impervious:** The maximum permissible impervious areas are to be as defined on Maven Drawing C485. I.e. 90% imperviousness for the internal lots and 100% for the perimeter ones where adjacent to a conveyance channel and planting buffer.
13. **External Infrastructure Servicing Triggers:** where a development stage relies on an upgrade to the external transport or three waters network, that upgrade should be operational before the stage is certified. Strategic infrastructure should be required by the date provided for in the consent conditions or upon the earlier exceedance of the confirmed interim servicing capacity, whichever occurs first.
14. **Wastewater Capacity:** prior to each development stage, confirmation should be provided that adequate interim and/or ultimate wastewater servicing capacity is available for the proposed stage, supported by updated network modelling where required by the infrastructure provider. Interim servicing arrangements should be operational before the first stage relying on them is certified, and development serviced through those arrangements should remain within the confirmed interim capacity.
15. **Water Supply Capacity:** prior to each development stage, confirmation should be provided that adequate interim and/or ultimate water supply capacity, including FW3 firefighting capability, is available for the proposed stage, supported by updated network modelling where required by the infrastructure provider. Interim servicing arrangements, including any associated external network upgrades, should be operational before the first stage relying on them is certified, with the strategic solution delivered in accordance with the servicing triggers recommended above.
16. **Infrastructure Easements:** appropriate easements should be established where required to protect infrastructure, maintenance access, overland flow paths and principal stormwater conveyance routes where not located within designated public corridors or reserves.
17. **Infrastructure Delivery:** each development stage should demonstrate that all infrastructure required to service that stage has either been constructed or its capacity demonstrated prior to the issue of titles or commencement of the associated land use.
18. **Operations and Maintenance:** Operation and Maintenance Plans should be prepared for all public stormwater assets prior to vesting or operation, including inspection, maintenance, sediment removal, vegetation management and access requirements.
19. **Adaptive Management:** the consent framework should provide for adaptive management throughout the development of Te Kōwhai East. Monitoring, updated modelling and performance

assessments undertaken during construction and completed development stages may be used to refine the detailed design of subsequent stages, provided the approved infrastructure performance outcomes and environmental effects are maintained or improved. Any material changes should be supported by appropriate engineering assessment and certification.

11. CONCLUSION

This Civil Infrastructure Report has assessed the civil infrastructure required to support the Te Kōwhai East Development. The assessment confirms that the development can be appropriately serviced by civil infrastructure, including earthworks, roading, stormwater management, overland flow and flood management, wastewater, water supply, power and telecommunications.

Bulk earthworks will establish suitable industrial development platforms, road corridors, stormwater reserves and infrastructure corridors, responding to existing ground conditions, shallow groundwater, geotechnical constraints and staging.

The proposed roading network provides safe and efficient access via a primary connection to Te Kowhai Road / SH39 and a secondary connection to Onion Road, and is designed to support industrial vehicle movements, walking and cycling, future public transport opportunities and utility servicing.

The stormwater strategy provides an integrated treatment, conveyance, attenuation and flood management approach, with overland flow and flood management addressed through the proposed landform, dedicated conveyance channels, stormwater reserves, wetlands and the enhanced Mangaheka Stream corridor. Hydraulic modelling demonstrates that peak flow and flood effects can be appropriately managed, with the residual effects associated with increased runoff volume addressed through the adaptive management framework set out in the SWMP and proposed consent conditions.

Flood effects are assessed as acceptable within the FTAA framework, with no worsening of downstream flood risk within LDMP-managed systems, subject to final confirmation through detailed flood modelling. Effects on the Mangaheka Stream corridor will be managed through the Streamworks Management Plan, with ecological effects addressed in the ecological assessments supporting the application.

Wastewater servicing will be provided through a new internal gravity network and Stage 1 pump station. An interim disposal solution has been established via a controlled discharge to the Far Western Interceptor, in accordance with the staged servicing pathway agreed with the network operator and recorded in the proposed consent conditions, enabling the initial development stages to proceed. The design and reporting have also provided for the planned ultimate solution, allowing a transition to the future connection to the planned Northern Interceptor.

Water supply servicing will be provided through a staged public servicing approach, with interim network connections supporting development in advance of the ultimate strategic solution comprising a reservoir and bulk main connection servicing the development. The available interim capacity and associated external network upgrades will be confirmed through Hamilton City Council and IAWAI network modelling. For both networks, each development stage will proceed only where adequate capacity, including firefighting supply, has been confirmed and the external infrastructure that stage relies on is operational, in accordance with the servicing triggers set out in the proposed consent conditions.

Power and telecommunications services can be provided, with electrical supply available for the initial development area and further supply through planned network upgrades as development progresses.

Construction-phase effects — principally sediment discharge, dust and erosion associated with bulk earthworks, streamworks and service trenching — will be avoided, remedied or mitigated through erosion and sediment control measures developed in accordance with Waikato Regional Council requirements, the Earthworks Management Plan and Streamworks Management Plan, staged construction and progressive stabilisation. Groundwater effects, including temporary dewatering, drawdown and settlement risk, have been assessed in the Hydrogeological Assessment and will be managed through detailed design and construction management controls.

Overall, the actual and potential effects associated with the proposed civil infrastructure can be appropriately avoided, remedied or mitigated through the design measures, management plans, staging controls and consent conditions identified in this report.

APPENDICES

APPENDIX A – SCHEME PLANS (APPENDED SEPARATELY)

APPENDIX B – ENGINEERING DESIGN PLANS (APPENDED SEPARATELY)

APPENDIX C – ENGINEERING CALCULATIONS

APPENDIX D – EARTHWORKS MANAGEMENT PLAN (EWMP) (APPENDED SEPARATELY)

APPENDIX E – STORMWATER MANAGEMENT PLAN (SWMP) (APPENDED SEPARATELY)

APPENDIX F – STREAM WORKS MANAGEMENT PLAN (STMP) (APPENDED SEPARATELY)

APPENDIX G – SERVICE PROVIDER CORRESPONDENCE

APPENDIX H – WATER SUPPLY MODELLING MEMO

APPENDIX C – ENGINEERING CALCULATIONS

APPENDIX C.1 – SW RAINFALL PARAMETERS

**MAVEN ASSOCIATES**

Job Number

Sheet

Rev

180010

1

A

Job Title

Te Kowhai East

Author

Date

Checked

Calc Title

Design Rainfall - Summary

KY

3-Nov

RW

STEP 1: PARAMETERS**Historical Rainfall Data via HIRDS NIWA**

Historical Depth (mm)				
	2yr	5Yr	10Yr	100Yr
24hr	63.2	81.9	96	148
48hr	76.3	98.8	116	178
72hr	84.5	109	128	196

Historical Intensity (mm/hr)				
	2yr	5Yr	10Yr	100Yr
10min	59.1	77.3	91.1	143.0

% Increase per °C Increase per RITS				
	2Yr	5Yr	10Yr	100Yr
1hr	12.2	12.8	13.1	13.6
24hr	7.2	7.8	8.1	8.6
48hr	6.1	6.7	7	7.5
72hr	5.5	6.2	6.5	6.9

STEP 2: ADJUSTED RAINFALL

RCP 6.0 Adjusted Depth (mm)				
	2yr		10Yr	100Yr
24hr				
48hr				
72hr				

RCP 6.0 Adjusted Intensity (mm/hr)				
	2yr		10Yr	100Yr
10min				

SSP5-8.5 Adjusted Depth (mm)				
	2yr	5Yr	10Yr	100Yr
24hr	81.4	107.5	127.1	198.9
48hr	94.9	125.3	148.5	231.4
72hr	103.1	136.0	161.3	250.1

SSP5-8.5 Adjusted Intensity (mm/hr)				
	2yr	5Yr	10Yr	100Yr
10min	87.9	116.9	138.8	220.8

APPENDIX C.2 – SW RAINGARDENS

	Sheet 1	Rev A	Job Title TE KOWHAI EAST Calc Title RAINGARDEN SUMMARY	Author AZ	Checked RW	Date 2/03/2026
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Catchment Number	Raingarden	Road Area (ha)	WQV (m ³)	EDV (m ³)	Rain Garden Depth (m)	Retention Volume (m ³)	Storage depth under drainage layer (m)	Rain Garden Min. Area (m ²)	Rain Garden Design Area (m ²)	Check Design > Minimum
1	1A	1.3949	266	1218	1.45	126	0.30	1,196	1,946	OK
	1B	3.9296	749		1.55	354	0.40	2,526	2,863	OK
2	2	1.6348	312	374	1.55	147	0.40	1,051	1,115	OK
3	3A	1.3492	257	619	1.45	121	0.30	1,156	1,185	OK
	3B	1.3541	258		1.55	122	0.40	870	930	OK
4	4A	1.3973	266	922	1.45	126	0.30	1,198	1,424	OK
	4B	2.6329	502		1.65	237	0.50	1,354	1,380	OK
5	5	1.361	259	311	1.45	122	0.30	1,167	1,212	OK
6	6	3.3952	647	777	1.55	306	0.40	2,183	2,322	OK
8	8	1.7027	325	390	1.45	153	0.30	1,459	2,146	OK

		MAVEN ASSOCIATES	Job Number 180010	Sheet 1	Rev A	Job Title Calc Title	TE KOWHAI EAST RAINGARDEN 1A	Author AZ	Checked KY	Date 8/05/2026
TR20-07 Bioretention Device Sizing Criteria			Design parameters			RAIN GARDEN E 1				
Ponding depth	200-300 mm	200mm @ 100% Void space				0.30	(m)			
Mulch layer	50-100mm	50mm @ 100% Void space				0.05	(m)			
Bioretention filter media	≥ 500-1000mm	500mm @ 30% Void space				0.50	(m)			
Transition layer	100mm	100mm @ 30% Void space				0.10	(m)			
Drainage layer	≥ 200-300mm	300mm @ 35% Void space				0.20	(m)			
Storage beneath under-drain layer	≥300mm	300mm @ 35% Void space				0.30	(m)			
Total Depth						1.45	(m)			
Required detention volume = Pervious water quality volume +Impervious water quality volume										
Bioretention device design										
Water Quality Volume=										
Live volume of storage needed										
Required surface area (m ²) $A_s=(WQV)(d)/k(h+d)t$										
		265.96		m ³						
V _{live} =		106.38419		m ³						
		301.42186		m ²						
WQV=		265.96046		m3						
d=		0.85		m						
k=		0.75		m/day						
h=		0.15		m						
t=		1		day						
Live Storage Available=Surface area × Maximum water depth										
V _{WQV} =		90.43		m ³						
Difference between Live storage needed and available:										
		-15.96		m ³						
Area Short Fall:										
		53.19		m ²						
Bioretention live storage surface area :										
		354.61		m ²		Minimum Device Sizing based on Live Storage				
Retention Volume (where 10mm over 90% Impervious Road)										
V _{Retention} =		125.54		m ³		Minimum Device Sizing based on Retention Volume				
Min Area based on Initial Abstraction:										
		1195.6286		m ²						
Min. Area base on Live Storage:										
		355		m ²						
Rain Garden Sizing:										
		1195.63		m ²						

A schematic of a typical cross section of a raingarden is shown in Figure 8-11 below.

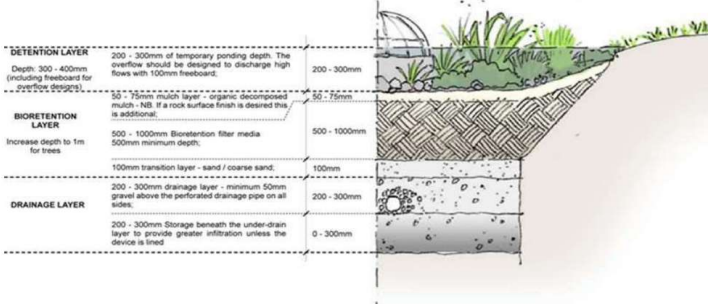


Figure 8-11: Schematic of a rain garden cross section¹⁰⁷

A schematic of a typical cross section of a raingarden is shown in Figure 8-11 below.

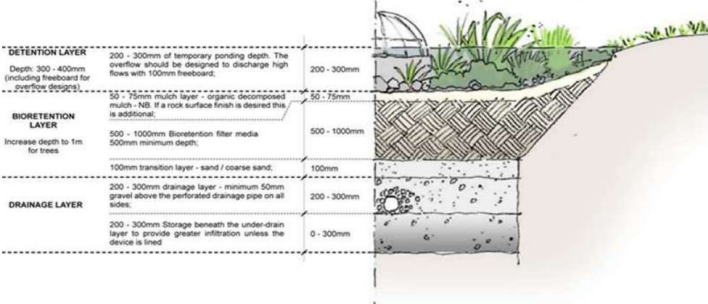


Figure 8-11: Schematic of a rain garden cross section¹⁰⁷

		MAVEN ASSOCIATES	Job Number 180010	Sheet 1	Rev A	Job Title Calc Title	TE KOWHAI EAST RAINGARDEN 1B	Author AZ	Checked KY	Date 8/05/2026
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TR20-07 Bioretention Device Sizing Criteria		Design parameters		RAIN GARDEN E 1	
Ponding depth	200-300 mm	200mm @ 100% Void space		0.30	(m)
Mulch layer	50-100mm	50mm @ 100% Void space		0.05	(m)
Bioretention filter media	≥ 500-1000mm	500mm @ 30% Void space		0.50	(m)
Transition layer	100mm	100mm @ 30% Void space		0.10	(m)
Drainage layer	≥ 200-300mm	300mm @ 35% Void space		0.20	(m)
Storage beneath under-drain layer	≥300mm	300mm @ 35% Void space		0.40	(m)
				Total Depth	1.55 (m)

Required detention volume = Pervious water quality volume +Impervious water quality volume

Bioretention device design

Water Quality Volume=

Live volume of storage needed V_{live}

Required surface area (m^2) $A_s=(WQV)(d)/k(h+d)t$

749.24

m^3

$V_{live} =$

299.69696

m^3

849.1414

m^2

WQV=

749.24241

m^3

d=

0.85

m

k=

0.75

m/day

h=

0.15

m

t=

1

day

Live Storage Available=Surface area × Maximum water depth

254.74

m^3

Difference between Live storage needed and available:

Area Short Fall:

Bioretention live storage surface area :

Retention Volume (where 10mm over 90% Impervious Road)

353.66

m^3

Min Area based on Initial Abstraction:

Min. Area base on Live Storage:

Rain Garden Sizing:

2526.17

m^2

A schematic of a typical cross section of a raingarden is shown in Figure 8-11 below.

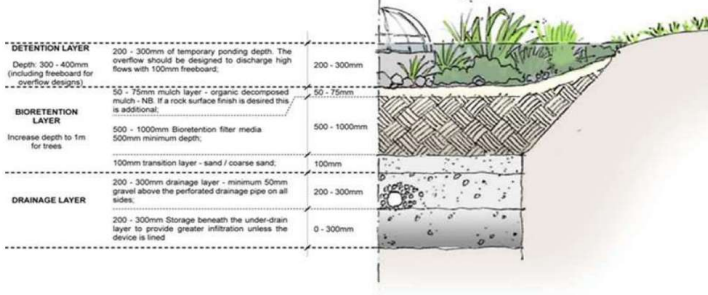


Figure 8-11: Schematic of a rain garden cross section¹⁰⁷

Minimum Device Sizing based on Live Storage

Minimum Device Sizing based on Retention Volume

		MAVEN ASSOCIATES	Job Number 180010	Sheet 1	Rev A	Job Title Calc Title	TE KOWHAI EAST RAINGARDEN 2	Author AZ	Checked KY	Date 8/05/2026
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TR20-07 Bioretention Device Sizing Criteria		Design parameters		RAIN GARDEN E 1	
Ponding depth	200-300 mm	200mm @ 100% Void space		0.30	(m)
Mulch layer	50-100mm	50mm @ 100% Void space		0.05	(m)
Bioretention filter media	≥ 500-1000mm	500mm @ 30% Void space		0.50	(m)
Transition layer	100mm	100mm @ 30% Void space		0.10	(m)
Drainage layer	≥ 200-300mm	300mm @ 35% Void space		0.20	(m)
Storage beneath under-drain layer	≥300mm	300mm @ 35% Void space		0.40	(m)
Total Depth				1.55	(m)

Required detention volume = Pervious water quality volume +Impervious water quality volume

Bioretention device design

Water Quality Volume=

Live volume of storage needed V_{live}

Required surface area (m^2) $A_s=(WQV)(d)/(k(h+d)t)$

WQV=

d=

k=

h=

t=

Live Storage Available=Surface area × Maximum water depth

Difference between Live storage needed and available:

Area Short Fall:

Bioretention live storage surface area :

Retention Volume (where 10mm over 90% Impervious Road)

Min Area based on Initial Abstraction:

Min. Area base on Live Storage:

Rain Garden Sizing:

V_{live} =	311.70	m^3
	124.68053	m^3
	353.26149	m^2
WQV=	311.70132	m^3
d=	0.85	m
k=	0.75	m/day
h=	0.15	m
t=	1	day
V_{WQV} =	105.98	m^3
	147.13	m^3
	1050.94	m^2

A schematic of a typical cross section of a raingarden is shown in Figure 8-11 below.

Figure 8-11: Schematic of a rain garden cross section¹⁰⁷

	MAVEN ASSOCIATES	Job Number 180010	Sheet 1	Rev A	Job Title Calc Title	TE KOWHAI EAST RAINGARDEN 3A	Author AZ	Checked KY	Date 8/05/2026
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TR20-07 Bioretention Device Sizing Criteria		Design parameters		RAIN GARDEN E 1	
Ponding depth	200-300 mm	200mm @ 100% Void space		0.30	(m)
Mulch layer	50-100mm	50mm @ 100% Void space		0.05	(m)
Bioretention filter media	≥ 500-1000mm	500mm @ 30% Void space		0.50	(m)
Transition layer	100mm	100mm @ 30% Void space		0.10	(m)
Drainage layer	≥ 200-300mm	300mm @ 35% Void space		0.20	(m)
Storage beneath under-drain layer	≥ 300mm	300mm @ 35% Void space		0.30	(m)
				Total Depth	1.45 (m)

Required detention volume = Pervious water quality volume +Impervious water quality volume

Bioretention device design

Water Quality Volume=

Live volume of storage needed V_{live}

Required surface area (m^2) $A_s=(WQV)(d)/k(h+d)t$

	257.25	m^3
$V_{live} =$	102.8988	m^3
	291.54661	m^2
WQV=	257.24701	m^3
d=	0.85	m
k=	0.75	m/day
h=	0.15	m
t=	1	day

Live Storage Available=Surface area × Maximum water depth

Difference between Live storage needed and available:

Area Short Fall:

Bioretention live storage surface area :

Retention Volume (where 10mm over 90% Impervious Road)

Min Area based on Initial Abstraction:

Min. Area base on Live Storage:

Rain Garden Sizing:

$V_{WQV} =$	87.46	m^3
	-15.43	m^3
	51.45	
	343.00	m^2
$V_{Retention} =$	121.43	m^3
	1156.4571	m^2
	343	m^2
	1156.46	m^2

A schematic of a typical cross section of a raingarden is shown in Figure 8-11 below.

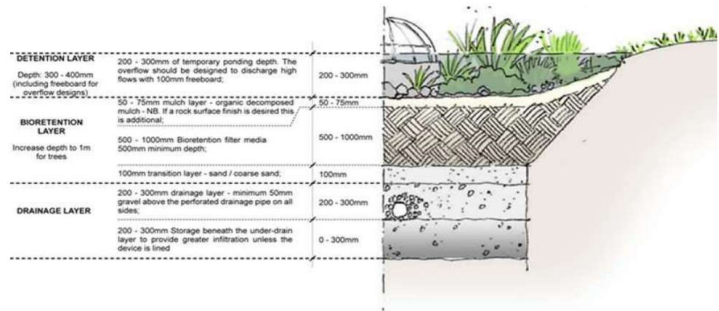


Figure 8-11: Schematic of a rain garden cross section¹⁰⁷

	MAVEN ASSOCIATES	Job Number 180010	Sheet 1	Rev A	Job Title Calc Title	TE KOWHAI EAST RAINGARDEN 3B	Author AZ	Checked KY	Date 8/05/2026
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TR20-07 Bioretention Device Sizing Criteria		Design parameters		RAIN GARDEN E 1	
Ponding depth	200-300 mm	200mm @ 100% Void space		0.30	(m)
Mulch layer	50-100mm	50mm @ 100% Void space		0.05	(m)
Bioretention filter media	≥ 500-1000mm	500mm @ 30% Void space		0.50	(m)
Transition layer	100mm	100mm @ 30% Void space		0.10	(m)
Drainage layer	≥ 200-300mm	300mm @ 35% Void space		0.20	(m)
Storage beneath under-drain layer	≥ 300mm	300mm @ 35% Void space		0.40	(m)
Total Depth				1.55	(m)

Required detention volume = Pervious water quality volume +Impervious water quality volume

Bioretention device design

Water Quality Volume=

Live volume of storage needed V_{live}

Required surface area (m^2) $A_s=(WQV)(d)/k(h+d)t$

$V_{live} = 103.27251 \text{ m}^3$

$WQV = 258.18128 \text{ m}^3$

$d = 0.85 \text{ m}$

$k = 0.75 \text{ m/day}$

$h = 0.15 \text{ m}$

$t = 1 \text{ day}$

$V_{WQV} = 87.78 \text{ m}^3$

Live Storage Available=Surface area × Maximum water depth

Difference between Live storage needed and available: -15.49 m³

Area Short Fall: 51.64

Bioretention live storage surface area : 344.24 m²

Retention Volume (where 10mm over 90% Impervious Road) $V_{Retention} = 121.87 \text{ m}^3$

Min Area based on Initial Abstraction: 870.49286 m²

Min. Area base on Live Storage: 344 m²

Rain Garden Sizing: 870.49 m²

A schematic of a typical cross section of a raingarden is shown in Figure 8-11 below.

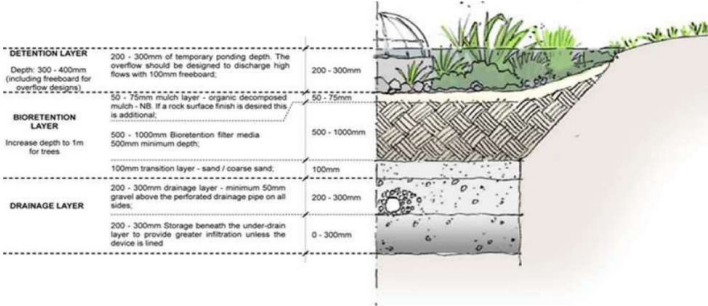


Figure 8-11: Schematic of a rain garden cross section¹⁰⁷

		MAVEN ASSOCIATES	Job Number 180010	Sheet 1	Rev A	Job Title Calc Title	TE KOWHAI EAST RAINGARDEN 4A	Author AZ	Checked KY	Date 8/05/2026
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TR20-07 Bioretention Device Sizing Criteria		Design parameters		RAIN GARDEN E 1	
Ponding depth	200-300 mm	200mm @ 100% Void space		0.30	(m)
Mulch layer	50-100mm	50mm @ 100% Void space		0.05	(m)
Bioretention filter media	≥ 500-1000mm	500mm @ 30% Void space		0.50	(m)
Transition layer	100mm	100mm @ 30% Void space		0.10	(m)
Drainage layer	≥ 200-300mm	300mm @ 35% Void space		0.20	(m)
Storage beneath under-drain layer	≥300mm	300mm @ 35% Void space		0.30	(m)
				Total Depth	1.45 (m)

Required detention volume = Pervious water quality volume +Impervious water quality volume

Bioretention device design

Water Quality Volume=

Live volume of storage needed V_{live}

Required surface area (m^2) $A_s=(WQV)(d)/k(h+d)t$

	266.42	m^3
$V_{live} =$	106.56722	m^3
	301.94047	m^2
WQV=	266.41806	m^3
d=	0.85	m
k=	0.75	m/day
h=	0.15	m
t=	1	day

Live Storage Available=Surface area × Maximum water depth

$V_{WQV} =$	90.58	m^3
-------------	-------	-------

Difference between Live storage needed and available:

-15.99 m^3

Area Short Fall:

53.28

Bioretention live storage surface area :

355.22 m^2

Retention Volume (where 10mm over 90% Impervious Road)

$V_{Retention} =$	125.76	m^3
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Minimum Device Sizing based on Live Storage

Minimum Device Sizing based on Retention Volu

Min Area based on Initial Abstraction:

1197.6857 m^2

Min. Area base on Live Storage:

355 m^2

Rain Garden Sizing:

1197.69 m^2

A schematic of a typical cross section of a raingarden is shown in Figure 8-11 below.

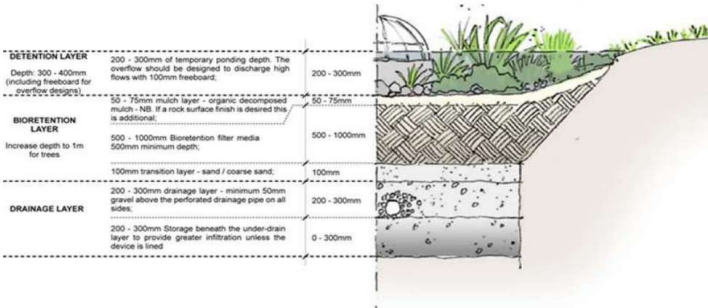


Figure 8-11: Schematic of a rain garden cross section¹⁰⁷

A schematic of a typical cross section of a raingarden is shown in Figure 8-11 below.

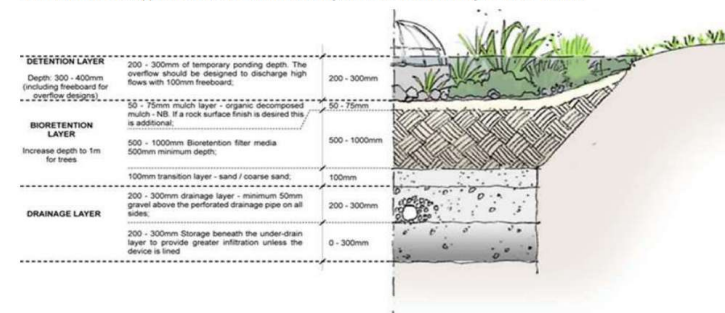


Figure 8-11: Schematic of a rain garden cross section¹⁰⁷

	MAVEN ASSOCIATES	Job Number 180010	Sheet 1	Rev A	Job Title Calc Title	TE KOWHAI EAST RAINGARDEN 4B	Author AZ	Checked KY	Date 8/05/2026
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TR20-07 Bioretention Device Sizing Criteria	Design parameters	RAIN GARDEN E 1
Ponding depth	200-300 mm	200mm @ 100% Void space (m)
Mulch layer	50-100mm	50mm @ 100% Void space (m)
Bioretention filter media	≥ 500-1000mm	500mm @ 30% Void space (m)
Transition layer	100mm	100mm @ 30% Void space (m)
Drainage layer	≥ 200-300mm	300mm @ 35% Void space (m)
Storage beneath under-drain layer	≥ 300mm	300mm @ 35% Void space (m)
Total Depth		1.65 (m)

Required detention volume = Pervious water quality volume +Impervious water quality volume

Bioretention device design

Water Quality Volume=

Live volume of storage needed V_{live}

Required surface area (m^2) $A_s=(WQV)(d)/k(h+d)t$

Live Storage Available=Surface area × Maximum water depth

Difference between Live storage needed and available:

Area Short Fall:

Bioretention live storage surface area :

Retention Volume (where 10mm over 90% Impervious Road)

Min Area based on Initial Abstraction:

Min. Area base on Live Storage:

Rain Garden Sizing:

$V_{live} = 502.01 m^3$

$V_{live} = 200.80215 m^3$

$A_s = 568.93943 m^2$

WQV= 502.00538 m³

d= 0.85 m

k= 0.75 m/day

h= 0.15 m

t= 1 day

$V_{WQV} = 170.68 m^3$

-30.12 m³

100.40

669.34 m²

Minimum Device Sizing based on Live Storage

$V_{Retention} = 236.96 m^3$

Minimum Device Sizing based on Retention Volume

1354.0629 m²

669 m²

1354.06 m²

A schematic of a typical cross section of a raingarden is shown in Figure 8-11 below.

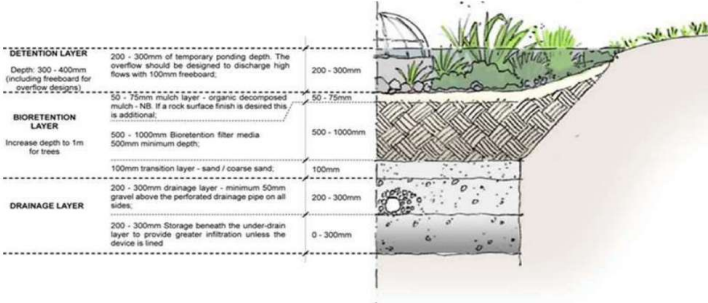


Figure 8-11: Schematic of a rain garden cross section¹⁰⁷

		MAVEN ASSOCIATES	Job Number 180010	Sheet 1	Rev A	Job Title Calc Title	TE KOWHAI EAST RAINGARDEN 5	Author AZ	Checked KY	Date 8/05/2026
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TR20-07 Bioretention Device Sizing Criteria		Design parameters	RAIN GARDEN E 1
Ponding depth	200-300 mm	200mm @ 100% Void space	0.30 (m)
Mulch layer	50-100mm	50mm @ 100% Void space	0.05 (m)
Bioretention filter media	≥ 500-1000mm	500mm @ 30% Void space	0.50 (m)
Transition layer	100mm	100mm @ 30% Void space	0.10 (m)
Drainage layer	≥ 200-300mm	300mm @ 35% Void space	0.20 (m)
Storage beneath under-drain layer	≥300mm	300mm @ 35% Void space	0.30 (m)
Total Depth			1.45 (m)

Required detention volume = Pervious water quality volume +Impervious water quality volume

Bioretention device design

Water Quality Volume= 259.50 m³

Live volume of storage needed V_{live} = 103.79875 m³

Required surface area (m²) A_s=(WQV)(d)/k(h+d)t 294.09646 m²

WQV= 259.49687 m3

d= 0.85 m

k= 0.75 m/day

h= 0.15 m

t= 1 day

Live Storage Available=Surface area × Maximum water depth V_{WQV} = 88.23 m³

Difference between Live storage needed and available: -15.57 m3

Area Short Fall: 51.90

Bioretention live storage surface area : 346.00 m² Minimum Device Sizing based on Live Storage

Retention Volume (where 10mm over 90% Impervious Road) V_{Retention} = 122.49 m³ Minimum Device Sizing based on Retention Volu

Min Area based on Initial Abstraction: 1166.5714 m²

Min. Area base on Live Storage: 346 m²

Rain Garden Sizing: 1166.57 m²

A schematic of a typical cross section of a raingarden is shown in Figure 8-11 below.

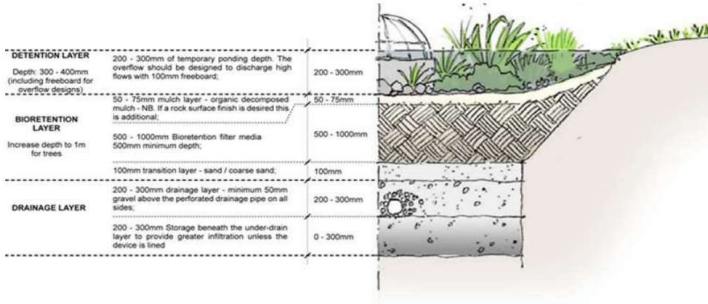


Figure 8-11: Schematic of a rain garden cross section¹⁰⁷

	MAVEN ASSOCIATES	Job Number 180010	Sheet 1	Rev A	Job Title Calc Title	TE KOWHAI EAST RAINGARDEN 6	Author AZ	Checked KY	Date 8/05/2026
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TR20-07 Bioretention Device Sizing Criteria		Design parameters		RAIN GARDEN E 1	
Ponding depth	200-300 mm	200mm @ 100% Void space		0.30	(m)
Mulch layer	50-100mm	50mm @ 100% Void space		0.05	(m)
Bioretention filter media	≥ 500-1000mm	500mm @ 30% Void space		0.50	(m)
Transition layer	100mm	100mm @ 30% Void space		0.10	(m)
Drainage layer	≥ 200-300mm	300mm @ 35% Void space		0.20	(m)
Storage beneath under-drain layer	≥ 300mm	300mm @ 35% Void space		0.40	(m)
Total Depth				1.55	(m)

Required detention volume = Pervious water quality volume +Impervious water quality volume

Bioretention device design

Water Quality Volume=

Live volume of storage needed V_{live}

Required surface area (m^2) $A_s=(WQV)(d)/k(h+d)t$

$V_{live} = 647.35 \text{ m}^3$

$V_{live} = 258.94013 \text{ m}^3$

733.6637 m^2

WQV= 647.35032 m³

d= 0.85 m

k= 0.75 m/day

h= 0.15 m

t= 1 day

Live Storage Available=Surface area × Maximum water depth

Difference between Live storage needed and available:

Area Short Fall:

Bioretention live storage surface area :

Retention Volume (where 10mm over 90% Impervious Road)

Min Area based on Initial Abstraction:

Min. Area base on Live Storage:

Rain Garden Sizing:

$V_{WQV} = 220.10 \text{ m}^3$

-38.84 m³

129.47

863.13 m²

$V_{Retention} = 305.57 \text{ m}^3$

2182.6286 m²

863 m²

2182.63 m²

Minimum Device Sizing based on Live Storage

Minimum Device Sizing based on Retention Volume

A schematic of a typical cross section of a raingarden is shown in Figure 8-11 below.

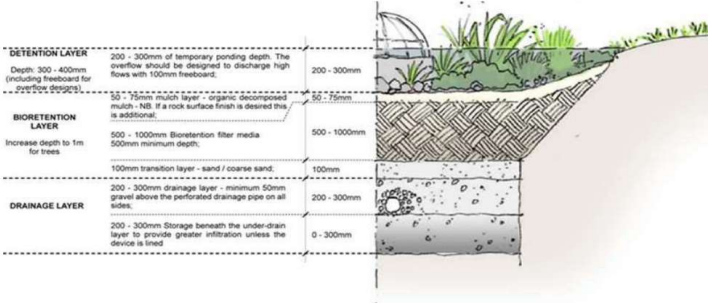


Figure 8-11: Schematic of a rain garden cross section¹⁰⁷

		MAVEN ASSOCIATES	Job Number 180010	Sheet 1	Rev A	Job Title Calc Title	TE KOWHAI EAST RAINGARDEN 8	Author AZ	Checked KY	Date 8/05/2026
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TR20-07 Bioretention Device Sizing Criteria		Design parameters		RAIN GARDEN E 1	
Ponding depth	200-300 mm	200mm @ 100% Void space		0.30	(m)
Mulch layer	50-100mm	50mm @ 100% Void space		0.05	(m)
Bioretention filter media	≥ 500-1000mm	500mm @ 30% Void space		0.50	(m)
Transition layer	100mm	100mm @ 30% Void space		0.10	(m)
Drainage layer	≥ 200-300mm	300mm @ 35% Void space		0.20	(m)
Storage beneath under-drain layer	≥300mm	300mm @ 35% Void space		0.30	(m)
Total Depth				1.45	(m)

Required detention volume = Pervious water quality volume +Impervious water quality volume

Bioretention device design

Water Quality Volume= 324.65 m³

Live volume of storage needed V_{live} = 129.85902 m³

Required surface area (m²) A_s=(WQV)(d)/k(h+d)t 367.9339 m²

WQV= 324.64756 m3

d= 0.85 m

k= 0.75 m/day

h= 0.15 m

t= 1 day

Live Storage Available=Surface area × Maximum water depth V_{WQV} = 110.38 m³

Difference between Live storage needed and available: -19.48 m3

Area Short Fall: 64.93

Bioretention live storage surface area : 432.86 m²

Retention Volume (where 10mm over 90% Impervious Road) V_{Retention} = 153.24 m³

Min Area based on Initial Abstraction: 1459.4571 m²

Min. Area base on Live Storage: 433 m²

Rain Garden Sizing: 1459.46 m²

A schematic of a typical cross section of a raingarden is shown in Figure 8-11 below.

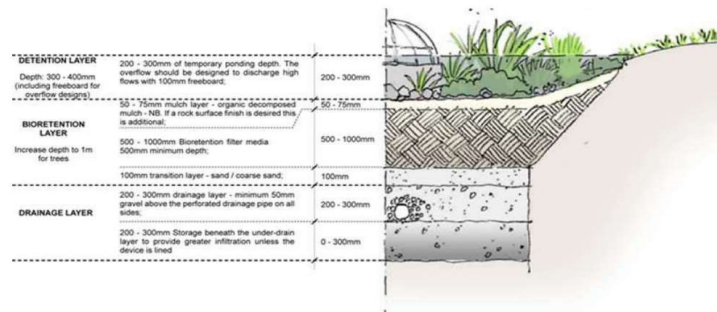


Figure 8-11: Schematic of a rain garden cross section¹⁰⁷

	MAVEN ASSOCIATES	Job Number 180010	Sheet 1	Rev A
Job Title Calc Title	TE KOWHAI EAST Raingarden Sizing_RG1A	Author AZ	Date 8/05/2026	Checked KY

1. Runoff Curve Number (CN) and initial Abstraction (Ia) Road Catchment(ha) = 1.3949

Soil name and classification	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1 ha	Product of CN x area
C	Road Reserve Area (10% Perviousness)	80	0.13949	11.16
C	Road Reserve Area (90% Imperviousness)	98	1.255	123.03
* from Appendix B			Totals =	1.395 134.19

$CN \text{ (weighted)} = \frac{\text{total product}}{\text{total area}} = \frac{134.19}{1.395} = \boxed{96.2}$

$S = \left(\frac{1000}{CN} - 10 \right) 25.4 \text{ (mm)} = \boxed{10.03} \text{ mm}$

$Ia = 0.05 * S = \boxed{0.50} \text{ mm}$

Average recurrence interval, ARI

24 hour rainfall depth, P24

Runoff depth, $Q_{24} = (P_{24} - Ia)^2 / (P_{24} - Ia) + S$

Runoff volume, $V_{24} = 10 \times Q_{24}A$

WQV	2 Year	10 Year	100 Year
26.8333	80.5	125.5	196.4
19.067	71.083	115.711	186.35391
265.96	991.54	1614.05	2599.4506

mm
m3

For Rain Garden Design

	MAVEN ASSOCIATES	Job Number 180010	Sheet 1	Rev A
Job Title Calc Title	TE KOWHAI EAST Raingarden Sizing_RG1B	Author AZ	Date 8/05/2026	Checked KY

1. Runoff Curve Number (CN) and initial Abstraction (Ia) Road Catchment(ha) = 3.9296

Soil name and classification	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1 ha	Product of CN x area
C	Road Reserve Area (10% Perviousness)	80	0.39296	31.44
C	Road Reserve Area (90% Imperviousness)	98	3.537	346.59
* from Appendix B Totals =			3.930	378.03

CN (weighted) = $\frac{\text{total product}}{\text{total area}} = \frac{378.03}{3.930} =$ 96.2

S = $\left(\frac{1000}{CN} - 10\right) 25.4 \text{ (mm)}$ = 10.03 mm

Ia = 0.05 * S = 0.50 mm

Average recurrence interval, ARI

24 hour rainfall depth, P24

Runoff depth, $Q_{24} = (P_{24} - Ia)^2 / (P_{24} - Ia) + S$

Runoff volume, $V_{24} = 10 \times Q_{24}A$

WQV	2 Year	10 Year	100 Year	
26.8333	80.5	125.5	196.4	
19.067	71.083	115.711	186.35391	mm
749.24	2793.3	4546.96	7322.9631	m3

For Rain Garden Design

	MAVEN ASSOCIATES	Job Number 180010	Sheet 1	Rev A
Job Title Calc Title	TE KOWHAI EAST Raingarden Sizing_RG2	Author AZ	Date 8/05/2026	Checked KY

1. Runoff Curve Number (CN) and initial Abstraction (Ia) Road Catchment(ha) = 1.6348

Soil name and classification	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1 ha	Product of CN x area
C	Road Reserve Area (10% Perviousness)	80	0.16348	13.08
C	Road Reserve Area (90% Imperviousness)	98	1.471	144.19
* from Appendix B			Totals =	1.635

$$CN \text{ (weighted)} = \frac{\text{total product}}{\text{total area}} = \frac{157.27}{1.635} = \boxed{96.2}$$

$$S = \left(\frac{1000}{CN} - 10 \right) 25.4 \text{ (mm)} = \boxed{10.03} \text{ mm}$$

$$Ia = 0.05 * S = \boxed{0.50} \text{ mm}$$

Average recurrence interval, ARI

24 hour rainfall depth, P24

Runoff depth, $Q_{24} = (P_{24} - Ia)^2 / (P_{24} - Ia) + S$

Runoff volume, $V_{24} = 10 \times Q_{24}A$

WQV	2 Year	10 Year	100 Year
26.8333	80.5	125.5	196.4
19.067	71.083	115.711	186.35391
311.7	1162.1	1891.64	3046.5137

mm
m3

For Rain Garden Design

	MAVEN ASSOCIATES	Job Number 180010	Sheet 1	Rev A
Job Title Calc Title	TE KOWHAI EAST Raingarden Sizing_RG3A	Author AZ	Date 8/05/2026	Checked KY

1. Runoff Curve Number (CN) and initial Abstraction (Ia) Road Catchment(ha) = 1.3492

Soil name and classification	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1 ha	Product of CN x area
C	Road Reserve Area (10% Perviousness)	80	0.13492	10.79
C	Road Reserve Area (90% Imperviousness)	98	1.214	119.00
* from Appendix B			Totals =	1.349 129.79

$$CN \text{ (weighted)} = \frac{\text{total product}}{\text{total area}} = \frac{129.79}{1.349} = \boxed{96.2}$$

$$S = \left(\frac{1000}{CN} - 10 \right) 25.4 \text{ (mm)} = \boxed{10.03} \text{ mm}$$

$$Ia = 0.05 * S = \boxed{0.50} \text{ mm}$$

Average recurrence interval, ARI

24 hour rainfall depth, P24

Runoff depth, $Q_{24} = (P_{24} - Ia)^2 / (P_{24} - Ia) + S$

Runoff volume, $V_{24} = 10 \times Q_{24}A$

WQV	2 Year	10 Year	100 Year	
26.8333	80.5	125.5	196.4	
19.067	71.083	115.711	186.35391	mm
257.25	959.05	1561.17	2514.2869	m3

For Rain Garden Design

	MAVEN ASSOCIATES	Job Number 180010	Sheet 1	Rev A
Job Title Calc Title	TE KOWHAI EAST Raingarden Sizing_RG3B	Author AZ	Date 8/05/2026	Checked KY

1. Runoff Curve Number (CN) and initial Abstraction (Ia) Road Catchment(ha) = 1.3541

Soil name and classification	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1 ha	Product of CN x area
C	Road Reserve Area (10% Perviousness)	80	0.13541	10.83
C	Road Reserve Area (90% Imperviousness)	98	1.219	119.43
* from Appendix B			Totals =	1.354 130.26

$CN \text{ (weighted)} = \frac{\text{total product}}{\text{total area}} = \frac{130.26}{1.354} =$
96.2

$S = \left(\frac{1000}{CN} - 10 \right) 25.4 \text{ (mm)} =$
10.03 mm

$Ia = 0.05 * S =$
0.50 mm

Average recurrence interval, ARI

24 hour rainfall depth, P24

Runoff depth, $Q_{24} = (P_{24} - Ia)^2 / (P_{24} - Ia) + S$

Runoff volume, $V_{24} = 10 \times Q_{24}A$

WQV	2 Year	10 Year	100 Year
26.8333	80.5	125.5	196.4
19.067	71.083	115.711	186.35391
258.18	962.54	1566.84	2523.4183

For Rain Garden Design

	MAVEN ASSOCIATES	Job Number 180010	Sheet 1	Rev A
Job Title Calc Title	TE KOWHAI EAST Raingarden Sizing_RG4A	Author AZ	Date 8/05/2026	Checked KY

1. Runoff Curve Number (CN) and initial Abstraction (Ia) Road Catchment(ha) = 1.3973

Soil name and classification	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1 ha	Product of CN x area
C	Road Reserve Area (10% Perviousness)	80	0.13973	11.18
C	Road Reserve Area (90% Imperviousness)	98	1.258	123.24
* from Appendix B			Totals =	1.397

$$CN \text{ (weighted)} = \frac{\text{total product}}{\text{total area}} = \frac{134.42}{1.397} = \boxed{96.2}$$

$$S = \left(\frac{1000}{CN} - 10 \right) 25.4 \text{ (mm)} = \boxed{10.03} \text{ mm}$$

$$Ia = 0.05 * S = \boxed{0.50} \text{ mm}$$

Average recurrence interval, ARI

24 hour rainfall depth, P24

Runoff depth, $Q_{24} = (P_{24} - Ia)^2 / (P_{24} - Ia) + S$

Runoff volume, $V_{24} = 10 \times Q_{24}A$

WQV	2 Year	10 Year	100 Year
26.8333	80.5	125.5	196.4
19.067	71.083	115.711	186.35391
266.42	99.325	161.682	260.39231

mm
m3

For Rain Garden Design

	MAVEN ASSOCIATES	Job Number 180010	Sheet 1	Rev A
Job Title Calc Title	TE KOWHAI EAST Raingarden Sizing_RG4B	Author AZ	Date 8/05/2026	Checked KY

1. Runoff Curve Number (CN) and initial Abstraction (Ia) Road Catchment(ha) = 2.6329

Soil name and classification	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1 ha	Product of CN x area
C	Road Reserve Area (10% Perviousness)	80	0.26329	21.06
C	Road Reserve Area (90% Imperviousness)	98	2.370	232.22
* from Appendix B			Totals =	2.633 253.28

$CN \text{ (weighted)} = \frac{\text{total product}}{\text{total area}} = \frac{253.28}{2.633} =$
96.2

$S = \left(\frac{1000}{CN} - 10 \right) 25.4 \text{ (mm)} =$
10.03 mm

$Ia = 0.05 * S =$
0.50 mm

Average recurrence interval, ARI

24 hour rainfall depth, P24

Runoff depth, $Q_{24} = (P_{24} - Ia)^2 / (P_{24} - Ia) + S$

Runoff volume, $V_{24} = 10 \times Q_{24}A$

WQV	2 Year	10 Year	100 Year
26.8333	80.5	125.5	196.4
19.067	71.083	115.711	186.35391
502.01	1871.5	3046.54	4906.512

mm
m3

For Rain Garden Design

	MAVEN ASSOCIATES	Job Number 180010	Sheet 1	Rev A
Job Title Calc Title	TE KOWHAI EAST Raingarden Sizing_RG5	Author AZ	Date 8/05/2026	Checked KY

1. Runoff Curve Number (CN) and initial Abstraction (Ia)

Road Catchment(ha) = 1.361

Soil name and classification	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1 ha	Product of CN x area
C	Road Reserve Area (10% Perviousness)	80	0.1361	10.89
C	Road Reserve Area (90% Imperviousness)	98	1.225	120.04
* from Appendix B			Totals =	130.93

CN (weighted) = $\frac{\text{total product}}{\text{total area}} = \frac{130.93}{1.361} =$
96.2

S = $\left(\frac{1000}{CN} - 10\right) 25.4 \text{ (mm)}$
=
10.03 mm

Ia = $0.05 * S$
=
0.50 mm

Average recurrence interval, ARI

24 hour rainfall depth, P24

Runoff depth, $Q_{24} = (P_{24} - Ia)^2 / (P_{24} - Ia) + S$

Runoff volume, $V_{24} = 10 \times Q_{24}A$

WQV	2 Year	10 Year	100 Year
26.8333	80.5	125.5	196.4
19.067	71.083	115.711	186.35391
259.5	967.44	1574.82	2536.2767

mm

m3

For Rain Garden Design

	MAVEN ASSOCIATES	Job Number 180010	Sheet 1	Rev A
Job Title Calc Title	TE KOWHAI EAST Raingarden Sizing_RG6	Author AZ	Date 8/05/2026	Checked KY

1. Runoff Curve Number (CN) and initial Abstraction (Ia) Road Catchment(ha) = 3.3952

Soil name and classification	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1 ha	Product of CN x area
C	Road Reserve Area (10% Perviousness)	80	0.33952	27.16
C	Road Reserve Area (90% Imperviousness)	98	3.056	299.46
* from Appendix B			Totals =	3.395 326.62

$$CN \text{ (weighted)} = \frac{\text{total product}}{\text{total area}} = \frac{326.62}{3.395} = \boxed{96.2}$$

$$S = \left(\frac{1000}{CN} - 10 \right) 25.4 \text{ (mm)} = \boxed{10.03} \text{ mm}$$

$$Ia = 0.05 * S = \boxed{0.50} \text{ mm}$$

Average recurrence interval, ARI

24 hour rainfall depth, P24

Runoff depth, $Q_{24} = (P_{24} - Ia)^2 / (P_{24} - Ia) + S$

Runoff volume, $V_{24} = 10 \times Q_{24}A$

WQV	2 Year	10 Year	100 Year
26.8333	80.5	125.5	196.4
19.067	71.083	115.711	186.35391
647.35	2413.4	3928.61	6327.0878

mm
m3

For Rain Garden Design

	MAVEN ASSOCIATES	Job Number 180010	Sheet 1	Rev A
Job Title Calc Title	TE KOWHAI EAST Raingarden Sizing_RG8	Author AZ	Date 8/05/2026	Checked KY

1. Runoff Curve Number (CN) and initial Abstraction (Ia) Road Catchment(ha) = 1.7027

Soil name and classification	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1 ha	Product of CN x area
C	Road Reserve Area (10% Perviousness)	80	0.17027	13.62
C	Road Reserve Area (90% Imperviousness)	98	1.532	150.18
* from Appendix B			Totals =	1.703 163.80

$$CN \text{ (weighted)} = \frac{\text{total product}}{\text{total area}} = \frac{163.80}{1.703} = \boxed{96.2}$$

$$S = \left(\frac{1000}{CN} - 10 \right) 25.4 \text{ (mm)} = \boxed{10.03} \text{ mm}$$

$$Ia = 0.05 * S = \boxed{0.50} \text{ mm}$$

Average recurrence interval, ARI

24 hour rainfall depth, P24

Runoff depth, $Q_{24} = (P_{24} - Ia)^2 / (P_{24} - Ia) + S$

Runoff volume, $V_{24} = 10 \times Q_{24}A$

WQV	2 Year	10 Year	100 Year
26.8333	80.5	125.5	196.4
19.067	71.083	115.711	186.35391
324.65	1210.3	1970.2	3173.048

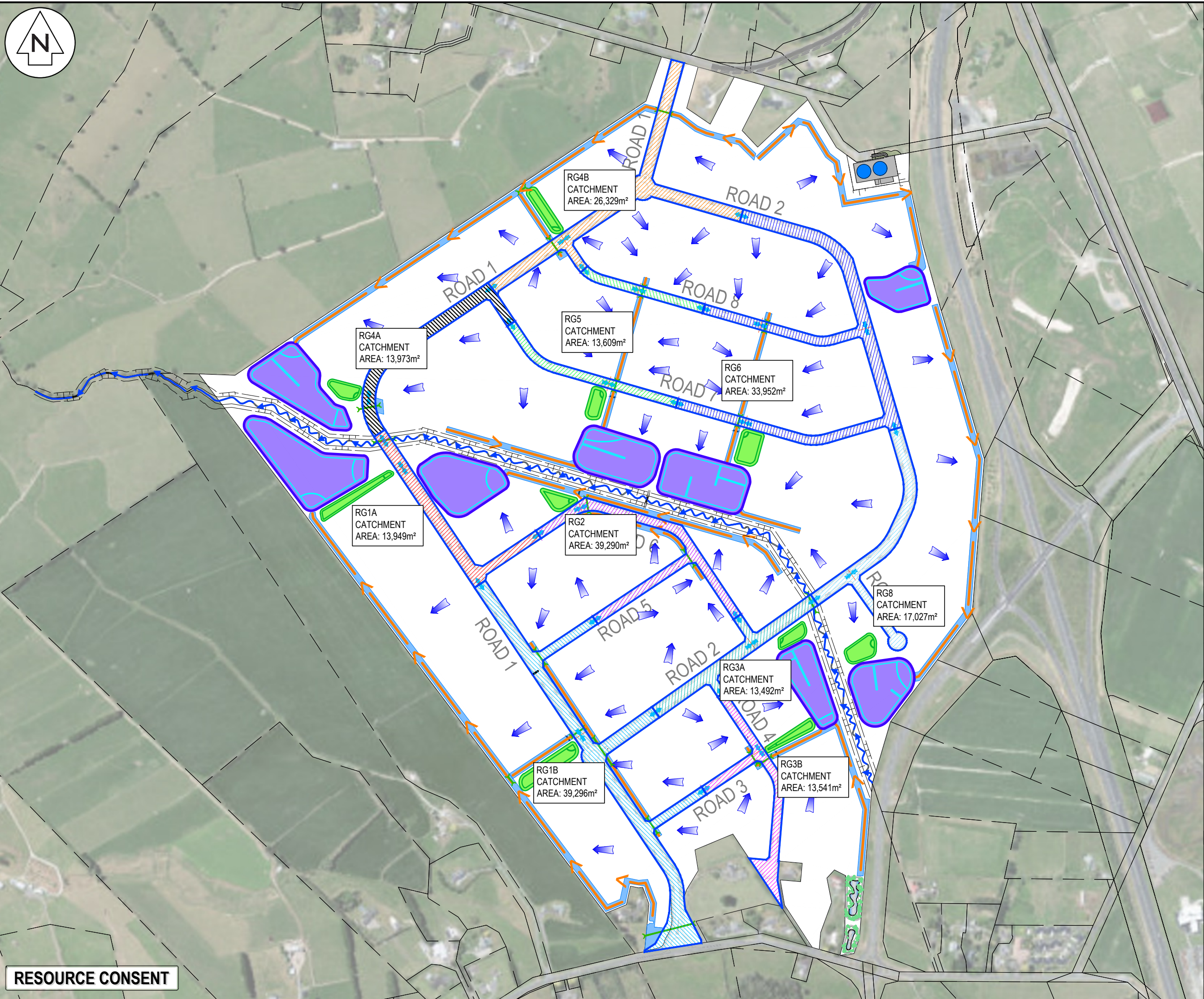
mm
m3

For Rain Garden Design



RESOURCE CONSENT

DATE: 8/24/25 FILE PATH: F:\Maven\PROJECTS\180010 - TE KOWHAI EAST\DWG\180010-C481 SUB-CATCHMENT PLAN.DWG



- NOTES
1. ALL WORKS TO BE IN ACCORDANCE WITH THE WAIKATO DISTRICT COUNCIL/IAWAI STANDARD DETAILS AND THE WAIKATO REGIONAL INFRASTRUCTURE TECHNICAL SPECIFICATION.
 2. COORDINATES IN TERMS OF NZ GEODETIC DATUM MT EDEN 2000. LEVELS IN TERMS OF THE NEW ZEALAND VERTICAL DATUM 2016.
 3. IT IS THE CONTRACTOR'S RESPONSIBILITY TO LOCATE ALL SERVICES THAT MAY BE AFFECTED BY THEIR OPERATIONS.
 4. ALL CATCHMENT AREAS SHOWN ON THIS PLAN ARE IN SQUARE METRES (m²).

- LEGEND
- PR SUB-CATCHMENT BOUNDARY
 - PR MODIFIED STREAM
 - PR 12.5m WIDE CONVEYANCE CHANNEL
 - PR WETLAND
 - PR RAINGARDEN
 - PR BERM/LANDSCAPE
 - PR LOT SHEET FLOW
 - PR ROADING OLF/DIRECTION
 - PR ROADING CREST/HIGH PT
 - PR ROADING SAG/LOW PT

A	RESOURCE CONSENT	AZ	06/2026
Rev	Description	By	Date
		By	Date
Survey	MAVEN		10/2025
Design	AF		07/2026
Drawn	AF		07/2026
Checked	KY		07/2026

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Project
PROPOSED INDUSTRIAL DEVELOPMENT AT TE KOWHAI EAST FOR TE KOWHAI EAST LIMITED PARTNERSHIP

Title
PR STORMWATER SUB-CATCHMENT PLAN RAINGARDENS

Project no.	180010
Scale	1:7500 @ A3
Cad file	180010-C481 SUB-CATCHMENT PLAN.DWG
Drawing no.	C483
Rev	A

APPENDIX C.3 – SW PRIMARY PIPED NETWORK

 MAVEN ASSOCIATES		Job Number	Sheet	Rev	Title	Author		Date	Checked
		180010	1	A	SW Pipe Capacity (10yrs)	AF		06/2026	

Rainfall Depth	IA	ARI 10YR	ARI 100YR	Pipe ks factor	1.5	mm (pipes up to 1.0m dia)
NIWA Rainfall Data		91.1	143	mm/hr	0.6	mm (pipes over 1.0m dia)
SSP5-8.5	10	138.84	220.80	mm/hr	0.6	

	C Number	
Lots	0.7	
JOAL	0.7	
Roads	0.9	

Pipes under this assessment are all concrete

Area to fill

SSP5-8.5 Adjusted Depth (mm)				
	2yr	5Yr	10Yr	100Yr
24hr	81.4	107.5	127.1	198.9
48hr	94.9	125.3	148.5	231.4
72hr	103.1	136.0	161.3	250.1

SSP5-8.5 Adjusted Intensity (mm/hr)				
	2yr	5Yr	10Yr	100Yr
10min	87.9	116.9	138.84	220.8

ge Point: rain garden and dry basin for catchment E

PLAN CHANGE AREA

From	To	Catchment	Catchment Area	C	Peak Flow rate - 10YR ARI	Cum. Flow	Pipe dia	Gradient	Capacity	Percent Capacity	Remaining	Velocity	Check
MH number	MH number	letter	m ²		l/s	l/s	m	%	l/s	%	l/s	m/s	OK

CATCHMENT 1

CATCHMENT 1 - 1A.1-1A.6

LINE 1A.1													
OUTLET 1A.1	1A.1.2		2870	0.90	99.62	384.26	0.525	1.00	432.3	88.9%	48.0	2.0	OK
1A.1.2	1A.1.3		1791	0.90	62.16	232.76	0.525	0.40	273.3	85.2%	40.6	1.3	OK
1A.1.3	1A.1.4		1792	0.90	62.20	170.60	0.450	0.45	193.0	88.4%	22.4	1.2	OK
1A.1.4	1A.1.5		1353	0.90	46.96	108.40	0.300	1.20	122.6	88.4%	14.2	1.7	OK
1A.1.5	1A.1.6		1770	0.90	61.44	61.44	0.300	0.40	70.8	86.8%	9.3	1.0	OK
1A.1.2	1A.1.2.1		1495	0.90	51.89	51.89	0.300	0.30	61.3	84.7%	9.4	0.9	OK

CATCHMENT 1 - 1.2DS-1B.1.3

LINE 1B.1													
1.2ds	1B.1.1		1793	0.90	62.23	176.39	0.400	0.89	198.8	88.7%	22.4	1.6	OK
1B.1.1	1B.1.2		1791	0.90	62.16	114.16	0.375	0.40	127.3	89.7%	13.2	1.2	OK
1B.1.2	1B.1.3		1498	0.90	51.99	51.99	0.300	0.27	58.1	89.5%	6.1	0.8	OK

CATCHMENT 1 - 1.2DS-1B.2.2

LINE 1B.2													
1.2ds	1B.2.1		2303	0.90	79.94	135.85	0.300	1.90	154.3	88.0%	18.5	2.2	OK
1B.2.1	1B.2.2		1611	0.90	55.92	55.92	0.300	0.50	79.1	70.7%	23.2	1.1	OK

CATCHMENT 1 - 1.3DS-1B.3.1-3.3

LINE 1B.3													
1.3ds	1B.3.1		1789	0.90	62.09	575.06	0.600	1.10	644.9	89.2%	69.8	2.3	OK
1B.3.1	1B.3.2		5225	0.90	181.36	512.97	0.600	0.90	583.3	87.9%	70.3	2.1	OK
1B.3.2	1B.3.3		9554	0.90	331.61	331.61	0.450	1.70	375.1	88.4%	43.5	2.4	OK

 MAVEN ASSOCIATES		Job Number	Sheet	Rev	Title	Author		Date	Checked
		180010	1	A	SW Pipe Capacity (10yrs)	AF		06/2026	

Rainfall Depth	IA	ARI 10YR	ARI 100YR	Pipe ks factor	1.5	mm (pipes up to 1.0m dia)
NIWA Rainfall Data		91.1	143	mm/hr	0.6	mm (pipes over 1.0m dia)
SSP5-8.5	10	138.84	220.80	mm/hr	0.6	

	C Number	
Lots	0.7	
JOAL	0.7	
Roads	0.9	

Pipes under this assessment are all concrete

Area to fill

SSP5-8.5 Adjusted Depth (mm)				
	2yr	5Yr	10Yr	100Yr
24hr	81.4	107.5	127.1	198.9
48hr	94.9	125.3	148.5	231.4
72hr	103.1	136.0	161.3	250.1

SSP5-8.5 Adjusted Intensity (mm/hr)				
	2yr	5Yr	10Yr	100Yr
10min	87.9	116.9	138.84	220.8

ge Point: rain garden and dry basin for catchment E

PLAN CHANGE AREA

From	To	Catchment	Catchment Area	C	Peak Flow rate - 10YR ARI	Cum. Flow	Pipe dia	Gradient	Capacity	Percent Capacity	Remaining	Velocity	Check
MH number	MH number	letter	m ²		l/s	l/s	m	%	l/s	%	l/s	m/s	OK

CATCHMENT 1 - 1B.4.1-4.4													
LINE 1B.4													
1B.4	1B.4.1		1800	0.90	62.48	762.59	0.900	0.23	857.6	88.9%	95.0	1.3	OK
1B.4.1	1B.4.2		1770	0.90	61.44	700.12	0.900	0.19	779.4	89.8%	79.3	1.2	OK
1B.4.2	1B.4.3		6531	0.90	226.69	638.68	0.750	0.42	717.3	89.0%	78.6	1.6	OK
1B.4.3	1B.4.4		1092	0.90	37.90	412.00	0.750	0.18	469.5	87.8%	57.5	1.1	OK
1B.4.4	1B.4.5		6290	0.90	218.32	374.09	0.750	0.15	428.6	87.3%	54.5	1.0	OK
1B.4.5	1B.4.6		4488	0.90	155.77	155.77	0.450	0.37	175.0	89.0%	19.2	1.1	OK

CATCHMENT 1 - 1.5DS-1B.5.2													
LINE 1B.5													
1.5ds	1B.5.1		935	0.90	32.45	580.02	0.600	1.10	644.9	89.9%	64.8	2.3	OK
1B.5.1	1B.5.2		6299	0.90	218.63	547.57	0.600	1.00	614.8	89.1%	67.3	2.2	OK
1B.5.2	1B.5.3		9477	0.90	328.94	328.94	0.450	1.70	375.1	87.7%	46.2	2.4	OK

CATCHMENT 1 - OUTLET_1B 6.1.1-1B.6.3													
LINE 1B.6													
1B 6.1.1	1B.6.1		846	0.90	29.36	633.23	0.675	0.71	706.7	89.6%	73.4	2.0	OK
1B.6.1	1B.6.2		1781	0.90	61.82	603.87	0.600	1.20	673.5	89.7%	69.7	2.4	OK
1B.6.2	1B.6.3		15617	0.90	542.05	542.05	0.600	1.00	614.8	88.2%	72.8	2.2	OK

CATCHMENT 1 - 1.3DS-1B.7.11													
LINE 1B.7													
1.3ds	1B.7.1		1683	0.90	58.42	709.84	0.825	0.31	791.9	89.6%	82.1	1.5	OK
1B.7.1	1B.7.2		1793	0.90	62.23	651.42	0.750	0.43	725.8	89.8%	74.4	1.6	OK
1B.7.2	1B.7.3		1765	0.90	61.26	589.19	0.675	0.62	660.3	89.2%	71.2	1.8	OK
1B.7.3	1B.7.4		1061	0.90	36.83	527.93	0.675	0.50	593.0	89.0%	65.1	1.7	OK
1B.7.4	1B.7.5		1791	0.90	62.16	491.10	0.675	0.44	556.3	88.3%	65.2	1.6	OK
1B.7.5	1B.7.6		1731	0.90	60.08	428.94	0.675	0.33	481.7	89.0%	52.8	1.3	OK
1B.7.6	1B.7.7		1800	0.90	62.48	368.85	0.600	0.46	417.0	88.5%	48.1	1.5	OK
1B.7.7	1B.7.8		1737	0.90	60.29	306.38	0.525	0.70	361.6	84.7%	55.3	1.7	OK
1B.7.8	1B.7.9		1789	0.90	62.09	246.09	0.375	2.00	284.9	86.4%	38.8	2.6	OK
1B.7.9	1B.7.10		967	0.90	33.56	183.99	0.375	1.10	211.2	87.1%	27.2	1.9	OK
1B.7.10	1B.7.11		1795	0.90	62.30	62.30	0.300	0.40	70.8	88.0%	8.5	1.0	OK
1B.7.10	1B.7.10.1		2539	0.90	88.13	88.13	0.300	0.80	100.1	88.0%	12.0	1.4	OK

M A E N MAVEN ASSOCIATES				Job Number	Sheet	Rev	Title	Author		Date	Checked		
				180010	1	A	SW Pipe Capacity (10yrs)	AF		06/2026			
Rainfall Depth				IA	ARI 10YR	ARI 100YR							
NIWA Rainfall Data					91.1	143							
SSP5-8.5		10	138.84	220.80	mm/hr	Pipe ks factor	1.5	mm (pipes up to 1.0m dia)					
					mm/hr	0.6	mm (pipes over 1.0m dia)						
					mm/hr	0.6							
				C Number	Pipes under this assessment are all concrete								
Lots				0.7	Area to fill								
JOAL				0.7									
Roads				0.9									
				SSP5-8.5 Adjusted Depth (mm)									
				2yr	5Yr	10Yr	100Yr						
24hr				81.4	107.5	127.1	198.9						
48hr				94.9	125.3	148.5	231.4						
72hr				103.1	136.0	161.3	250.1						
				SSP5-8.5 Adjusted Intensity (mm/hr)									
				2yr	5Yr	10Yr	100Yr						
10min				87.9	116.9	138.84	220.8						
ge Point: rain garden and dry basin for catchment E													
PLAN CHANGE AREA													
From	To	Catchment	Catchment Area	C	Peak Flow rate - 10YR ARI	Cum. Flow	Pipe dia	Gradient	Capacity	Percent Capacity	Remaining	Velocity	Check
MH number	MH number	letter	m ²		l/s	l/s	m	%	l/s	%	l/s	m/s	OK
CATCHMENT 2													
CATCHMENT 2 -2.1DS-2.1.3													
LINE 2.1													
C2.1 DS	2.1.1		1877	0.90	65.15	284.96	0.450	1.30	328.0	86.9%	43.1	2.1	OK
2.1.1	2.1.2		1325	0.90	45.99	219.81	0.375	1.50	246.7	89.1%	26.9	2.2	OK
2.1.2	2.1.3		5008	0.90	173.82	173.82	0.300	3.00	194.0	89.6%	20.1	2.7	OK
CATCHMENT -2.1DS-2.2.6													
LINE 2.2													
C2.1 DS	2.2.1		1666	0.90	57.83	232.27	0.450	0.82	260.5	89.2%	28.2	1.6	OK
2.2.1	2.2.2		1285	0.90	44.60	174.45	0.375	1.00	201.4	86.6%	26.9	1.8	OK
2.2.2	2.2.3		1605	0.90	55.71	129.85	0.375	0.52	145.2	89.4%	15.3	1.3	OK
2.2.3	2.2.4		0	0.90	0.00	74.14	0.375	0.17	82.9	89.4%	8.8	0.8	OK
2.2.4	2.2.5		1444	0.90	50.12	74.14	0.375	0.17	82.9	89.4%	8.8	0.8	OK
2.2.5	2.2.6		692	0.90	24.02	24.02	0.225	0.30	28.7	83.7%	4.7	0.7	OK
CATCHMENT -2.2DS-2.3.4													
LINE 2.3													
CULVERT 2.2	2.3.1		1054	0.90	36.58	1376.35	0.975	0.49	1545.1	89.1%	168.7	2.1	OK
2.3.1	2.3.2		6797	0.90	235.92	1339.77	0.975	0.46	1497.0	89.5%	157.3	2.0	OK
2.3.2	2.3.3		11788	0.90	409.15	1103.85	0.900	0.48	1239.0	89.1%	135.2	1.9	OK
2.3.3	2.3.4		20015	0.90	694.70	694.70	0.750	0.50	782.7	88.8%	88.0	1.8	OK
CATCHMENT -CULVERT 2.2 DS-2.4.4													
LINE 2.4													
C2.1 DS	2.4.1		1794	0.90	62.27	1241.51	0.975	0.40	1396.0	88.9%	154.4	1.9	OK
2.4.1	2.4.2		12124	0.90	420.81	1179.24	0.975	0.36	1324.3	89.0%	145.1	1.8	OK
2.4.2	2.4.3		5355	0.90	185.87	758.43	0.975	0.15	854.7	88.7%	96.3	1.1	OK
2.4.3	2.4.4		5425	0.90	188.30	572.56	0.900	0.13	644.6	88.8%	72.1	1.0	OK
2.4.4	2.4.5		5495	0.90	190.73	384.26	0.750	0.15	428.6	89.7%	44.3	1.0	OK
2.4.5	2.4.6		5576	0.90	193.54	193.54	0.600	0.13	221.6	87.3%	28.0	0.8	OK

 MAVEN ASSOCIATES		Job Number	Sheet	Rev	Title	Author		Date	Checked
		180010	1	A	SW Pipe Capacity (10yrs)	AF		06/2026	

Rainfall Depth	IA	ARI 10YR	ARI 100YR	Pipe ks factor	1.5	mm (pipes up to 1.0m dia)
NIWA Rainfall Data		91.1	143	mm/hr	0.6	mm (pipes over 1.0m dia)
SSP5-8.5	10	138.84	220.80	mm/hr	0.6	

	C Number	Pipes under this assessment are all concrete
Lots	0.7	 Area to fill
JOAL	0.7	
Roads	0.9	

SSP5-8.5 Adjusted Depth (mm)				
	2yr	5Yr	10Yr	100Yr
24hr	81.4	107.5	127.1	198.9
48hr	94.9	125.3	148.5	231.4
72hr	103.1	136.0	161.3	250.1

SSP5-8.5 Adjusted Intensity (mm/hr)				
	2yr	5Yr	10Yr	100Yr
10min	87.9	116.9	138.84	220.8

ge Point: rain garden and dry basin for catchment E

PLAN CHANGE AREA

From	To	Catchment	Catchment Area	C	Peak Flow rate - 10YR ARI	Cum. Flow	Pipe dia	Gradient	Capacity	Percent Capacity	Remaining	Velocity	Check
MH number	MH number	letter	m ²		l/s	l/s	m	%	l/s	%	l/s	m/s	OK

CATCHMENT 3

CATCHMENT 3 -3A.1-3A.1.7

LINE 3A 3A.1-3A.1.7

OUTLET3A.1	3A.1.1		0	0.90	0.00	813.30	0.900	0.26	911.8	89.2%	98.5	1.4	OK
3A.1.1	3A.1.2		2366	0.90	82.12	813.30	0.900	0.26	911.8	89.2%	98.5	1.4	OK
3A.1.2	3A.1.3		3321	0.90	115.27	606.72	0.900	0.15	692.5	87.6%	85.8	1.1	OK
3A.1.3	3A.1.4		1717	0.90	59.60	491.45	0.750	0.25	553.4	88.8%	61.9	1.3	OK
3A.1.4	3A.1.5		2643	0.90	91.74	431.85	0.675	0.34	489.0	88.3%	57.1	1.4	OK
3A.1.5	3A.1.6		5831	0.90	202.39	340.11	0.600	0.40	388.8	87.5%	48.7	1.4	OK
3A.1.6	3A.1.7		3968	0.90	137.73	137.73	0.375	0.60	156.0	88.3%	18.2	1.4	OK

LINE 3A 3A.1.2-3A.1.2.1

3A.1.2	3A.1.2.1		3586	0.90	124.47	124.47	0.300	1.60	141.6	87.9%	17.2	2.0	OK
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CATCHMENT3 -3.1DS-3B.1.3

LINE 3B.1

3.1DS	3B.1.1		3610	0.90	125.30	521.68	0.600	0.90	583.3	89.4%	61.6	2.1	OK
3B.1.1	3B.1.2		1798	0.90	62.41	396.38	0.600	0.52	443.3	89.4%	46.9	1.6	OK
3B.1.2	3B.1.3		9622	0.90	333.97	333.97	0.525	0.80	386.6	86.4%	52.6	1.8	OK

 MAVEN ASSOCIATES		Job Number	Sheet	Rev	Title	Author		Date	Checked
		180010	1	A	SW Pipe Capacity (10yrs)	AF		06/2026	

Rainfall Depth	IA	ARI 10YR	ARI 100YR	mm/hr mm/hr	Pipe ks factor 1.5 0.6 0.6	mm (pipes up to 1.0m dia) mm (pipes over 1.0m dia)					
NIWA Rainfall Data		91.1	143								
SSP5-8.5	10	138.84	220.80								

		C Number									
Lots		0.7									
JOAL		0.7									
Roads		0.9									

Pipes under this assessment are all concrete

Area to fill

SSP5-8.5 Adjusted Depth (mm)									
	2yr	5Yr	10Yr	100Yr					
24hr	81.4	107.5	127.1	198.9					
48hr	94.9	125.3	148.5	231.4					
72hr	103.1	136.0	161.3	250.1					

SSP5-8.5 Adjusted Intensity (mm/hr)									
	2yr	5Yr	10Yr	100Yr					
10min	87.9	116.9	138.84	220.8					

ge Point: rain garden and dry basin for catchment E

PLAN CHANGE AREA

From	To	Catchment	Catchment Area	C	Peak Flow rate - 10YR ARI	Cum. Flow	Pipe dia	Gradient	Capacity	Percent Capacity	Remaining	Velocity	Check
MH number	MH number	letter	m ²		l/s	l/s	m	%	l/s	%	l/s	m/s	OK

CATCHMENT 3 .2DS-3B.2.2													
LINE 3B.2													
3.2DS	3B.2.1		1798	0.90	62.41	552.01	0.600	1.00	614.8	89.8%	62.8	2.2	OK
3B.2.1	2.1.2		14106	0.90	489.61	489.61	0.600	0.80	549.9	89.0%	60.3	1.9	OK

CATCHMENT 3 -3B.3.1-3B.3.5													
LINE 3B.3													
Culvert 3.1 DS	3B.3.1		0	0.90	0.00	404.64	0.600	0.54	451.8	89.6%	47.1	1.6	OK
3B.3.1	3B.3.2		1059	0.90	36.76	404.64	0.600	0.54	451.8	89.6%	47.1	1.6	OK
3B.3.2	3B.3.3		1786	0.90	61.99	367.88	0.525	0.90	410.1	89.7%	42.2	1.9	OK
3B.3.3	3B.3.4		7031	0.90	244.04	305.89	0.525	0.63	343.1	89.2%	37.2	1.6	OK
3B.3.4	3B.3.5		1782	0.90	61.85	61.85	0.300	0.40	70.8	87.4%	8.9	1.0	OK

CATCHMENT 3B.4-3B.4.1													
LINE 3B.4													
3B.4	3B.4.1		10655	0.90	369.83	563.54	0.600	1.05	630.0	89.4%	66.5	2.2	OK
3B.4.1	3B.4.2		5581	0.90	193.71	193.71	0.600	0.13	221.6	87.4%	27.9	0.8	OK

 MAVEN ASSOCIATES		Job Number	Sheet	Rev	Title	Author		Date	Checked
		180010	1	A	SW Pipe Capacity (10yrs)	AF		06/2026	

Rainfall Depth	IA	ARI 10YR	ARI 100YR	mm/hr	mm/hr	Pipe ks factor 1.5 0.6 0.6	mm (pipes up to 1.0m dia) mm (pipes over 1.0m dia)					
NIWA Rainfall Data		91.1	143									
SSP5-8.5	10	138.84	220.80									

		C Number										
Lots		0.7										
JOAL		0.7										
Roads		0.9										

Pipes under this assessment are all concrete

Area to fill

SSP5-8.5 Adjusted Depth (mm)									
	2yr	5Yr	10Yr	100Yr					
24hr	81.4	107.5	127.1	198.9					
48hr	94.9	125.3	148.5	231.4					
72hr	103.1	136.0	161.3	250.1					

SSP5-8.5 Adjusted Intensity (mm/hr)									
	2yr	5Yr	10Yr	100Yr					
10min	87.9	116.9	138.84	220.8					

ge Point: rain garden and dry basin for catchment E

PLAN CHANGE AREA

From	To	Catchment	Catchment Area	C	Peak Flow rate - 10YR ARI	Cum. Flow	Pipe dia	Gradient	Capacity	Percent Capacity	Remaining	Velocity	Check
MH number	MH number	letter	m ²		l/s	l/s	m	%	l/s	%	l/s	m/s	OK

CATCHMENT 4

CATCHMENT4-4.4DS-4A.1.7

LINE 4A.1

4.4DS	4A1.1		2500	0.90	86.77	1070.53	0.825	0.70	1190.2	89.9%	119.6	2.2	OK
4A1.1	4A1.2		1779	0.90	61.75	983.76	0.825	0.60	1101.9	89.3%	118.1	2.1	OK
4A1.2	4A1.3		0	0.90	0.00	922.01	0.750	0.88	1038.4	88.8%	116.4	2.4	OK
4A1.3	4A1.4		12448	0.90	432.06	922.01	0.750	0.88	1038.4	88.8%	116.4	2.4	OK
4A1.4	4A1.5		1794	0.90	62.27	489.95	0.600	0.80	549.9	89.1%	60.0	1.9	OK
4A1.5	4A1.6		1797	0.90	62.37	427.69	0.600	0.62	484.1	88.3%	56.4	1.7	OK
4A1.6	4A1.7		10525	0.90	365.31	365.31	0.600	0.45	412.4	88.6%	47.1	1.5	OK

CATCHMENT 4.4DS-4A.2.1

LINE 4A.2

4.4DS	4A.2.1		1021	0.90	35.44	35.44	0.300	0.20	50.0	70.9%	14.6	0.7	OK
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CATCHMENT4-B1.1-1.3

LINE 4B.1

4.2DS	4B.1.1		1794	0.90	62.27	414.04	0.525	1.20	473.5	87.4%	59.5	2.2	OK
4B.1.1	4B.1.2		8457	0.90	293.53	351.78	0.525	0.85	398.5	88.3%	46.7	1.8	OK
4B.1.2	4B.1.3		1678	0.90	58.24	58.24	0.300	0.35	66.2	88.0%	7.9	0.9	OK

 MAVEN ASSOCIATES		Job Number	Sheet	Rev	Title	Author		Date	Checked
		180010	1	A	SW Pipe Capacity (10yrs)	AF		06/2026	

Rainfall Depth	IA	ARI 10YR	ARI 100YR	Pipe ks factor	1.5	mm (pipes up to 1.0m dia)
NIWA Rainfall Data		91.1	143	mm/hr	0.6	mm (pipes over 1.0m dia)
SSP5-8.5	10	138.84	220.80	mm/hr	0.6	

	C Number	
Lots	0.7	
JOAL	0.7	
Roads	0.9	

Pipes under this assessment are all concrete

Area to fill

SSP5-8.5 Adjusted Depth (mm)				
	2yr	5Yr	10Yr	100Yr
24hr	81.4	107.5	127.1	198.9
48hr	94.9	125.3	148.5	231.4
72hr	103.1	136.0	161.3	250.1

SSP5-8.5 Adjusted Intensity (mm/hr)				
	2yr	5Yr	10Yr	100Yr
10min	87.9	116.9	138.84	220.8

ge Point: rain garden and dry basin for catchment E

PLAN CHANGE AREA

From	To	Catchment	Catchment Area	C	Peak Flow rate - 10YR ARI	Cum. Flow	Pipe dia	Gradient	Capacity	Percent Capacity	Remaining	Velocity	Check
MH number	MH number	letter	m ²		l/s	l/s	m	%	l/s	%	l/s	m/s	OK

CATCHMENT4-B.2

LINE 4B.2.1-2.7

4.2DS	4B2.1		4742	0.90	164.59	872.55	0.825	0.50	1005.8	86.7%	133.3	1.9	OK
4B2.1	4B2.2		6406	0.90	222.35	707.96	0.825	0.34	829.4	85.4%	121.4	1.6	OK
4B2.2	4B2.3		1737	0.90	60.29	485.62	0.750	0.28	585.6	82.9%	100.0	1.3	OK
4B2.3	4B2.4		1523	0.90	52.86	425.33	0.675	0.38	516.9	82.3%	91.6	1.4	OK
4B2.4	4B2.5		1363	0.90	47.31	152.27	0.600	0.10	194.3	78.4%	42.1	0.7	OK
4B2.5	4B2.6		1288	0.90	44.71	104.96	0.375	0.40	127.3	82.4%	22.3	1.2	OK
4B2.6	4B2.7		1736	0.90	60.25	60.25	0.300	0.40	70.8	85.1%	10.5	1.0	OK

LINE 4B.2.4.1-2.4.3

4B2.4	4B2.4.1		1014	0.90	35.20	220.19	0.525	0.33	248.3	88.7%	28.1	1.1	OK
4B2.4.1	4B2.4.2		1736	0.90	60.25	185.00	0.375	1.10	211.2	87.6%	26.2	1.9	OK
4B2.4.2	4B2.4.3		1798	0.90	62.41	124.74	0.300	1.60	141.6	88.1%	16.9	2.0	OK
4B2.4.3	4B2.4.4		1796	0.90	62.34	62.34	0.300	0.40	70.8	88.1%	8.4	1.0	OK

CATCHMENT4-B3.1-3.3

LINE 4B.3

4.3DS	4B.3.1		1344	0.90	46.65	201.90	0.300	5.00	250.4	80.6%	48.5	3.5	OK
4B.3.1	4B.3.2		1744	0.90	60.53	155.25	0.300	4.50	237.6	65.3%	82.3	3.4	OK
4B.3.2	4B.3.3		2729	0.90	94.72	94.72	0.225	4.50	111.3	85.1%	16.6	2.8	OK

 MAVEN ASSOCIATES		Job Number	Sheet	Rev	Title	Author		Date	Checked
		180010	1	A	SW Pipe Capacity (10yrs)	AF		06/2026	

Rainfall Depth	IA	ARI 10YR	ARI 100YR	Pipe ks factor	1.5	mm (pipes up to 1.0m dia)
NIWA Rainfall Data		91.1	143	mm/hr	0.6	mm (pipes over 1.0m dia)
SSP5-8.5	10	138.84	220.80	mm/hr	0.6	

	C Number	Pipes under this assessment are all concrete
Lots	0.7	
JOAL	0.7	
Roads	0.9	

SSP5-8.5 Adjusted Depth (mm)				
	2yr	5Yr	10Yr	100Yr
24hr	81.4	107.5	127.1	198.9
48hr	94.9	125.3	148.5	231.4
72hr	103.1	136.0	161.3	250.1

SSP5-8.5 Adjusted Intensity (mm/hr)				
	2yr	5Yr	10Yr	100Yr
10min	87.9	116.9	138.84	220.8

ge Point: rain garden and dry basin for catchment E

PLAN CHANGE AREA

From	To	Catchment	Catchment Area	C	Peak Flow rate - 10YR ARI	Cum. Flow	Pipe dia	Gradient	Capacity	Percent Capacity	Remaining	Velocity	Check
MH number	MH number	letter	m ²		l/s	l/s	m	%	l/s	%	l/s	m/s	OK

CATCHMENT 5													
CATCHMENT5-5.1DS-5.1.5													
LINE 5.1													
5.1DS	5.1.1		2831	0.90	98.26	772.07	0.750	0.62	871.6	88.6%	99.5	2.0	OK
5.1.1	5.1.2		6142	0.90	213.18	673.81	0.675	0.82	759.4	88.7%	85.6	2.1	OK
5.1.2	5.1.3		9163	0.90	318.04	460.62	0.600	0.71	518.0	88.9%	57.4	1.8	OK
5.1.3	5.1.4		0	0.90	0.00	142.58	0.450	0.32	162.7	87.6%	20.1	1.0	OK
5.1.4	5.1.5		4108	0.90	142.58	142.58	0.375	0.63	159.8	89.2%	17.2	1.4	OK
CATCHMENT5-5.1DS-5.2.2													
LINE 5.2													
5.1DS	5.2.1		1790	0.90	62.13	279.06	0.375	2.45	315.3	88.5%	36.2	2.9	OK
5.2.1	5.2.2		6250	0.90	216.93	216.93	0.375	1.47	244.2	88.8%	27.3	2.2	OK
CATCHMENT5-5.2DS-5.3.3													
LINE 5.3													
5.2DS	5.3.1		1823	0.90	63.27	630.80	0.675	0.71	706.7	89.3%	75.9	2.0	OK
5.3.1	5.3.2		1797	0.90	62.37	567.53	0.675	0.58	638.7	88.9%	71.2	1.8	OK
5.3.2	5.3.3		14554	0.90	505.16	505.16	0.525	1.70	563.6	89.6%	58.5	2.6	OK
CATCHMENT5-5.2DS-5.4.2													
LINE 5.4													
5.2DS	5.4.1		0	0.90	0.00	592.45	0.600	1.20	673.5	88.0%	81.1	2.4	OK
5.4.1	5.4.2		6308	0.90	218.95	592.45	0.600	1.20	673.5	88.0%	81.1	2.4	OK
5.4.2	5.4.3		10761	0.90	373.50	373.50	0.525	1.00	432.3	86.4%	58.8	2.0	OK
CATCHMENT5-OUT5.5DS-5.5.2													
LINE 5.5													
OUT5.5	5.5.1		12400	0.90	430.39	743.54	0.675	1.00	838.7	88.7%	95.2	2.3	OK
5.5.1	5.5.2		9022	0.90	313.15	313.15	0.450	1.49	351.2	89.2%	38.0	2.2	OK
CATCHMENT5-OUT5.6-5.6.1													
LINE 5.6													
OUT5.6	5.6.1		10511	0.90	364.83	364.83	0.675	0.25	419.2	87.0%	54.4	1.2	OK

 MAVEN ASSOCIATES	Job Number	Sheet	Rev	Title	Author		Date	Checked
	180010	1	A	SW Pipe Capacity (10yrs)	AF		06/2026	

Rainfall Depth	IA	ARI 10YR	ARI 100YR
NIWA Rainfall Data		91.1	143
SSP5-8.5	10	138.84	220.80

	C Number
Lots	0.7
JOAL	0.7
Roads	0.9

Pipes under this assessment are all concrete

Area to fill

SSP5-8.5 Adjusted Depth (mm)				
	2yr	5Yr	10Yr	100Yr
24hr	81.4	107.5	127.1	198.9
48hr	94.9	125.3	148.5	231.4
72hr	103.1	136.0	161.3	250.1

SSP5-8.5 Adjusted Intensity (mm/hr)				
	2yr	5Yr	10Yr	100Yr
10min	87.9	116.9	138.84	220.8

ge Point: rain garden and dry basin for catchment E

PLAN CHANGE AREA

From	To	Catchment	Catchment Area	C	Peak Flow rate - 10YR ARI	Cum. Flow	Pipe dia	Gradient	Capacity	Percent Capacity Remaining	Velocity	Check	
MH number	MH number	letter	m ²		l/s	l/s	m	%	l/s	%	l/s	m/s	OK

 MAVEN ASSOCIATES		Job Number	Sheet	Rev	Title	Author		Date	Checked
		180010	1	A	SW Pipe Capacity (10yrs)	AF		06/2026	

Rainfall Depth	IA	ARI 10YR	ARI 100YR	Pipe ks factor	1.5	mm (pipes up to 1.0m dia)
NIWA Rainfall Data		91.1	143	mm/hr	0.6	mm (pipes over 1.0m dia)
SSP5-8.5	10	138.84	220.80	mm/hr	0.6	

	C Number	Pipes under this assessment are all concrete
Lots	0.7	 Area to fill
JOAL	0.7	
Roads	0.9	

SSP5-8.5 Adjusted Depth (mm)				
	2yr	5Yr	10Yr	100Yr
24hr	81.4	107.5	127.1	198.9
48hr	94.9	125.3	148.5	231.4
72hr	103.1	136.0	161.3	250.1

SSP5-8.5 Adjusted Intensity (mm/hr)				
	2yr	5Yr	10Yr	100Yr
10min	87.9	116.9	138.84	220.8

ge Point: rain garden and dry basin for catchment E

PLAN CHANGE AREA

From	To	Catchment	Catchment Area	C	Peak Flow rate - 10YR ARI	Cum. Flow	Pipe dia	Gradient	Capacity	Percent Capacity	Remaining	Velocity	Check
MH number	MH number	letter	m ²		l/s	l/s	m	%	l/s	%	l/s	m/s	OK

CATCHMENT 6													
CATCHMENT6-6.1DS-6.1.6													
LINE 6.1													
6.1DS	6.1.1		2964	0.90	102.88	831.63	0.750	0.70	926.1	89.8%	94.5	2.1	OK
6.1.1	6.1.2		5800	0.90	201.31	728.75	0.750	0.55	820.9	88.8%	92.1	1.9	OK
6.1.2	6.1.3		8593	0.90	298.26	527.44	0.600	0.95	599.3	88.0%	71.8	2.1	OK
6.1.3	6.1.4		0	0.90	0.00	229.18	0.450	0.80	257.3	89.1%	28.1	1.6	OK
6.1.4	6.1.5		5986	0.90	207.77	229.18	0.450	0.80	257.3	89.1%	28.1	1.6	OK
6.1.5	6.1.6		617	0.90	21.42	21.42	0.300	1.00	111.9	19.1%	90.5	1.6	OK
CATCHMENT6-6.1DS-6.2.3													
LINE 6.2													
6.2DS	6.2.1		1781	0.90	61.82	278.12	0.450	1.20	315.2	88.2%	37.0	2.0	OK
6.2.1	6.2.2		6232	0.90	216.31	216.31	0.375	1.50	246.7	87.7%	30.4	2.2	OK
CATCHMENT6-6.2ds-6.3.11													
LINE 6.3 6.2DS-6.3.10													
6.2DS	6.3.1		0	0.90	0.00	1910.08	0.975	0.93	2128.8	89.7%	218.7	2.9	OK
6.3.1	6.3.2		6056	0.90	210.20	1910.08	0.975	0.93	2128.8	89.7%	218.7	2.9	OK
6.3.2	6.3.3		19651	0.90	682.07	1699.88	0.975	0.74	1898.8	89.5%	199.0	2.5	OK
6.3.3	6.3.4		2553	0.90	88.61	1017.81	0.825	0.65	1146.9	88.7%	129.1	2.1	OK
6.3.4	6.3.5		1161	0.90	40.30	351.60	0.600	0.41	393.6	89.3%	42.0	1.4	OK
6.3.5	6.3.6		1799	0.90	62.44	311.31	0.600	0.32	347.7	89.5%	36.4	1.2	OK
6.3.6	6.3.7		1790	0.90	62.13	248.86	0.525	0.42	280.1	88.8%	31.2	1.3	OK
6.3.7	6.3.8		1797	0.90	62.37	186.73	0.450	0.53	209.4	89.2%	22.7	1.3	OK
6.3.8	6.3.9		1799	0.90	62.44	124.36	0.375	0.48	139.5	89.2%	15.1	1.3	OK
6.3.9	6.3.10		1784	0.90	61.92	61.92	0.300	0.40	70.8	87.5%	8.8	1.0	OK
LINE 6.3 6.3.4-6.3.4.2													
6.3.4	6.3.4.1		8393	0.90	291.31	577.59	0.750	0.35	654.8	88.2%	77.2	1.5	OK
6.3.4.1	6.3.4.2		8248	0.90	286.28	286.28	0.525	0.60	334.8	85.5%	48.5	1.5	OK

 MAVEN ASSOCIATES		Job Number	Sheet	Rev	Title	Author		Date	Checked
		180010	1	A	SW Pipe Capacity (10yrs)	AF		06/2026	

Rainfall Depth	IA	ARI 10YR	ARI 100YR	Pipe ks factor	1.5	mm (pipes up to 1.0m dia)
NIWA Rainfall Data		91.1	143	mm/hr	0.6	mm (pipes over 1.0m dia)
SSP5-8.5	10	138.84	220.80	mm/hr	0.6	

	C Number	
Lots	0.7	
JOAL	0.7	
Roads	0.9	

Pipes under this assessment are all concrete

Area to fill

SSP5-8.5 Adjusted Depth (mm)				
	2yr	5Yr	10Yr	100Yr
24hr	81.4	107.5	127.1	198.9
48hr	94.9	125.3	148.5	231.4
72hr	103.1	136.0	161.3	250.1

SSP5-8.5 Adjusted Intensity (mm/hr)				
	2yr	5Yr	10Yr	100Yr
10min	87.9	116.9	138.84	220.8

ge Point: rain garden and dry basin for catchment E

PLAN CHANGE AREA

From	To	Catchment	Catchment Area	C	Peak Flow rate - 10YR ARI	Cum. Flow	Pipe dia	Gradient	Capacity	Percent Capacity	Remaining	Velocity	Check
MH number	MH number	letter	m ²		l/s	l/s	m	%	l/s	%	l/s	m/s	OK

CATCHMENT6-6.2ds-6.4.3													
LINE 6.4													
6.2ds	6.4.1		2133	0.90	74.03	665.89	0.600	1.50	753.1	88.4%	87.2	2.7	OK
6.4.1	6.4.2		6310	0.90	219.01	591.86	0.600	1.20	673.5	87.9%	81.7	2.4	OK
6.4.2	6.4.3		10742	0.90	372.85	372.85	0.525	1.00	432.3	86.3%	59.4	2.0	OK

CATCHMENT6-out6.5-6.5.2													
LINE 6.5													
out 6.5	6.5.1		4511	0.90	156.57	887.34	0.750	0.80	990.1	89.6%	102.7	2.2	OK
6.5.1	6.5.2		9022	0.90	313.15	365.38	0.525	0.90	410.1	89.1%	44.7	1.9	OK
6.5.1	6.5.1.1		10527	0.90	365.38	365.38	0.600	0.45	412.4	88.6%	47.0	1.5	OK

CATCHMENT6-out6.6-6.6.1													
LINE 6.6													
out 6.6	6.6.1		10496	0.90	364.31	364.31	0.600	0.50	434.7	83.8%	70.4	1.5	OK

CATCHMENT6-out6.7-6.7.4													
LINE 6.7													
OUT 6.7	6.7.1		0	0.90	0.00	1290.31	0.825	1.05	1457.7	88.5%	167.4	2.7	OK
6.7.1	6.7.2		11030	0.90	382.84	1290.31	0.825	1.05	1457.7	88.5%	167.4	2.7	OK
6.7.2	6.7.3		11517	0.90	399.74	907.47	0.825	0.51	1015.9	89.3%	108.4	1.9	OK
6.7.3	6.7.4		14628	0.90	507.72	507.72	0.750	0.27	575.1	88.3%	67.3	1.3	OK

 MAVEN ASSOCIATES		Job Number	Sheet	Rev	Title	Author		Date	Checked
		180010	1	A	SW Pipe Capacity (10yrs)	AF		06/2026	

Rainfall Depth	IA	ARI 10YR	ARI 100YR	Pipe ks factor	1.5	mm (pipes up to 1.0m dia)
NIWA Rainfall Data		91.1	143	mm/hr	0.6	mm (pipes over 1.0m dia)
SSP5-8.5	10	138.84	220.80	mm/hr	0.6	

	C Number	
Lots	0.7	
JOAL	0.7	
Roads	0.9	

Pipes under this assessment are all concrete

Area to fill

SSP5-8.5 Adjusted Depth (mm)				
	2yr	5Yr	10Yr	100Yr
24hr	81.4	107.5	127.1	198.9
48hr	94.9	125.3	148.5	231.4
72hr	103.1	136.0	161.3	250.1

SSP5-8.5 Adjusted Intensity (mm/hr)				
	2yr	5Yr	10Yr	100Yr
10min	87.9	116.9	138.84	220.8

ge Point: rain garden and dry basin for catchment E

PLAN CHANGE AREA

From	To	Catchment	Catchment Area	C	Peak Flow rate - 10YR ARI	Cum. Flow	Pipe dia	Gradient	Capacity	Percent Capacity	Remaining	Velocity	Check
MH number	MH number	letter	m ²		l/s	l/s	m	%	l/s	%	l/s	m/s	OK

CATCHMENT 8													
--------------------	--	--	--	--	--	--	--	--	--	--	--	--	--

CATCHMENT8-out8.1-8.1.8

LINE 8.1 8.1-8.1.9													
OUT 8.1	8.1.1		9218	0.90	319.95	1492.18	0.975	0.57	1666.5	89.5%	174.3	2.2	OK
8.1.1	8.1.2		15692	0.90	544.66	1172.23	0.825	0.85	1311.5	89.4%	139.3	2.5	OK
8.1.2	8.1.3		5408	0.90	187.71	627.58	0.750	0.40	700.0	89.7%	72.4	1.6	OK
8.1.3	8.1.4		1955	0.90	67.86	377.57	0.600	0.47	421.5	89.6%	43.9	1.5	OK
8.1.4	8.1.5		1748	0.90	60.67	309.71	0.525	0.65	348.5	88.9%	38.8	1.6	OK
8.1.5	8.1.6		1797	0.90	62.37	249.04	0.525	0.42	280.1	88.9%	31.1	1.3	OK
8.1.6	8.1.7		1790	0.90	62.13	186.67	0.525	0.24	211.7	88.2%	25.0	1.0	OK
8.1.7	8.1.8		1792	0.90	62.20	124.54	0.375	0.49	140.9	88.4%	16.4	1.3	OK
8.1.8	8.1.9		1796	0.90	62.34	62.34	0.300	0.40	70.8	88.1%	8.4	1.0	OK
LINE 8.1 8.1.3-8.1.3.1													
8.1.3	8.1.3.1		1795	0.90	62.30	62.30	0.300	0.40	70.8	88.0%	8.5	1.0	OK



- NOTES
1. ALL WORKS TO BE IN ACCORDANCE WITH THE WAIKATO DISTRICT COUNCIL/IAWAI STANDARD DETAILS AND THE WAIKATO REGIONAL INFRASTRUCTURE TECHNICAL SPECIFICATION.
 2. COORDINATES IN TERMS OF NZ GEODETIC DATUM MT EDEN 2000. LEVELS IN TERMS OF THE NEW ZEALAND VERTICAL DATUM 2016.
 3. IT IS THE CONTRACTOR'S RESPONSIBILITY TO LOCATE ALL SERVICES THAT MAY BE AFFECTED BY THEIR OPERATIONS.
 4. ALL CATCHMENT AREAS SHOWN ON THIS PLAN ARE IN SQUARE METRES (m²).

- LEGEND
- PR SUB-CATCHMENT BOUNDARY
 - PR MODIFIED STREAM
 - PR 12.5m WIDE CONVEYANCE CHANNEL
 - PR WETLAND
 - PR RAINGARDEN
 - PR BERMLANDSCAPE
 - PR LOT SHEET FLOW
 - PR ROADING OLFP DIRECTION
 - PR ROADING CREST/HIGH PT
 - PR ROADING SAG/LOW PT

A	RESOURCE CONSENT	AF	06/2026
Rev	Description	By	Date
		By	Date
Survey	MAVEN		10/2025
Design	AF		07/2026
Drawn	AF		07/2026
Checked	KY		07/2026

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Project
PROPOSED INDUSTRIAL DEVELOPMENT AT TE KOWHAI EAST FOR TE KOWHAI EAST LIMITED PARTNERSHIP

Title
PR STORMWATER SUB-CATCHMENT PLAN PRIMARY 10-YEAR ARI

Project no.	180010
Scale	1:7500 @ A3
Cad file	180010-C481 SUB-CATCHMENT PLAN.DWG
Drawing no.	C481
Rev	A

RESOURCE CONSENT

APPENDIX C.4 – SW CULVERTS

 MAVEN ASSOCIATES					Job Number	Sheet	Rev	Job Title: Te Kowhai East			Author	Date	Checked
					180010	1	A	Culvert Sizing Summary Table			KY	18/05/2026	RW
Culvert	Fish Passage	Type	Internal Height (m)	Internal Width (m)	Q5 (m³/s)	Headwater (m)	Q5 Surcharge?	Q100 (m³/s)	Headwater (m)	Q100 Surcharge?	Height to Top of Culvert (m)	Overtopping?	
M.1*	Yes	Box	1.50*	3.00	***DESIGNED VIA TE MIRO WATER HYDRAULIC TUFLOW MODEL								
M.2*	Yes	Box	1.50*	2.00	***DESIGNED VIA TE MIRO WATER HYDRAULIC TUFLOW MODEL								
O.1*	Yes	Box	1.00*	1.50	1.14	0.52	No Surcharge	2.13	0.88	Surcharging!	3.00	No Overtopping	
1.1	No	Box	1.00	2.00	2.56	0.79	No Surcharge	4.79	1.25	Surcharging!	1.50	No Overtopping	
1.2	No	Box	1.00	2.00	2.19	0.73	No Surcharge	4.10	1.12	Surcharging!	1.50	No Overtopping	
1.3	No	Box	1.00	1.50	1.05	0.52	No Surcharge	1.97	0.82	No Surcharge	1.50	No Overtopping	
1.4	No	Box	1.00	1.50	0.53	0.33	No Surcharge	0.98	0.52	No Surcharge	1.50	No Overtopping	
1.5	No	Box	1.00	1.50	0.54	0.33	No Surcharge	1.02	0.51	No Surcharge	1.50	No Overtopping	
2.1	No	Box	1.00	1.50	1.82	0.78	No Surcharge	3.41	1.30	Surcharging!	1.50	No Overtopping	
2.2	No	Box	1.00	1.50	1.00	0.51	No Surcharge	1.87	0.79	No Surcharge	1.50	No Overtopping	
3.1	No	Box	1.00	1.50	1.00	0.50	No Surcharge	1.87	0.80	No Surcharge	1.50	No Overtopping	
3.2	No	Box	1.00	1.50	0.51	0.32	No Surcharge	0.95	0.48	No Surcharge	1.50	No Overtopping	
4.1	No	Box	1.00	1.50	1.70	0.69	No Surcharge	3.19	1.18	Surcharging!	1.50	No Overtopping	
4.2	No	Box	1.00	1.50	0.44	0.30	No Surcharge	0.82	0.43	No Surcharge	1.50	No Overtopping	
4.3	No	Box	1.00	1.50	1.10	0.54	No Surcharge	2.06	0.82	No Surcharge	1.50	No Overtopping	
4.4	No	Box	1.00	1.50	0.79	0.42	No Surcharge	1.48	0.65	No Surcharge	1.50	No Overtopping	
5.1	No	Box	1.00	1.50	2.10	0.85	No Surcharge	3.94	1.45	Surcharging!	1.50	No Overtopping	
5.2	No	Box	1.00	1.50	0.89	0.47	No Surcharge	1.67	0.70	No Surcharge	1.50	No Overtopping	
6.1	No	Box	1.00	2.00	2.63	0.82	No Surcharge	4.92	1.40	Surcharging!	1.50	No Overtopping	
6.2	No	Box	1.00	1.50	1.10	0.53	No Surcharge	2.07	0.83	No Surcharge	1.50	No Overtopping	

M1* & M2*

Actual Prefab Size 1.5m Internal Height, designed based on min. 25% fish passage embedment as 1.0m Clear Height

O.1*

Actual Prefab Size 1.0m Internal Height, designed based on min. 25% fish passage embedment as 0.8m Clear Height

 Maven Associates			Job Number 180010	Sheet 1	Rev: A Ver: 1.0
Job Title Calc Title Catchment		Te Kowhai East Culvert Sizing ALL	Author KY	Date 26/03/2026	Checked RW

1. Runoff Curve Number (CN) and initial Abstraction (Ia)
CALCS to WRC TR2020/06

For Culvert Calculations only and assumes a 90% Impervious Catchment to calculate Storage and Initial Abstraction

Soil name and classification	ID	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1 ha	Product of CN x area
D		10% Pervious Post-Dev	80	0.10	8.00
D		90% Impervious Post-Dev	98	0.90	88.20
			0		0.00
Totals =				1.00	96.20

CN (weighted) = $\frac{\text{total product}}{\text{total area}} = \frac{96.20}{1.000} =$
96.2

S= $\frac{(1000}{CN} - 10) * 25.4}{96.2} =$
10.0 mm

Ia= $0.05 * S = 0.05 * 10.0 =$
0.50 mm

2. Time of Concentration

For Culvert Calculations only and assumes a 10minute minimum Time of Concentration. Subject to further EPA at detailed design

Tc= 0.167 hours

Worksheet 1: Runoff Parameters and Time of Concentration

	Maven Associates	Job Number 180010	Sheet 2	Rev: A Ver: 1.0
Job Title Calc Title	Te Kowhai East Culvert Sizing Post-Dev Peak Flows	Author KY	Date 26/03/2026	Checked RW

1. Data

Runoff curve number CN= 96.2 (from worksheet 1) Subject to detailed design

Initial abstraction Ia= 0.5 mm (from worksheet 1) Subject to detailed design

Time of concentration tc= 0.17 hrs (from worksheet 1) Subject to detailed design

2. Calculate storage, $S = (1000/CN - 10)/25.4$ = 10 mm

3. Average recurrence interval, ARI

4. 24 hour rainfall depth, P24

5. Compute $c^* = P24 - 2Ia/P24 - 2Ia + 2S$

6. Specific peak flow rate q^*

	2 (yr)	5 (yr)	10 (yr)	100
24 hour rainfall depth, P24 (mm)	81.40	107.45	127.10	198.91
Compute c^*	0.800	0.841	0.863	0.908
Specific peak flow rate q^*	0.160	0.163	0.164	0.165



Empirical Calcs
(Designed via NZBC Excel)

Attenuation Culverts
(Designed via TUFLOW software)

Worksheet 2: Graphical Peak Flow Rate

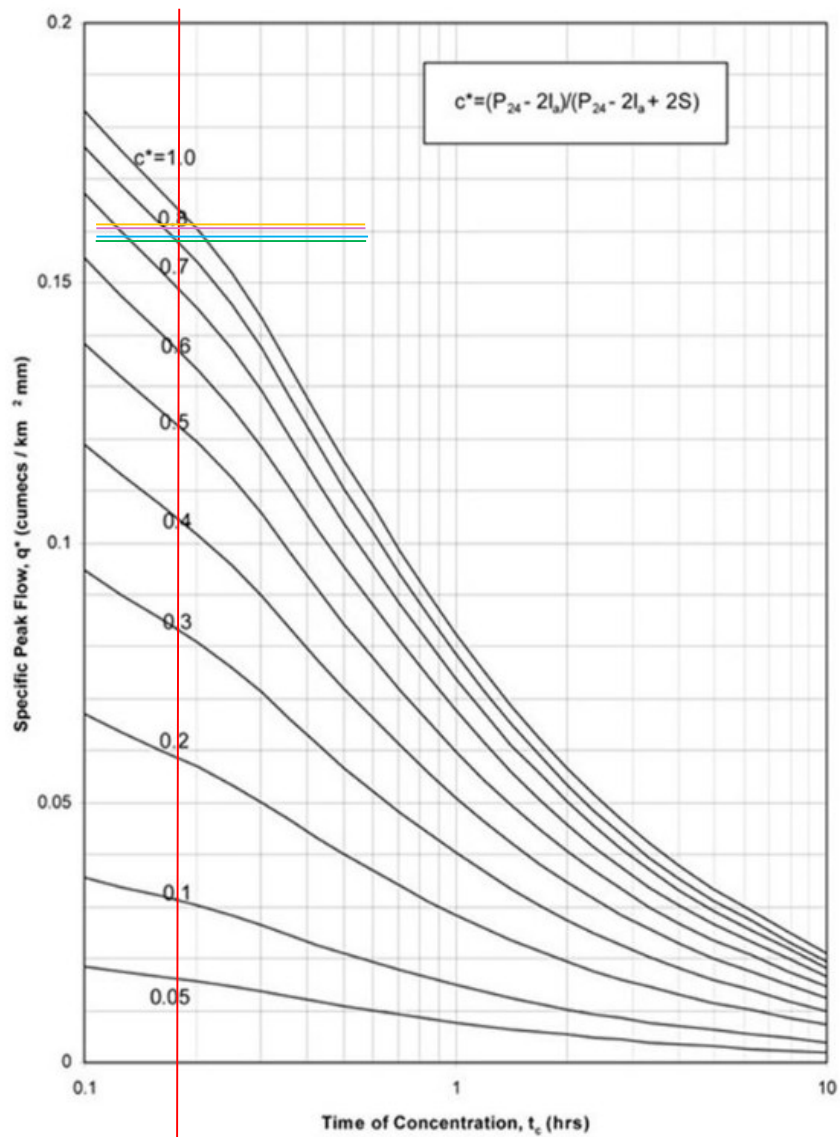


Figure 8-1: Determining the specific flow rate³²

 Maven Associates		Job Number 180010	Sheet 3	Rev: A Ver: 1.0
Job Title Calc Title	Te Kowhai East Culvert Sizing CULVERT 0.1 - BOX	Author KY	Date 26/03/2026	Checked RW

CULVERT 0.1 - BOX

Catchment Area	A=	0.065	km ² (100ha =1km ²)
Design Case		Local/Collector	
Average recurrence interval, ARI		2 (yr)	5 (yr)
Peak flow rate, $q_p=q^*A^*P_{24}$		0.847 m ³ /s	1.138 m ³ /s
		10 (yr)	100 (yr)
		1.355 m ³ /s	2.133 m ³ /s
Internal Height of Culvert	D=	0.8	m
Internal Base of Culvert	B=	1.5	m
Datum Invert to Top of Culvert	=	3	m
		Fish passage, used clear height in calculation	
Discharge per m of box width	Q/B=		m ³ /sm
		0.759	m ³ /sm
			m ³ /sm
		1.422221	m ³ /sm
Headwater depth/Culvert Height	Hw/D=		
From graph. Use Scale (1)		0.655	
			1.100
Headwater	Hw=		m
Surcharge?		0.524	m
Surcharge Height			m
Compliance Check		No Surcharge	
		N/A	m
		OK	
		0.880	m
		Surcharging!	
		0.080	m
		Check Overtopping	
Datum Invert to Top of Culvert	m=	3.000	m
Overtopping?		3.000	m
Compliance Check		No Overtopping	
		OK	
		3.000	m
		No Overtopping	
		OK	

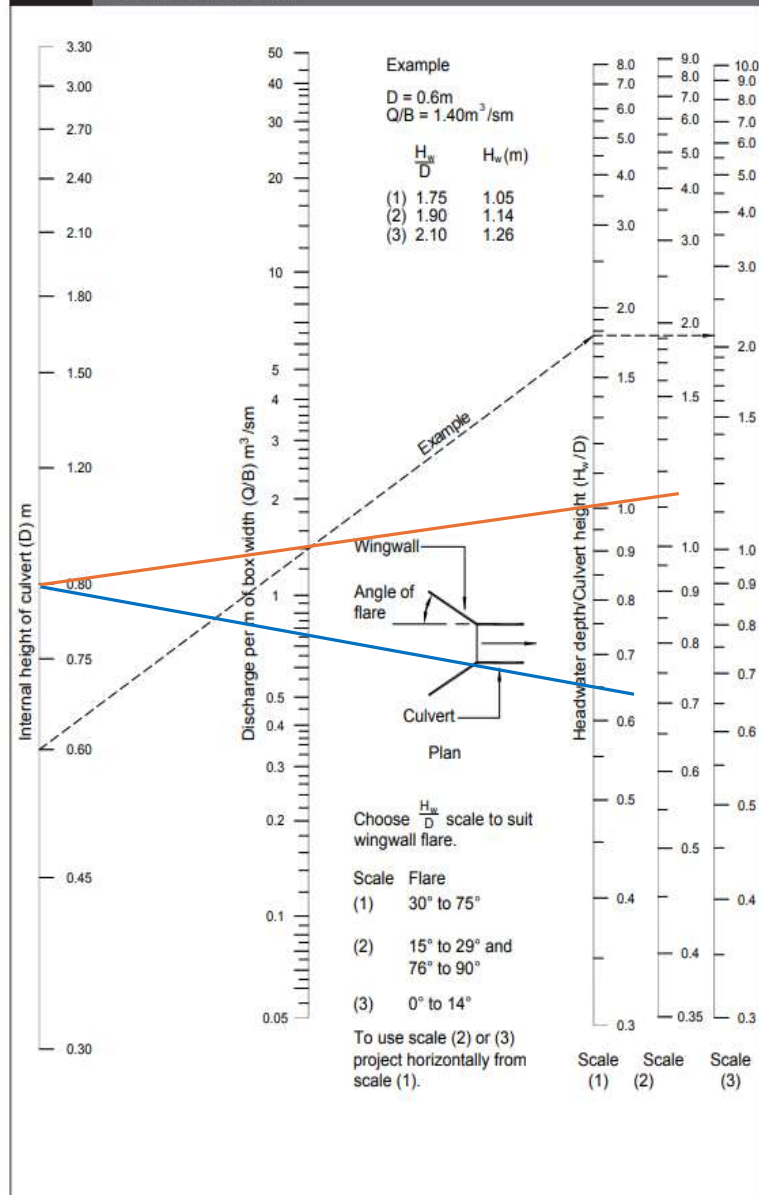
4.2.12.1 Design storm

Culverts must be designed as a minimum to accommodate storms as per the table below. The design must not cause any increase in upstream water levels that will result in flooding on neighbouring properties.

Table 60: Culvert design storms

DESIGN CASE	ARI YEAR STORM TO PASS WITHOUT SURCHARGE (DESIGN FLOW)	ARI YEAR STORM NOT OVERTOPPING STRUCTURE (PEAK FLOW)
Driveway or private way	2	NA
Pedestrian or cycleway walk	5	10
Local or collector road	5	100
Arterial roads and railways	20	100

Figure 7: Headwater Depth for Box Culverts
Paragraphs 3.2.3 b) and 4.1.4 b)



 Maven Associates		Job Number 180010	Sheet 3	Rev: A Ver: 1.0
Job Title Calc Title	Te Kowhai East Culvert Sizing CULVERT 1.1 - BOX	Author KY	Date 26/03/2026	Checked RW

CULVERT 1.1 - BOX

Catchment Area	A=	0.146	km ² (100ha =1km ²)
Design Case		Local/Collector	
Average recurrence interval, ARI		2 (yr)	5 (yr)
Peak flow rate, $q_p=q^*A^*P_{24}$		1.902 m ³ /s	2.557 m ³ /s
		10 (yr)	100 (yr)
		3.043 m ³ /s	4.792 m ³ /s
Internal Height of Culvert	D=	1	m
Internal Base of Culvert	B=	2	m
Datum Invert to Top of Culvert	=	1.5	m
Discharge per m of box width	Q/B=		m ³ /sm
		1.2786	m ³ /sm
			m ³ /sm
		2.395895	m ³ /sm
Headwater depth/Culvert Height	Hw/D=		
From graph. Use Scale (1)		0.790	
			1.210
Headwater	Hw=		m
Surcharge?		0.790	m
Surcharge Height		No Surcharge	
Compliance Check		N/A	m
		OK	
			1.250
			Surcharging!
			0.250 m
			Check Overtopping
Datum Invert to Top of Culvert	m=	1.500	m
Overtopping?		1.500	m
Compliance Check		No Overtopping	
		OK	
			No Overtopping
			OK

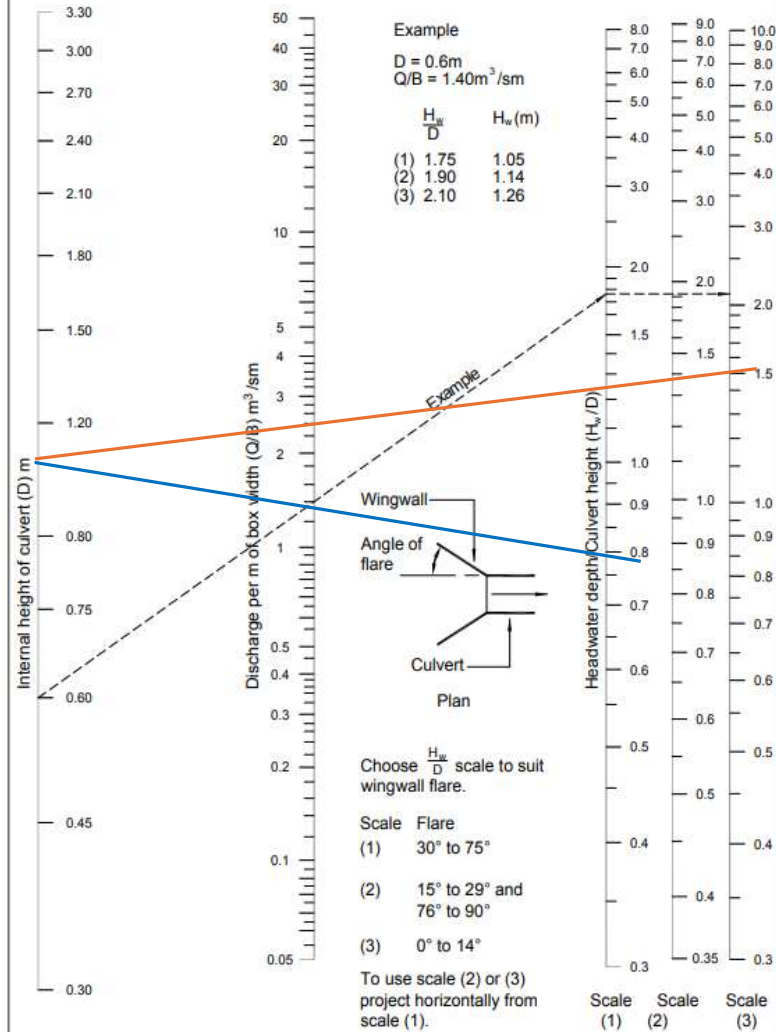
4.2.12.1 Design storm

Culverts must be designed as a minimum to accommodate storms as per the table below. The design must not cause any increase in upstream water levels that will result in flooding on neighbouring properties.

Table 60: Culvert design storms

DESIGN CASE	ARI YEAR STORM TO PASS WITHOUT SURCHARGE (DESIGN FLOW)	ARI YEAR STORM NOT OVERTOPPING STRUCTURE (PEAK FLOW)
Driveway or private way	2	NA
Pedestrian or cycleway walk	5	10
Local or collector road	5	100
Arterial roads and railways	20	100

Figure 7: Headwater Depth for Box Culverts
Paragraphs 3.2.3 b) and 4.1.4 b)



 Maven Associates		Job Number 180010	Sheet 4	Rev: A Ver: 1.0
Job Title Calc Title	Te Kowhai East Culvert Sizing CULVERT 1.2 - BOX	Author KY	Date 26/03/2026	Checked RW

CULVERT 1.2 - BOX

Catchment Area	A=	0.125	km ² (100ha =1km ²)
Design Case		Local/Collector	
Average recurrence interval, ARI		2 (yr)	5 (yr)
Peak flow rate, $q_p=q \cdot A \cdot P_{24}$		1.628 m ³ /s	2.189 m ³ /s
		10 (yr)	100 (yr)
		2.606 m ³ /s	4.103 m ³ /s
Internal Height of Culvert	D=	1	m
Internal Base of Culvert	B=	2	m
Datum Invert to Top of Culvert	=	1.5	m
Discharge per m of box width	Q/B=		m ³ /sm
		1.0947	m ³ /sm
			m ³ /sm
		2.05128	m ³ /sm
Headwater depth/Culvert Height	Hw/D=		
From graph. Use Scale (1)		0.730	
			1.120
Headwater	Hw=		m
Surcharge?		0.730	m
Surcharge Height		No Surcharge	
Compliance Check		N/A	m
		OK	
			1.120
			Surcharging!
			0.120 m
			Check Overtopping
Datum Invert to Top of Culvert	m=	1.500	m
Overtopping?		1.500	m
Compliance Check		No Overtopping	
		OK	
			No Overtopping
			OK

4.2.12.1 Design storm

Culverts must be designed as a minimum to accommodate storms as per the table below. The design must not cause any increase in upstream water levels that will result in flooding on neighbouring properties.

Table 60: Culvert design storms

DESIGN CASE	ARI YEAR STORM TO PASS WITHOUT SURCHARGE (DESIGN FLOW)	ARI YEAR STORM TO PASS SURCHARGE OVERTOPPING STRUCTURE (PEAK FLOW)	NOT
Driveway or private way	2	NA	
Pedestrian or cycleway walk	5	10	
Local or collector road	5	100	
Arterial roads and railways	20	100	

Figure 7: Headwater Depth for Box Culverts
Paragraphs 3.2.3 b) and 4.1.4 b)

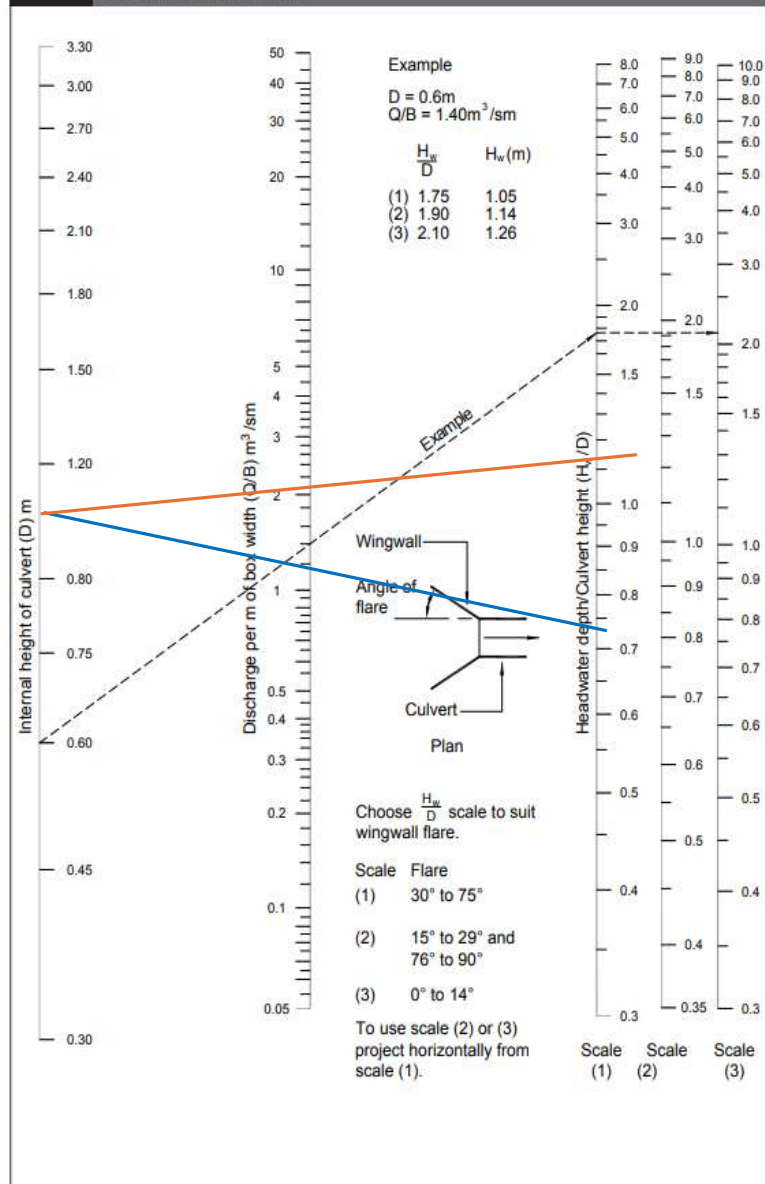
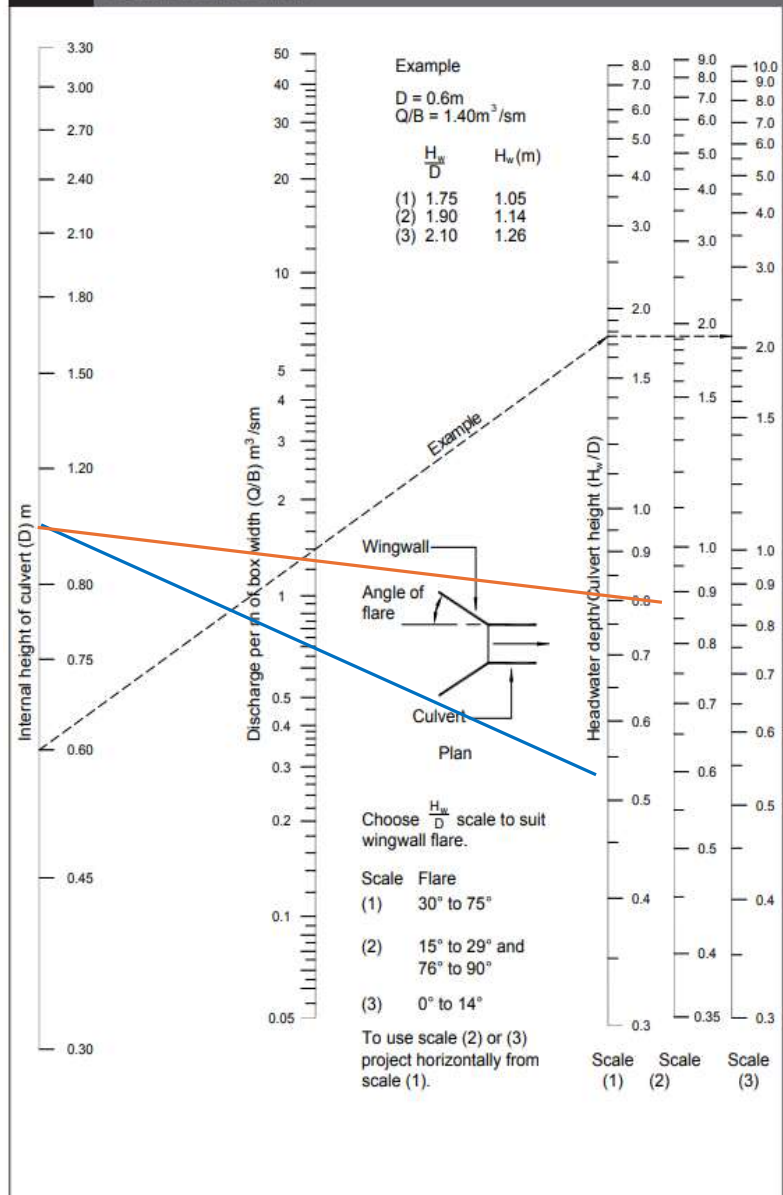


Figure 7: Headwater Depth for Box Culverts
Paragraphs 3.2.3 b) and 4.1.4 b)



	Maven Associates	Job Number 180010	Sheet 3	Rev: A Ver: 1.0
Job Title Calc Title	Te Kowhai East Culvert Sizing CULVERT 1.4 - BOX	Author KY	Date 26/03/2026	Checked RW

CULVERT 1.4 - BOX

Catchment Area	A=	0.03	km ² (100ha =1km ²)
Design Case		Local/Collector	
Average recurrence interval, ARI		2 (yr)	5 (yr)
Peak flow rate, $q_p = q \cdot A \cdot P_{24}$		0.391 m ³ /s	0.525 m ³ /s
		10 (yr)	100 (yr)
		0.625 m ³ /s	0.985 m ³ /s
Internal Height of Culvert	D=	1	m
Internal Base of Culvert	B=	1.5	m
Datum Invert to Top of Culvert	=	1.5	m
Discharge per m of box width	Q/B=		0.3503 m ³ /sm
			0.65641 m ³ /sm
Headwater depth/Culvert Height	Hw/D=		0.330
From graph. Use Scale (1)			0.520
Headwater	Hw=		0.330 m
Surcharge?		No Surcharge	No Surcharge
Surcharge Height		N/A	N/A
Compliance Check		OK	OK
Datum Invert to Top of Culvert	m=	1.500	m
Overtopping?		No Overtopping	No Overtopping
Compliance Check		OK	OK

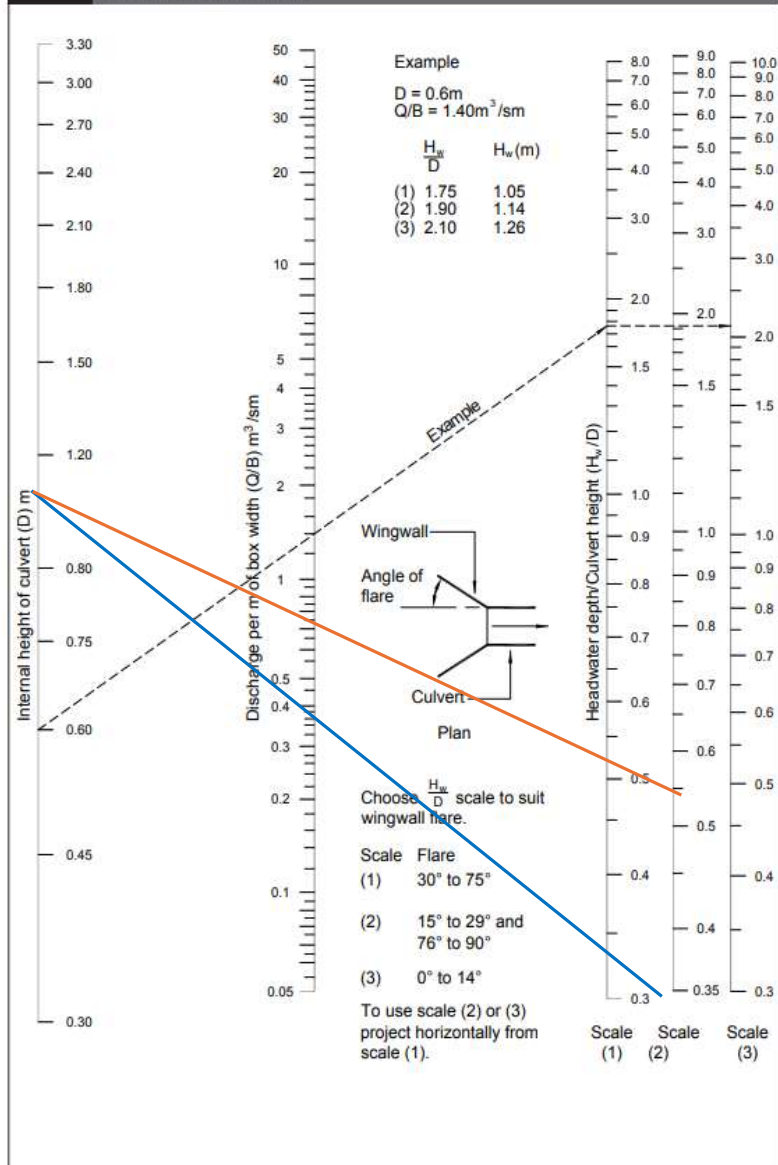
4.2.12.1 Design storm

Culverts must be designed as a minimum to accommodate storms as per the table below. The design must not cause any increase in upstream water levels that will result in flooding on neighbouring properties.

Table 60: Culvert design storms

DESIGN CASE	ARI YEAR STORM TO PASS WITHOUT SURCHARGE (DESIGN FLOW)	ARI YEAR STORM TO PASS OVERTOPPING STRUCTURE (PEAK FLOW)
Driveway or private way	2	NA
Pedestrian or cycleway walk	5	10
Local or collector road	5	100
Arterial roads and railways	20	100

Figure 7: Headwater Depth for Box Culverts
Paragraphs 3.2.3 b) and 4.1.4 b)



	Maven Associates	Job Number 180010	Sheet 4	Rev: A Ver: 1.0
Job Title Calc Title	Te Kowhai East Culvert Sizing CULVERT 1.5 - BOX	Author KY	Date 26/03/2026	Checked RW

CULVERT 1.5 - BOX

Catchment Area	A=	0.031	km ² (100ha =1km ²)	
Design Case		Local/Collector		
Average recurrence interval, ARI		2 (yr)	5 (yr)	10 (yr)
Peak flow rate, $q_p=q \cdot A \cdot P_{24}$		0.404 m ³ /s	0.543 m ³ /s	0.646 m ³ /s
Internal Height of Culvert	D=	1	m	
Internal Base of Culvert	B=	1.5	m	
Datum Invert to Top of Culvert	=	1.5	m	
Discharge per m of box width	Q/B=		0.362 m ³ /sm	0.67829 m ³ /sm
Headwater depth/Culvert Height	Hw/D=		0.330	0.510
From graph. Use Scale (1)				
Headwater	Hw=		0.330 m	0.510 m
Surcharge?			No Surcharge	No Surcharge
Surcharge Height			N/A m	N/A m
Compliance Check			OK	OK
Datum Invert to Top of Culvert	m=	1.500	m	1.500
Overtopping?			No Overtopping	No Overtopping
Compliance Check			OK	OK

4.2.12.1 Design storm

Culverts must be designed as a minimum to accommodate storms as per the table below. The design must not cause any increase in upstream water levels that will result in flooding on neighbouring properties.

Table 60: Culvert design storms

DESIGN CASE	ARI YEAR STORM TO PASS WITHOUT SURCHARGE (DESIGN FLOW)	ARI YEAR STORM NOT OVERTOPPING STRUCTURE (PEAK FLOW)
Driveway or private way	2	NA
Pedestrian or cycleway walk	5	10
Local or collector road	5	100
Arterial roads and railways	20	100

Figure 7: Headwater Depth for Box Culverts
Paragraphs 3.2.3 b) and 4.1.4 b)

Example

$D = 0.6\text{m}$
 $Q/B = 1.40\text{m}^3/\text{sm}$

$\frac{H_w}{D}$	$H_w(\text{m})$
(1) 1.75	1.05
(2) 1.90	1.14
(3) 2.10	1.26

Plan

Wingwall
Angle of flare
Culvert

Internal height of culvert (D) m

Discharge per m of box width (Q/B) m^3/sm

Headwater depth/Culvert height (H_w/D)

Choose $\frac{H_w}{D}$ scale to suit wingwall flare.

Scale Flare

(1) 30° to 75°
(2) 15° to 29° and 76° to 90°
(3) 0° to 14°

To use scale (2) or (3) project horizontally from scale (1).

Scale (1) **Scale (2)** **Scale (3)**

 Maven Associates M A E N	Job Number 180010	Sheet 5	Rev: A Ver: 1.0
Job Title Calc Title Te Kowhai East Culvert Sizing CULVERT 2.1 - BOX	Author KY	Date 26/03/2026	Checked RW

CULVERT 2.1 - BOX

Catchment Area **A=** 0.104 km²(100ha =1km²)

Design Case Local/Collector

Average recurrence interval, ARI	2 (yr)	5 (yr)	10 (yr)	100 (yr)
Peak flow rate, $q_p = q \cdot A \cdot P_{24}$	1.355 m ³ /s	1.822 m ³ /s	2.168 m ³ /s	3.413 m ³ /s

Internal Height of Culvert	D=	1 m	
Internal Base of Culvert	B=	1.5 m	
Datum Invert to Top of Culvert	=	1.5 m	

Discharge per m of box width	Q/B=	 m ³ /sm	1.2144 m ³ /sm	 m ³ /sm	2.275553 m ³ /sm
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Headwater depth/Culvert Height From graph. Use Scale (1)	Hw/D=		0.780		1.300
---	--------------	---	--	---	--

Headwater	Hw=	 m	0.780 m	 m	1.300 m
Surcharge?			No Surcharge		Surcharging!
Surcharge Height		m	N/A m	m	0.300 m
Compliance Check			OK		Check Overtopping

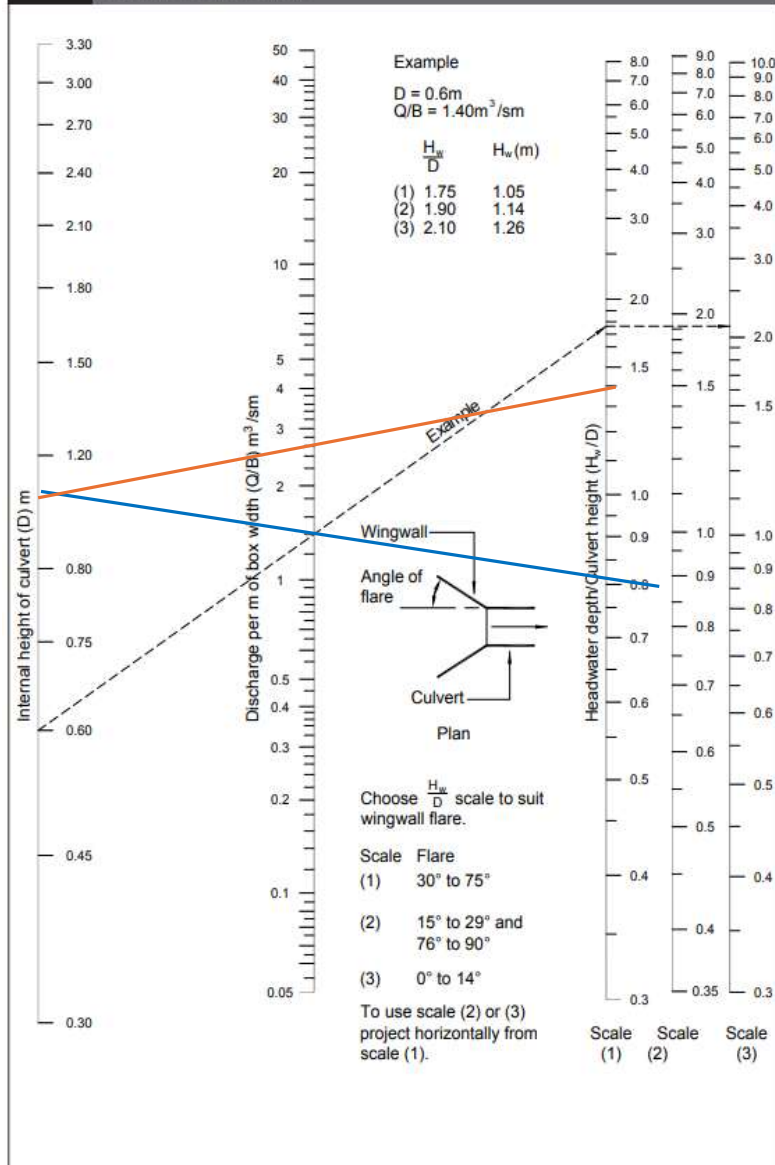
Datum Invert to Top of Culvert	m=	1.500 m	1.500 m	 m	1.500
Overtopping?			No Overtopping		No Overtopping
Compliance Check			OK		OK

4.2.12.1 Design storm

Culverts must be designed as a minimum to accommodate storms as per the table below. The design must not cause any increase in upstream water levels that will result in flooding on neighbouring properties.

Table 60: Culvert design storms			
DESIGN CASE	ARI YEAR STORM TO PASS WITHOUT SURCHARGE (DESIGN FLOW)	ARI YEAR STORM NOT OVERTOPPING STRUCTURE (PEAK FLOW)	
Driveway or private way	2	NA	
Pedestrian or cycleway walk	5	10	
Local or collector road	5	100	
Arterial roads and railways	20	100	

Figure 7: Headwater Depth for Box Culverts
Paragraphs 3.2.3 b) and 4.1.4 b)



	Maven Associates	Job Number 180010	Sheet 3	Rev: A Ver: 1.0
Job Title Calc Title	Te Kowhai East Culvert Sizing CULVERT 2.2 - BOX	Author KY	Date 26/03/2026	Checked RW

CULVERT 2.2 - BOX

Catchment Area	A=	0.057 km ² (100ha =1km ²)			
Design Case		Local/Collector			
Average recurrence interval, ARI		2 (yr)	5 (yr)	10 (yr)	100 (yr)
Peak flow rate, $q_p = q \cdot A \cdot P_{24}$		0.742 m ³ /s	0.998 m ³ /s	1.188 m ³ /s	1.871 m ³ /s
Internal Height of Culvert	D=	1 m			
Internal Base of Culvert	B=	1.5 m			
Datum Invert to Top of Culvert	=	1.5 m			
Discharge per m of box width	Q/B=		0.6656 m ³ /sm		1.247178 m ³ /sm
Headwater depth/Culvert Height	Hw/D=		0.510		0.790
From graph. Use Scale (1)					
Headwater	Hw=		0.510 m		0.790 m
Surcharge?			No Surcharge		No Surcharge
Surcharge Height			N/A m		N/A m
Compliance Check			OK		OK
Datum Invert to Top of Culvert	m=	1.500 m	1.500 m		1.500 m
Overtopping?			No Overtopping		No Overtopping
Compliance Check			OK		OK

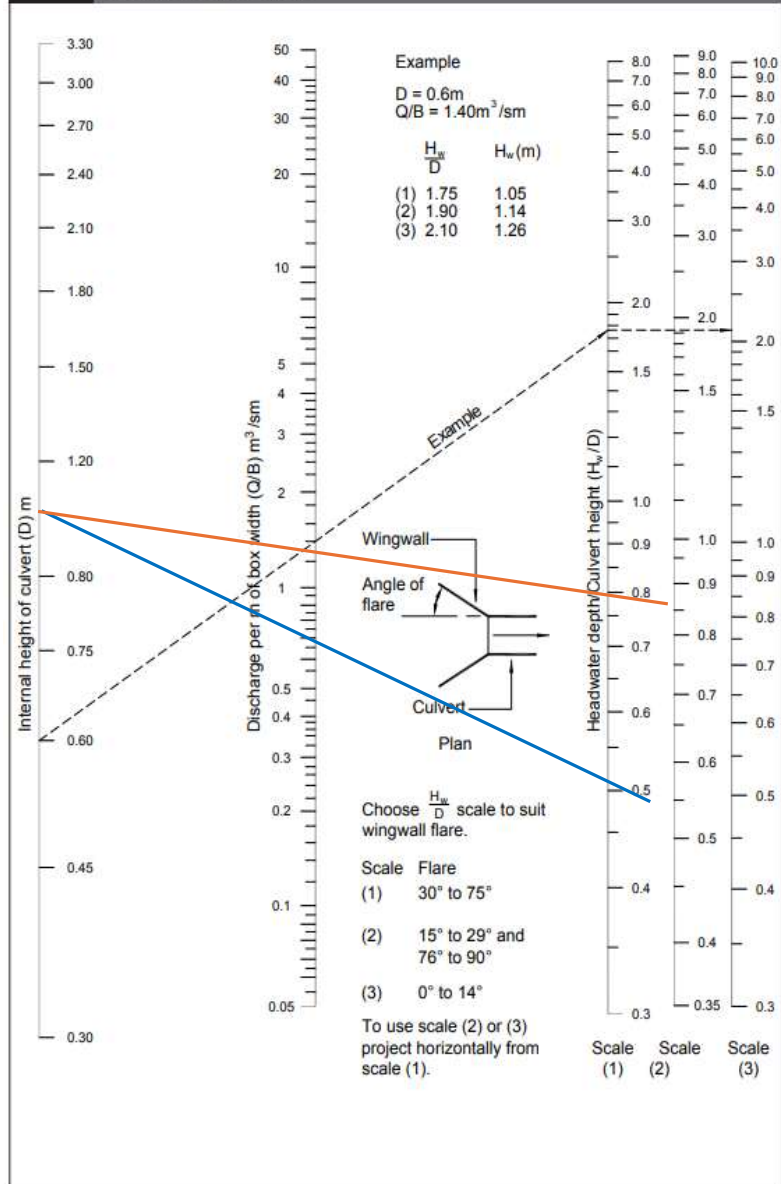
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Table 60: Culvert design storms

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Driveway or private way	2	NA
Pedestrian or cycleway walk	5	10
Local or collector road	5	100
Arterial roads and railways	20	100

Figure 7: Headwater Depth for Box Culverts
Paragraphs 3.2.3 b) and 4.1.4 b)



	Maven Associates	Job Number 180010	Sheet 4	Rev: A Ver: 1.0
Job Title Calc Title	Te Kowhai East Culvert Sizing CULVERT 3.1 - BOX	Author KY	Date 26/03/2026	Checked RW

CULVERT 3.1 - BOX

Catchment Area	A=	0.057 km ² (100ha =1km ²)			
Design Case		Local/Collector			
Average recurrence interval, ARI		2 (yr)	5 (yr)	10 (yr)	100 (yr)
Peak flow rate, $q_p = q \cdot A \cdot P_{24}$		0.742 m ³ /s	0.998 m ³ /s	1.188 m ³ /s	1.871 m ³ /s
Internal Height of Culvert	D=	1 m			
Internal Base of Culvert	B=	1.5 m			
Datum Invert to Top of Culvert	=	1.5 m			
Discharge per m of box width	Q/B=		0.6656 m ³ /sm		1.247178 m ³ /sm
Headwater depth/Culvert Height	Hw/D=		0.500		0.800
From graph. Use Scale (1)					
Headwater	Hw=		0.500 m		0.800 m
Surcharge?			No Surcharge		No Surcharge
Surcharge Height			N/A m		N/A m
Compliance Check			OK		OK
Datum Invert to Top of Culvert	m=	1.500 m	1.500 m		1.500 m
Overtopping?			No Overtopping		No Overtopping
Compliance Check			OK		OK

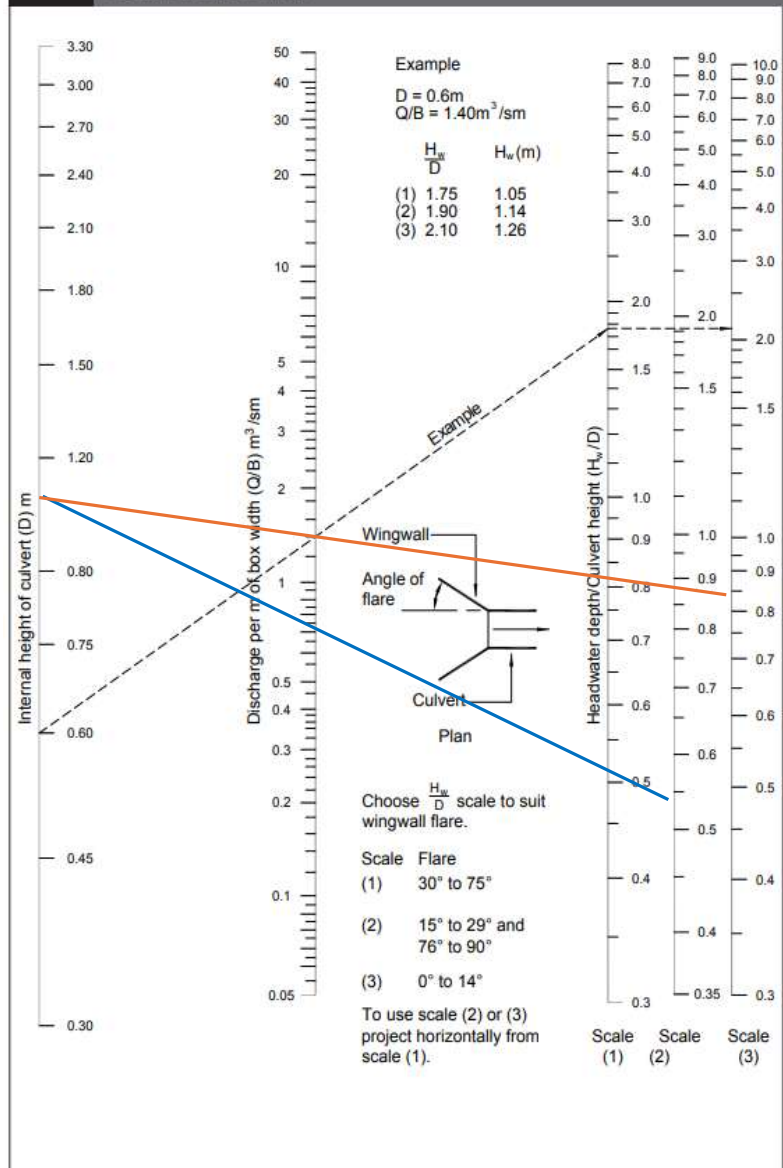
4.2.12.1 Design storm

Culverts must be designed as a minimum to accommodate storms as per the table below. The design must not cause any increase in upstream water levels that will result in flooding on neighbouring properties.

Table 60: Culvert design storms

DESIGN CASE	ARI YEAR STORM TO PASS WITHOUT SURCHARGE (DESIGN FLOW)	ARI YEAR STORM NOT OVERTOPPING STRUCTURE (PEAK FLOW)
Driveway or private way	2	NA
Pedestrian or cycleway walk	5	10
Local or collector road	5	100
Arterial roads and railways	20	100

Figure 7: Headwater Depth for Box Culverts
Paragraphs 3.2.3 b) and 4.1.4 b)



	Maven Associates	Job Number 180010	Sheet 5	Rev: A Ver: 1.0
Job Title Calc Title	Te Kowhai East Culvert Sizing CULVERT 3.2 - BOX	Author KY	Date 26/03/2026	Checked RW

CULVERT 3.2 - BOX

Catchment Area	A=	0.029 km ² (100ha = 1km ²)			
Design Case		Local/Collector			
Average recurrence interval, ARI		2 (yr)	5 (yr)	10 (yr)	100 (yr)
Peak flow rate, $q_p = q \cdot A \cdot P_{24}$		0.378 m ³ /s	0.508 m ³ /s	0.605 m ³ /s	0.952 m ³ /s
Internal Height of Culvert	D=	1 m			
Internal Base of Culvert	B=	1.5 m			
Datum Invert to Top of Culvert	=	1.5 m			
Discharge per m of box width	Q/B=		0.3386 m ³ /sm		0.634529 m ³ /sm
Headwater depth/Culvert Height	Hw/D=		0.320		0.480
From graph. Use Scale (1)					
Headwater	Hw=		0.320 m		0.480 m
Surcharge?			No Surcharge		No Surcharge
Surcharge Height			N/A m		N/A m
Compliance Check			OK		OK
Datum Invert to Top of Culvert	m=	1.500 m	1.500 m		1.500 m
Overtopping?			No Overtopping		No Overtopping
Compliance Check			OK		OK

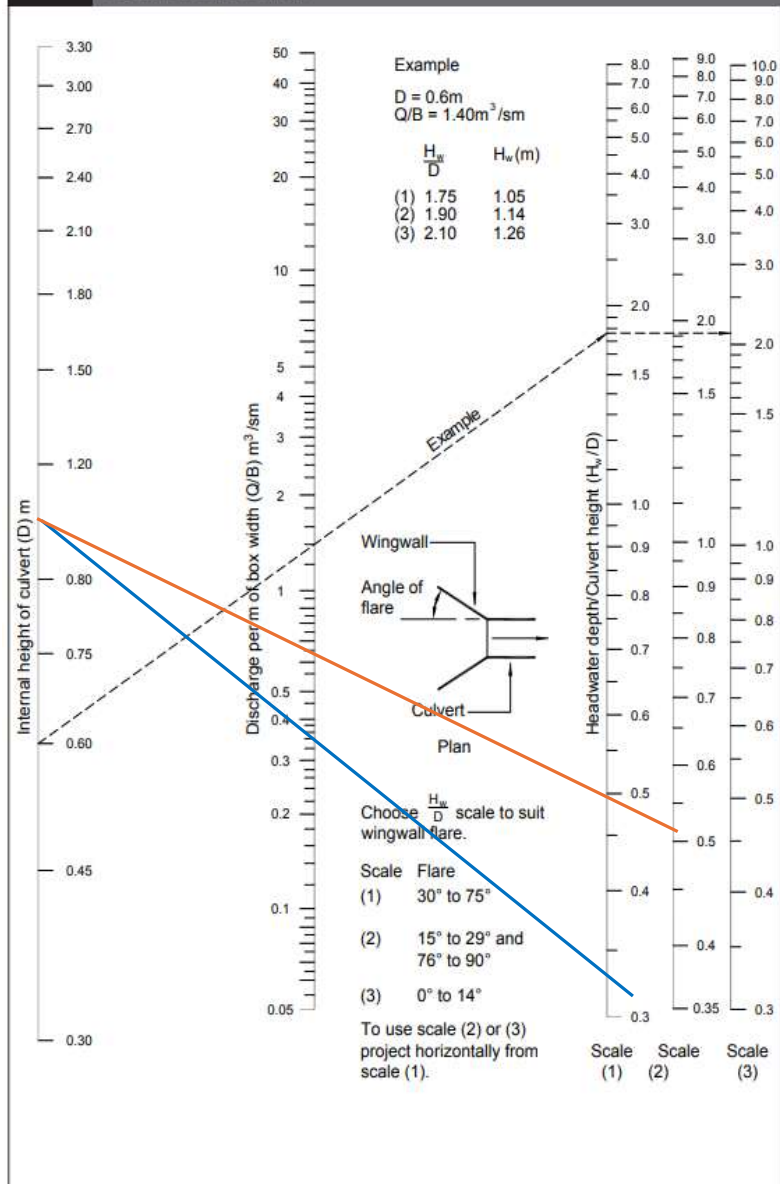
4.2.12.1 Design storm

Culverts must be designed as a minimum to accommodate storms as per the table below. The design must not cause any increase in upstream water levels that will result in flooding on neighbouring properties.

Table 60: Culvert design storms

DESIGN CASE	ARI YEAR STORM TO PASS WITHOUT SURCHARGE (DESIGN FLOW)	ARI YEAR STORM NOT OVERTOPPING STRUCTURE (PEAK FLOW)
Driveway or private way	2	NA
Pedestrian or cycleway walk	5	10
Local or collector road	5	100
Arterial roads and railways	20	100

Figure 7: Headwater Depth for Box Culverts
Paragraphs 3.2.3 b) and 4.1.4 b)



 Maven Associates		Job Number 180010	Sheet 7	Rev: A Ver: 1.0
Job Title Calc Title	Te Kowhai East Culvert Sizing CULVERT 4.1 - BOX	Author KY	Date 26/03/2026	Checked RW

CULVERT 4.1 - BOX

Catchment Area	A=	0.0973 km ² (100ha =1km ²)			
Design Case		Local/Collector			
Average recurrence interval, ARI		2 (yr)	5 (yr)	10 (yr)	100 (yr)
Peak flow rate, $q_p=q \cdot A \cdot P_{24}$		1.267 m ³ /s	1.704 m ³ /s	2.028 m ³ /s	3.193 m ³ /s
Internal Height of Culvert	D=	1 m			
Internal Base of Culvert	B=	1.5 m			
Datum Invert to Top of Culvert	=	1.5 m			
Discharge per m of box width	Q/B=		1.1361 m ³ /sm		2.128955 m ³ /sm
Headwater depth/Culvert Height	Hw/D=		0.690		1.180
From graph. Use Scale (1)					
Headwater	Hw=		0.690 m		1.180 m
Surcharge?			No Surcharge		Surcharging!
Surcharge Height			N/A m		0.180 m
Compliance Check			OK		Check Overtopping
Datum Invert to Top of Culvert	m=	1.500 m	1.500 m		1.500 m
Overtopping?			No Overtopping		No Overtopping
Compliance Check			OK		OK

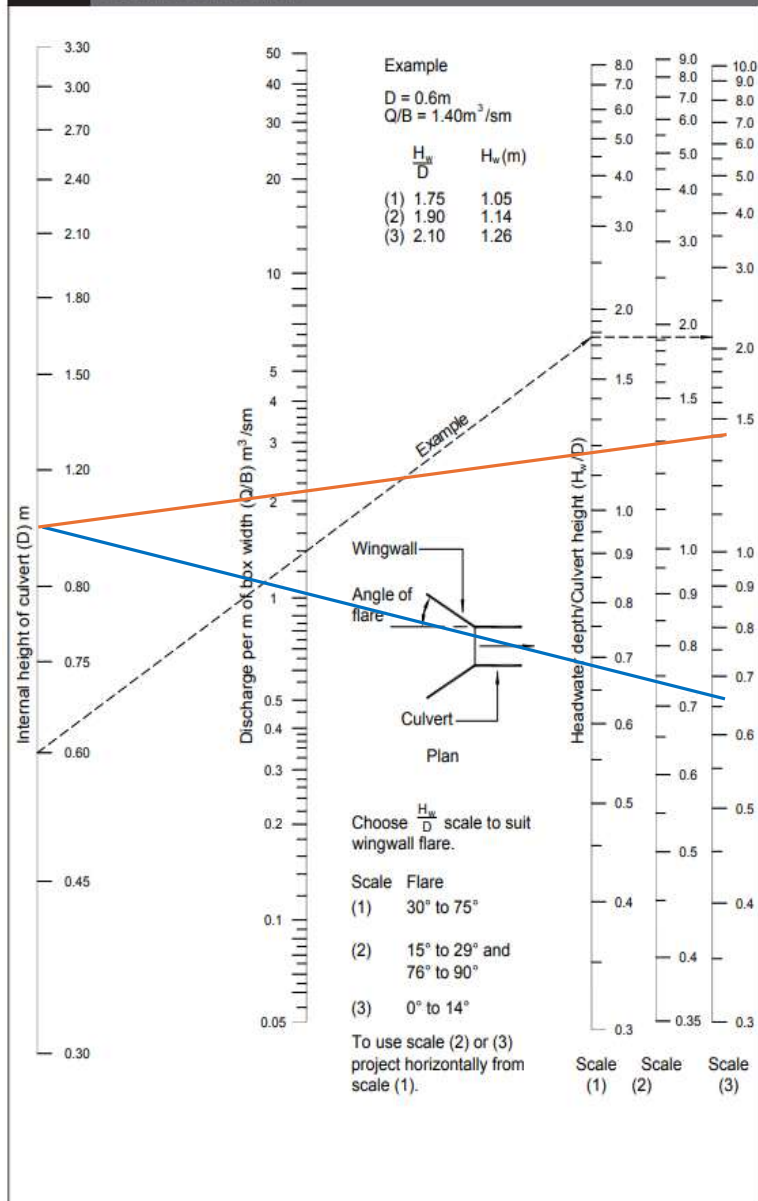
4.2.12.1 Design storm

Culverts must be designed as a minimum to accommodate storms as per the table below. The design must not cause any increase in upstream water levels that will result in flooding on neighbouring properties.

Table 60: Culvert design storms

DESIGN CASE	ARI YEAR STORM TO PASS WITHOUT SURCHARGE (DESIGN FLOW)	ARI YEAR STORM NOT OVERTOPPING STRUCTURE (PEAK FLOW)
Driveway or private way	2	NA
Pedestrian or cycleway walk	5	10
Local or collector road	5	100
Arterial roads and railways	20	100

Figure 7: Headwater Depth for Box Culverts
Paragraphs 3.2.3 b) and 4.1.4 b)



	Maven Associates	Job Number 180010	Sheet 8	Rev: A Ver: 1.0
Job Title Calc Title	Te Kowhai East Culvert Sizing CULVERT 4.2 - BOX	Author KY	Date 26/03/2026	Checked RW

CULVERT 4.2 - BOX

Catchment Area	A=	0.025	km ² (100ha =1km ²)
Design Case		Local/Collector	
Average recurrence interval, ARI		2 (yr)	5 (yr)
Peak flow rate, $q_p=q \cdot A \cdot P_{24}$		0.326 m ³ /s	0.438 m ³ /s
		10 (yr)	100 (yr)
		0.521 m ³ /s	0.821 m ³ /s
Internal Height of Culvert	D=	1	m
Internal Base of Culvert	B=	1.5	m
Datum Invert to Top of Culvert	=	1.5	m
Discharge per m of box width	Q/B=		m ³ /sm
		0.2919	m ³ /sm
			m ³ /sm
		0.547008	m ³ /sm
Headwater depth/Culvert Height	Hw/D=		
From graph. Use Scale (1)		0.300	0.430
Headwater	Hw=		m
Surcharge?		0.300	m
Surcharge Height		No Surcharge	No Surcharge
Compliance Check		N/A	N/A
		OK	OK
Datum Invert to Top of Culvert	m=	1.500	m
Overtopping?		1.500	m
Compliance Check		No Overtopping	No Overtopping
		OK	OK

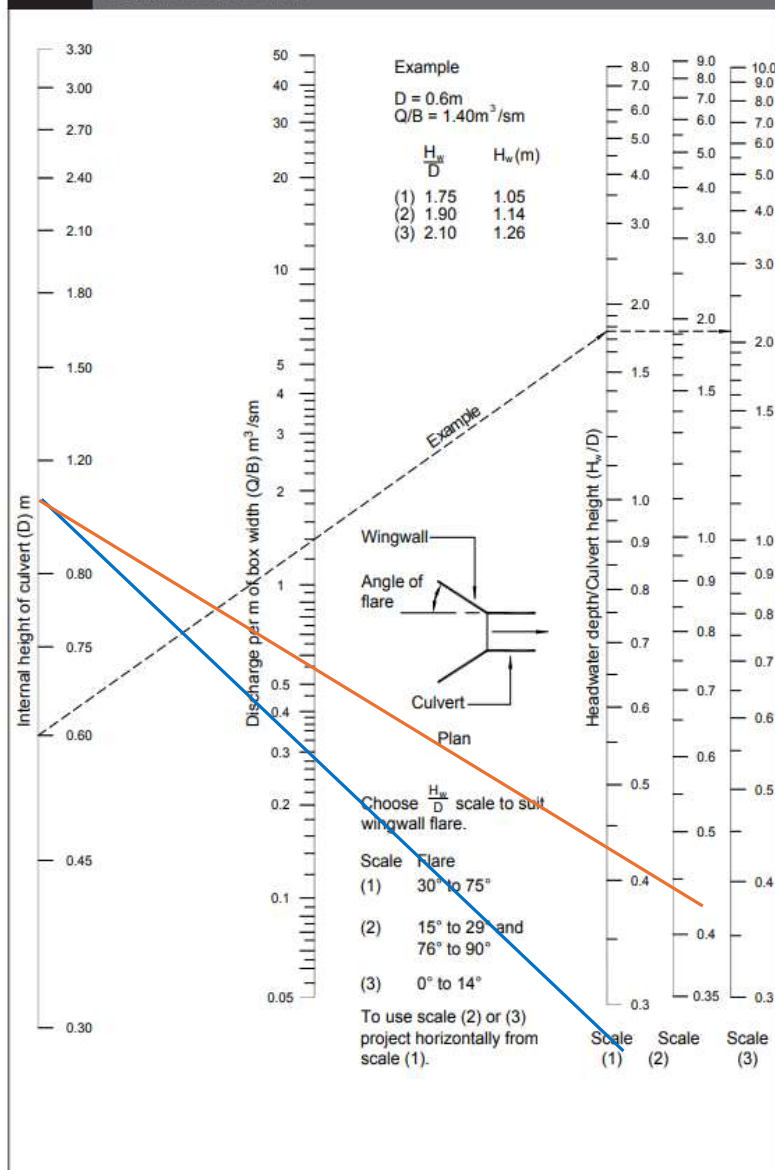
4.2.12.1 Design storm

Culverts must be designed as a minimum to accommodate storms as per the table below. The design must not cause any increase in upstream water levels that will result in flooding on neighbouring properties.

Table 60: Culvert design storms

DESIGN CASE	ARI YEAR STORM TO PASS WITHOUT SURCHARGE (DESIGN FLOW)	ARI YEAR STORM NOT OVERTOPPING STRUCTURE (PEAK FLOW)
Driveway or private way	2	NA
Pedestrian or cycleway walk	5	10
Local or collector road	5	100
Arterial roads and railways	20	100

Figure 7: Headwater Depth for Box Culverts
Paragraphs 3.2.3 b) and 4.1.4 b)



	Maven Associates	Job Number 180010	Sheet 9	Rev: A Ver: 1.0
Job Title Calc Title	Te Kowhai East Culvert Sizing CULVERT 4.3 - BOX	Author KY	Date 26/03/2026	Checked RW

CULVERT 4.3 - BOX

Catchment Area A= 0.0627 km²(100ha =1km²)

Design Case Local/Collector

Average recurrence interval, ARI	2 (yr)	5 (yr)	10 (yr)	100 (yr)
Peak flow rate, $q_p = q \cdot A \cdot P_{24}$	0.817 m ³ /s	1.098 m ³ /s	1.307 m ³ /s	2.058 m ³ /s

Internal Height of Culvert D= 1 m
 Internal Base of Culvert B= 1.5 m
 Datum Invert to Top of Culvert = 1.5 m

Discharge per m of box width Q/B= m³/sm 0.7321 m³/sm m³/sm 1.371896 m³/sm

Headwater depth/Culvert Height Hw/D= 0.540 0.820
 From graph. Use Scale (1)

Headwater	Hw= m	0.540 m	m	0.820 m
Surcharge?		No Surcharge		No Surcharge
Surcharge Height	m	N/A m	m	N/A m
Compliance Check		OK		OK

Datum Invert to Top of Culvert	m= 1.500 m	1.500 m	m	1.500
Overtopping?		No Overtopping		No Overtopping
Compliance Check		OK		OK

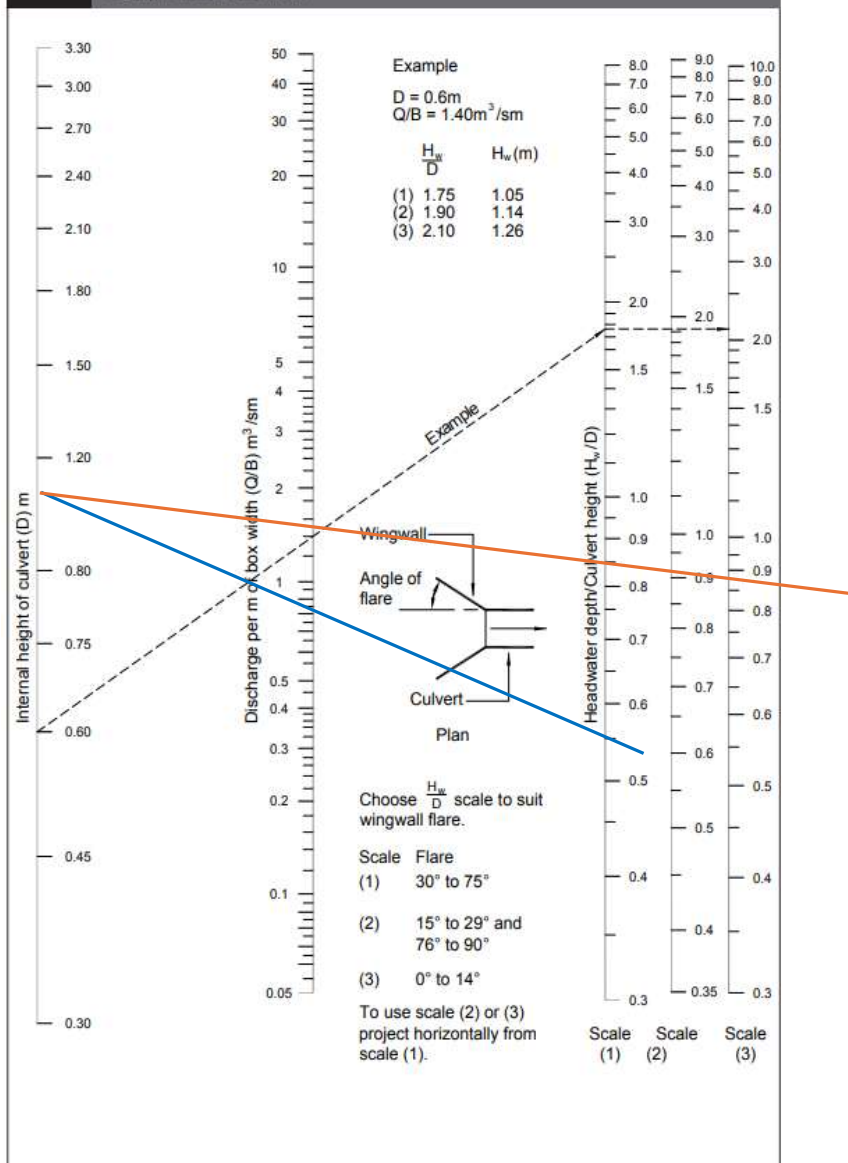
4.2.12.1 Design storm

Culverts must be designed as a minimum to accommodate storms as per the table below. The design must not cause any increase in upstream water levels that will result in flooding on neighbouring properties.

Table 60: Culvert design storms

DESIGN CASE	ARI YEAR STORM TO PASS WITHOUT SURCHARGE (DESIGN FLOW)	ARI YEAR STORM TO PASS OVERTOPPING STRUCTURE (PEAK FLOW)	NOT
Driveway or private way	2	NA	
Pedestrian or cycleway walk	5	10	
Local or collector road	5	100	
Arterial roads and railways	20	100	

Figure 7: Headwater Depth for Box Culverts
Paragraphs 3.2.3 bi and 4.1.4 bi



	Maven Associates	Job Number 180010	Sheet 6	Rev: A Ver: 1.0
Job Title Calc Title	Te Kowhai East Culvert Sizing CULVERT 4.4 - BOX	Author KY	Date 26/03/2026	Checked RW

CULVERT 4.4 - BOX

Catchment Area	A=	0.045	km ² (100ha =1km ²)
Design Case		Local/Collector	
Average recurrence interval, ARI		2 (yr)	5 (yr)
Peak flow rate, $q_p=q^*A^*P_{24}$		0.586 m ³ /s	0.788 m ³ /s
		10 (yr)	100 (yr)
		0.938 m ³ /s	1.477 m ³ /s
Internal Height of Culvert	D=	1	m
Internal Base of Culvert	B=	1.5	m
Datum Invert to Top of Culvert	=	1.5	m
Discharge per m of box width	Q/B=		0.5254 m ³ /sm
			0.984614 m ³ /sm
Headwater depth/Culvert Height	Hw/D=		0.420
From graph. Use Scale (1)			0.650
Headwater	Hw=		0.420 m
Surcharge?		No Surcharge	No Surcharge
Surcharge Height		N/A	N/A
Compliance Check		OK	OK
Datum Invert to Top of Culvert	m=	1.500	m
Overtopping?		No Overtopping	No Overtopping
Compliance Check		OK	OK

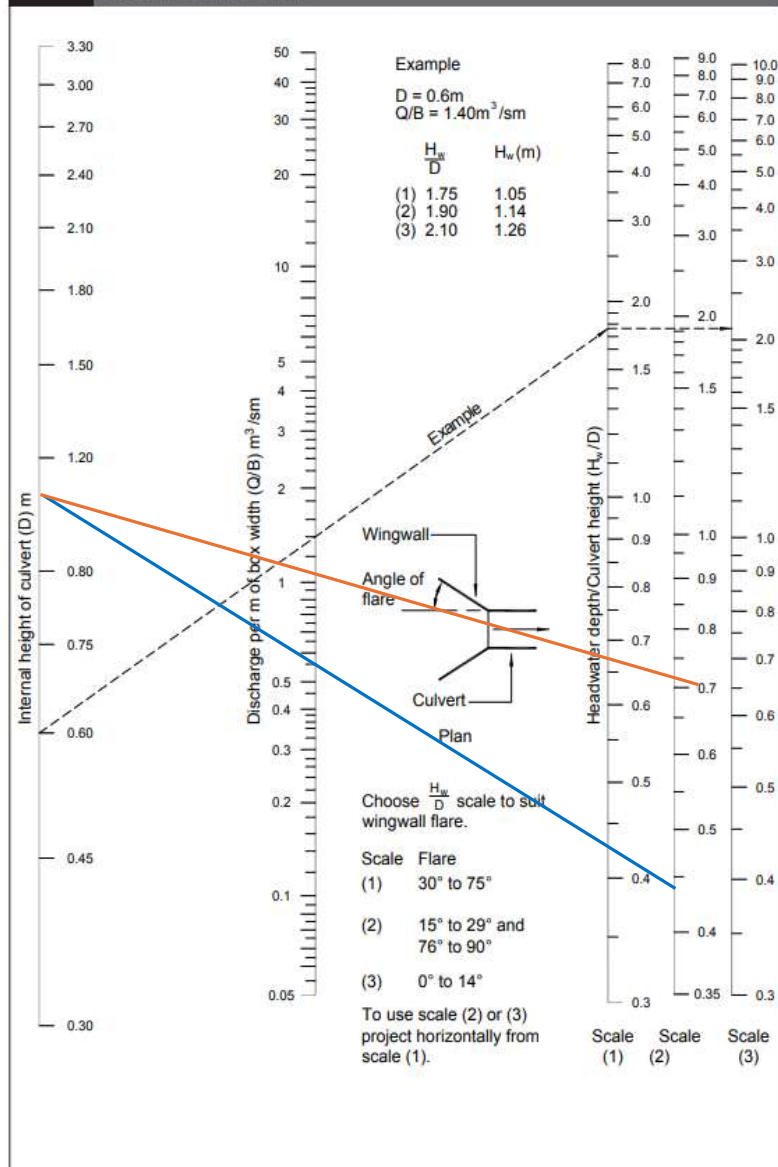
4.2.12.1 Design storm

Culverts must be designed as a minimum to accommodate storms as per the table below. The design must not cause any increase in upstream water levels that will result in flooding on neighbouring properties.

Table 60: Culvert design storms

DESIGN CASE	ARI YEAR STORM TO PASS WITHOUT SURCHARGE (DESIGN FLOW)	ARI YEAR STORM NOT OVERTOPPING STRUCTURE (PEAK FLOW)
Driveway or private way	2	NA
Pedestrian or cycleway walk	5	10
Local or collector road	5	100
Arterial roads and railways	20	100

Figure 7: Headwater Depth for Box Culverts
Paragraphs 3.2.3 b) and 4.1.4 b)



 Maven Associates		Job Number 180010	Sheet 10	Rev: A Ver: 1.0
Job Title Calc Title	Te Kowhai East Culvert Sizing CULVERT 5.1 - BOX	Author KY	Date 26/03/2026	Checked RW

CULVERT 5.1 - BOX

Catchment Area	A=	0.12	km ² (100ha =1km ²)	
Design Case		Local/Collector		
Average recurrence interval, ARI		2 (yr)	5 (yr)	10 (yr)
Peak flow rate, $q_p=q^*A^*P_{24}$		1.563 m ³ /s	2.102 m ³ /s	2.501 m ³ /s
Internal Height of Culvert	D=	1	m	
Internal Base of Culvert	B=	1.5	m	
Datum Invert to Top of Culvert	=	1.5	m	
Discharge per m of box width	Q/B=		1.4012 m ³ /sm	2.625638 m ³ /sm
Headwater depth/Culvert Height	Hw/D=		0.850	1.450
From graph. Use Scale (1)				
Headwater	Hw=		0.850 m	1.450 m
Surcharge?			No Surcharge	Surcharging!
Surcharge Height			N/A m	0.450 m
Compliance Check			OK	Check Overtopping
Datum Invert to Top of Culvert	m=	1.500	m	1.500
Overtopping?			No Overtopping	No Overtopping
Compliance Check			OK	OK

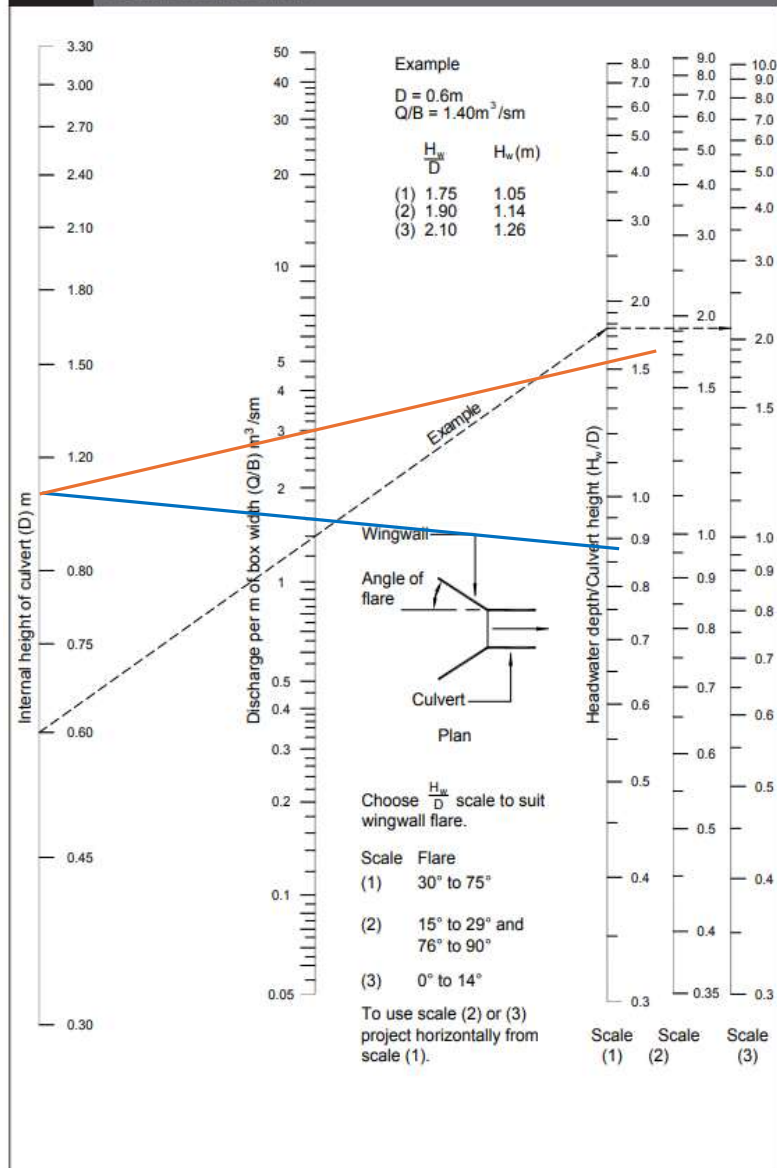
4.2.12.1 Design storm

Culverts must be designed as a minimum to accommodate storms as per the table below. The design must not cause any increase in upstream water levels that will result in flooding on neighbouring properties.

Table 60: Culvert design storms

DESIGN CASE	ARI YEAR STORM TO PASS WITHOUT SURCHARGE (DESIGN FLOW)	ARI YEAR STORM TO PASS SURCHARGE OVERTOPPING (PEAK FLOW)	NOT STRUCTURE
Driveway or private way	2	NA	
Pedestrian or cycleway walk	5	10	
Local or collector road	5	100	
Arterial roads and railways	20	100	

Figure 7: Headwater Depth for Box Culverts
Paragraphs 3.2.3 b) and 4.1.4 b)



	Maven Associates	Job Number 180010	Sheet 11	Rev: A Ver: 1.0
Job Title Calc Title	Te Kowhai East Culvert Sizing CULVERT 5.2 - BOX	Author KY	Date 26/03/2026	Checked RW

CULVERT 5.2 - BOX

Catchment Area	A=	0.051 km ² (100ha = 1km ²)			
Design Case		Local/Collector			
Average recurrence interval, ARI		2 (yr)	5 (yr)	10 (yr)	100 (yr)
Peak flow rate, $q_p = q \cdot A \cdot P_{24}$		0.664 m ³ /s	0.893 m ³ /s	1.063 m ³ /s	1.674 m ³ /s
Internal Height of Culvert	D=	1 m			
Internal Base of Culvert	B=	1.5 m			
Datum Invert to Top of Culvert	=	1.5 m			
Discharge per m of box width	Q/B=		0.5955 m ³ /sm		1.115896 m ³ /sm
Headwater depth/Culvert Height	Hw/D=		0.470		0.700
From graph. Use Scale (1)					
Headwater	Hw=		0.470 m		0.700 m
Surcharge?			No Surcharge		No Surcharge
Surcharge Height			N/A m		N/A m
Compliance Check			OK		OK
Datum Invert to Top of Culvert	m=	1.500 m	1.500 m		1.500 m
Overtopping?			No Overtopping		No Overtopping
Compliance Check			OK		OK

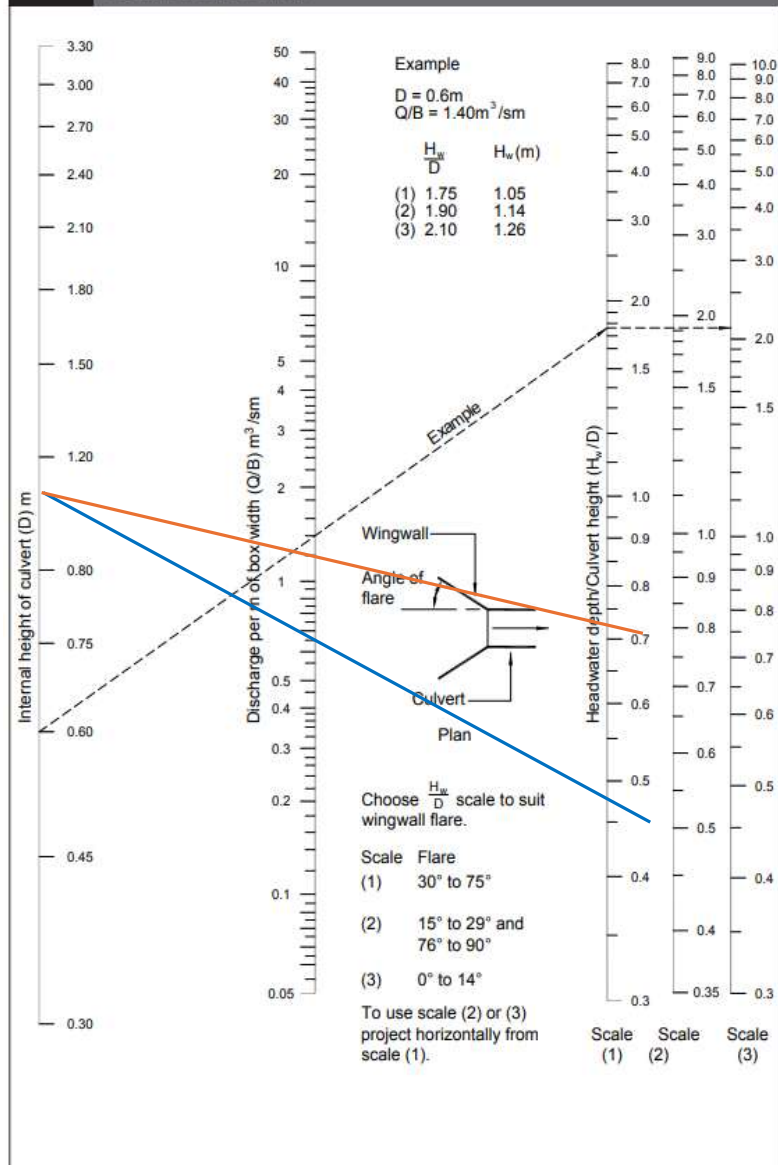
4.2.12.1 Design storm

Culverts must be designed as a minimum to accommodate storms as per the table below. The design must not cause any increase in upstream water levels that will result in flooding on neighbouring properties.

Table 60: Culvert design storms

DESIGN CASE	ARI YEAR STORM TO PASS WITHOUT SURCHARGE (DESIGN FLOW)	ARI YEAR STORM NOT OVERTOPPING STRUCTURE (PEAK FLOW)
Driveway or private way	2	NA
Pedestrian or cycleway walk	5	10
Local or collector road	5	100
Arterial roads and railways	20	100

Figure 7: Headwater Depth for Box Culverts
Paragraphs 3.2.3 b) and 4.1.4 b)



 Maven Associates	Job Number 180010	Sheet 12	Rev: A Ver: 1.0
	Author KY	Date 26/03/2026	Checked RW
Job Title Calc Title Te Kowhai East Culvert Sizing CULVERT 6.1 - BOX			

CULVERT 6.1 - BOX

Catchment Area	A=	0.15	km ² (100ha =1km ²)	
Design Case		Local/Collector		
Average recurrence interval, ARI		2 (yr)	5 (yr)	10 (yr)
Peak flow rate, $q_p=q \cdot A \cdot P_{24}$		1.954 m ³ /s	2.627 m ³ /s	3.127 m ³ /s
Internal Height of Culvert	D=	1	m	
Internal Base of Culvert	B=	2	m	
Datum Invert to Top of Culvert	=	1.5	m	
Discharge per m of box width	Q/B=		1.3136 m ³ /sm	2.461536 m ³ /sm
Headwater depth/Culvert Height	Hw/D=		0.820	1.400
From graph. Use Scale (1)				
Headwater	Hw=		0.820 m	1.400 m
Surcharge?			No Surcharge	Surcharging!
Surcharge Height			N/A m	0.400 m
Compliance Check			OK	Check Overtopping
Datum Invert to Top of Culvert	m=	1.500	m	1.500
Overtopping?			No Overtopping	No Overtopping
Compliance Check			OK	OK

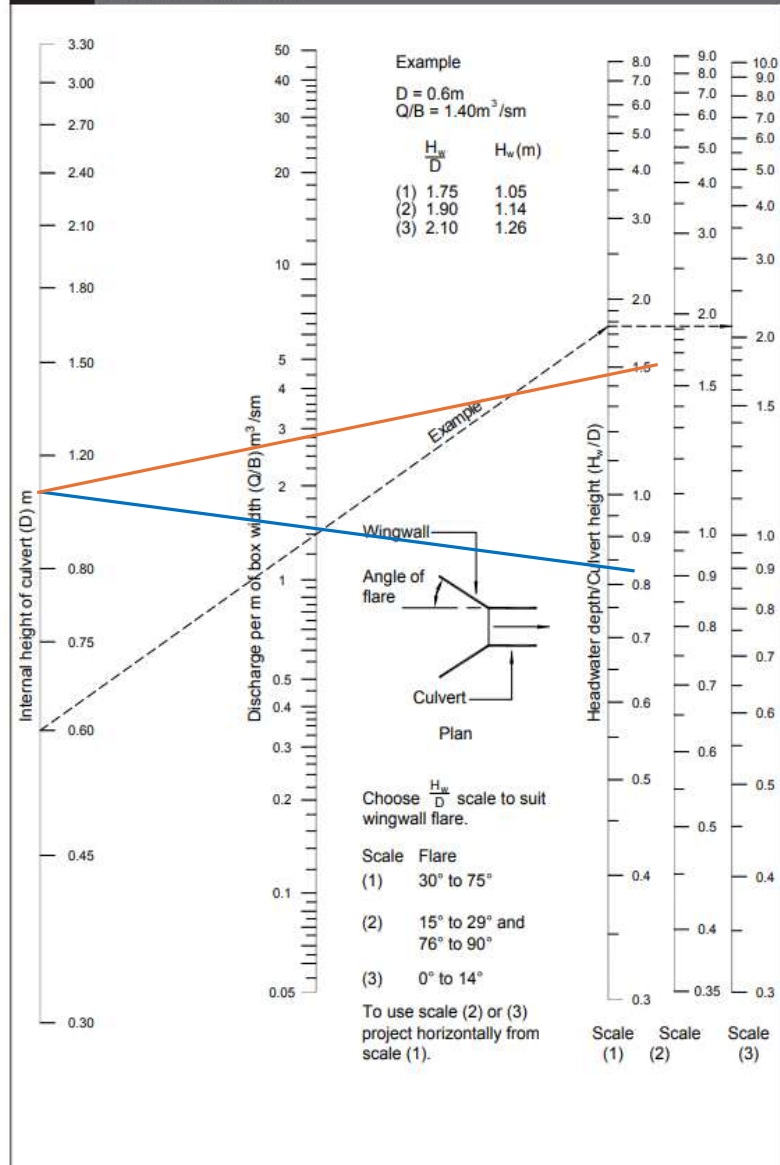
4.2.12.1 Design storm

Culverts must be designed as a minimum to accommodate storms as per the table below. The design must not cause any increase in upstream water levels that will result in flooding on neighbouring properties.

Table 60: Culvert design storms

DESIGN CASE	ARI YEAR STORM TO PASS WITHOUT SURCHARGE (DESIGN FLOW)	ARI YEAR STORM NOT OVERTOPPING STRUCTURE (PEAK FLOW)
Driveway or private way	2	NA
Pedestrian or cycleway walk	5	10
Local or collector road	5	100
Arterial roads and railways	20	100

Figure 7: Headwater Depth for Box Culverts
Paragraphs 3.2.3 b) and 4.1.4 b)



	Maven Associates	Job Number 180010	Sheet 13	Rev: A Ver: 1.0
Job Title Calc Title	Te Kowhai East Culvert Sizing CULVERT 6.2 - BOX	Author KY	Date 26/03/2026	Checked RW

CULVERT 6.2 - BOX

Catchment Area	A=	0.063 km ² (100ha = 1km ²)			
Design Case		Local/Collector			
Average recurrence interval, ARI		2 (yr)	5 (yr)	10 (yr)	100 (yr)
Peak flow rate, $q_p = q \cdot A \cdot P_{24}$		0.821 m ³ /s	1.103 m ³ /s	1.313 m ³ /s	2.068 m ³ /s
Internal Height of Culvert	D=	1 m			
Internal Base of Culvert	B=	1.5 m			
Datum Invert to Top of Culvert	=	1.5 m			
Discharge per m of box width	Q/B=		0.7356 m ³ /sm		1.37846 m ³ /sm
Headwater depth/Culvert Height	Hw/D=		0.530		0.830
From graph. Use Scale (1)					
Headwater	Hw=		0.530 m		0.830 m
Surcharge?			No Surcharge		No Surcharge
Surcharge Height			N/A m		N/A m
Compliance Check			OK		OK
Datum Invert to Top of Culvert	m=	1.500 m	1.500 m		1.500 m
Overtopping?			No Overtopping		No Overtopping
Compliance Check			OK		OK

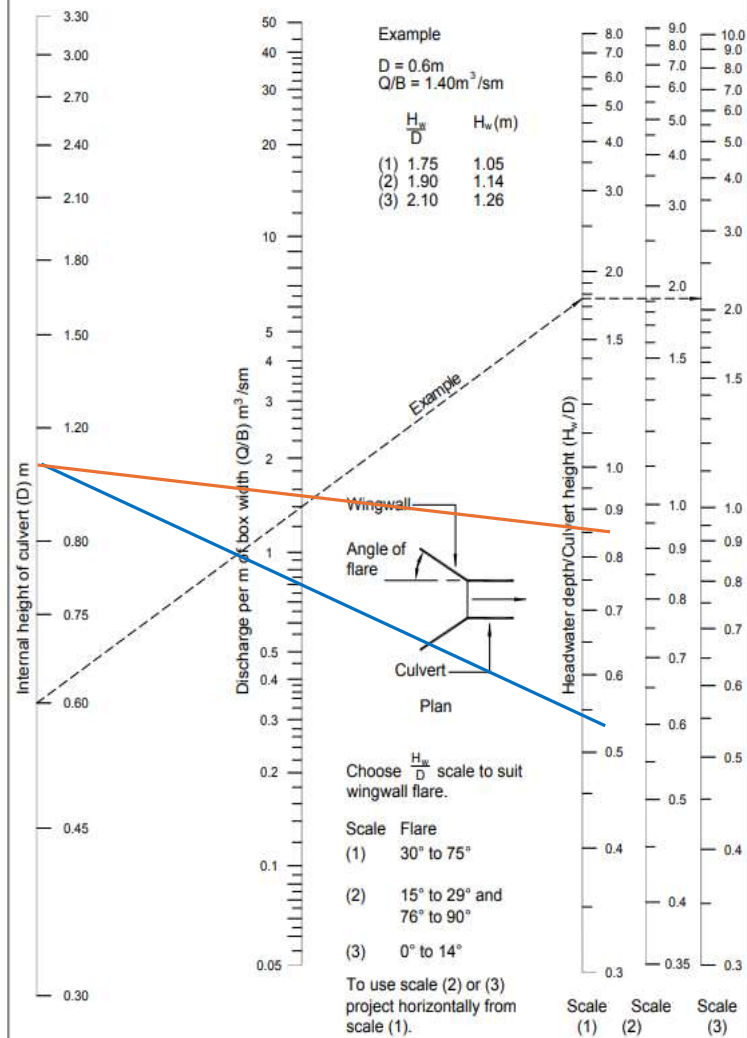
4.2.12.1 Design storm

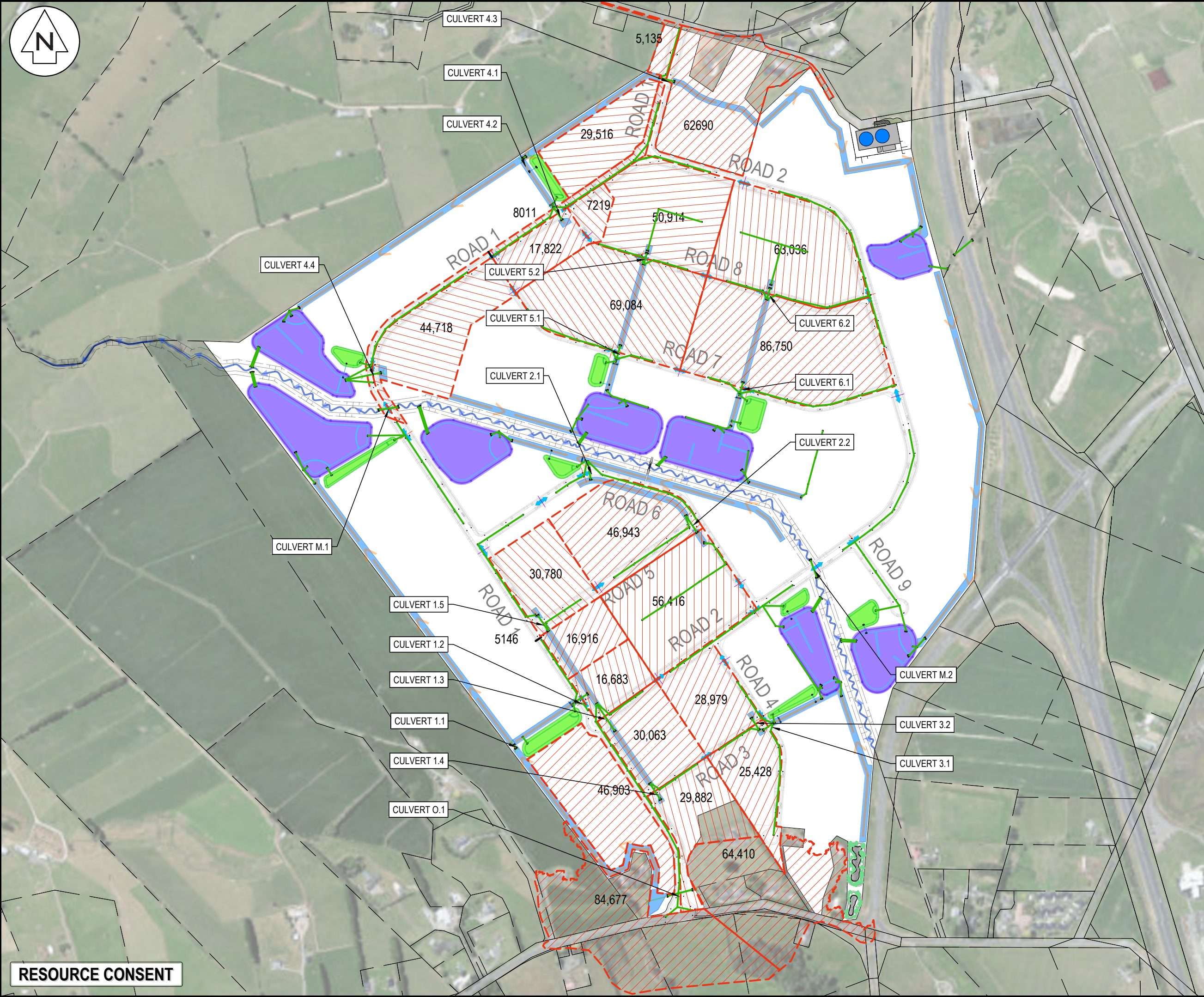
Culverts must be designed as a minimum to accommodate storms as per the table below. The design must not cause any increase in upstream water levels that will result in flooding on neighbouring properties.

Table 60: Culvert design storms

DESIGN CASE	ARI YEAR STORM TO PASS WITHOUT SURCHARGE (DESIGN FLOW)	ARI YEAR STORM NOT OVERTOPPING STRUCTURE (PEAK FLOW)
Driveway or private way	2	NA
Pedestrian or cycleway walk	5	10
Local or collector road	5	100
Arterial roads and railways	20	100

Figure 7: Headwater Depth for Box Culverts
Paragraphs 3.2.3 b) and 4.1.4 b)





NOTES

1. ALL WORKS TO BE IN ACCORDANCE WITH THE WAIKATO DISTRICT COUNCIL/IAWAI STANDARD DETAILS AND THE WAIKATO REGIONAL INFRASTRUCTURE TECHNICAL SPECIFICATION.
2. COORDINATES IN TERMS OF NZ GEODETIC DATUM MT EDEN 2000. LEVELS IN TERMS OF THE NEW ZEALAND VERTICAL DATUM 2016.
3. IT IS THE CONTRACTOR'S RESPONSIBILITY TO LOCATE ALL SERVICES THAT MAY BE AFFECTED BY THEIR OPERATIONS.
4. ALL CATCHMENT AREAS SHOWN ON THIS PLAN ARE IN SQUARE METRES (m²).

LEGEND

- PR SUB-CATCHMENT BOUNDARY
- PR MODIFIED STREAM
- PR 12.5m WIDE CONVEYANCE CHANNEL
- PR WETLAND
- PR RAINGARDEN
- PR BERM/LANDSCAPE
- PR LOT SHEET FLOW
- PR ROADING OLP DIRECTION
- PR ROADING CREST/HIGH PT
- PR ROADING SAG/LOW PT

A	RESOURCE CONSENT	AF	06/2026
Rev	Description	By	Date
Survey	MAVEN		10/2025
Design	AF		07/2026
Drawn	AF		07/2026
Checked	KY		07/2026



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5 Owens Road, Epsom
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Project

PROPOSED INDUSTRIAL DEVELOPMENT AT TE KOWHAI EAST FOR TE KOWHAI EAST LIMITED PARTNERSHIP

Title

PR STORMWATER SUB-CATCHMENT PLAN CULVERTS

Project no.	180010		
Scale	1:7500 @ A3		
Cad file	180010-C481 SUB-CATCHMENT PLAN.DWG		
Drawing no.	C482	Rev	A

RESOURCE CONSENT

APPENDIX C.5 – SW WETLAND SIZING

**MAVEN ASSOCIATES**Job Number
180010Sheet
1Rev
AJob Title
Calc TitleTe Kowhai East
WETLAND DESIGN MASTER SUMMARYAuthor
AZDate
15/07/2026Checked
KY

WETLAND DESIGN INFORMATION

WETLAND		WETLAND 1	WETLAND 2	WETLAND 3	WETLAND 4	WETLAND 5	WETLAND 6	WETLAND 7	WETLAND 8
CATCHMENT AREA (Ha)		38.12	19.50	16.50	29.71	20.88	27.62	9.83	17.34
TOP OF WL (RL)		26.25	27.00	28.70	26.25	27.85	28.00	30.10	28.90
100 YR WL		25.53	26.54	28.06	25.90	27.03	27.32	29.48	28.17
10 YR WL		25.13	25.78	27.61	25.37	26.65	26.93	29.20	27.76
2 YR WL		24.67	25.43	27.31	24.95	26.24	26.59	28.96	27.42
PWL (RL)		23.50	24.50	26.20	23.55	25.10	25.40	28.20	26.35
EDD (RL)		23.95	24.90	26.70	24.00	25.60	25.90	28.50	26.80
OUTLET CONTROL	EDD (ORIFICE)	5 x 160mm @ IL 23.60m	4 x 140mm @ IL 24.60m	4 x 120mm @ IL 26.30m	5 x 140mm @ IL 23.65m	4 x 130mm @ IL 25.15m	4 x 150mm @ IL 25.50m	4 x 100mm @ IL 28.20m	4 x 120mm @ IL 26.45m
	2 YR (ORIFICE)	5 x 180mm @ IL 23.95m	5 x 150mm @ IL 24.90m	5 x 180mm @ IL 26.70m	5 x 180mm @ IL 24.00m	4 x 150mm @ IL 25.60m	5 x 180mm @ IL 25.90m	5 x 130mm @ IL 28.50m	5 x 100mm @ IL 26.80m
	10 YR (SPILLWAY)	0.8m @ IL 24.85m	0.5m @ IL 25.55m	1.0m @ IL 27.40m	0.8m @ IL 24.90m	0.5m @ IL 26.55m	0.8m @ IL 26.60m	0.5m @ IL 28.90m	0.5m @ IL 27.60m
	100 YR (SPILLWAY)	1.5m @ IL 25.10m	0.2m @ IL 25.85m	0.5m @ IL 27.65m	2.0m @ IL 25.15m	0.5m @ IL 26.75m	1.5m @ IL 27.00m	1.0m @ IL 29.20m	NO WEIR
	EMERGENCY	30m @ IL 25.70m	15m @ IL 26.65m	15m @ IL 28.20m	35m @ IL 26.00m	15m @ IL 27.30m	25m @ IL 27.45m	15m @ IL 29.60m	15m @ IL 28.35m
OUTLET INVERT (IL)		23.40	24.40	26.10	23.45	25.00	25.30	28.10	26.25
WQV (m ³)	MIN	7,980	4,082	3,454	6,219	4,371	5,782	2,058	3,630
	DESIGNED	8,057	5,783	3,633	5,390	4,984	5,887	2,813	4,546
WETLAND PWL SURFACE AREA (m ²)	MIN	15,248	7,800	6,600	11,884	8,352	11,048	3,932	6,936
	DESIGNED	19,250	13,812	8,886	13,989	12,622	14,411	7,253	11,218
BATHYMETRY									
FOREBAY RL		22.00	23.00	24.70	22.05	23.60	23.90	26.70	24.85
SHALLOW MARSH RL		23.30	24.30	26.00	23.35	24.90	25.20	28.00	26.15
DEEP MARSH RL		23.15	24.15	25.85	23.20	24.75	25.05	27.85	26.00
INTERMEDIATE POOL		22.30	23.30	25.00	22.35	23.90	24.20	27.00	25.15
OUTLET POOL		22.00	23.00	24.70	22.05	23.60	23.90	26.70	24.85



Maven Associates

Job Number
180010

Sheet
3

Rev: A
Ver: 1.0

Job Title
Calc Title

Te Kowhai East
Post-development PERVIOUS- Volumes
Catchment 1

Author
IZ

Date
5/05/2026

Checked
RW

1. Runoff Curve Number (CN) and initial Abstraction (Ia)

Soil name and classification	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2= 1ha	Product of CN x area
D	Parkland and green space	80	3.81	304.96
D	Road Reserve Pervious Areas	80	0.00	0.00
D	Pervious Residential Lot Area	80	0.00	0.00
D	Commercial lot	80	0.00	0.00
D	Medium Density Housing	80	0.00	0.00
* from TR2018-2 Table 5-2		Totals =	3.8120	304.96

WQV

$$\text{CN (weighted)} = \frac{\text{total product}}{\text{total area}} = \frac{304.96}{3.812} = \boxed{80.0}$$

$$S = \frac{(1000}{\text{CN}} - 10) * 25.4 = \frac{(1000}{80.0} - 10) * 25.4 = \boxed{63.5} \text{ mm}$$

$$Ia = 0.05 * S = 0.05 * 63.5 = \boxed{3.2} \text{ mm}$$

Job Title
Calc Title

**Te Kowhai East
Post-development PERVIOUS- Volumes
Catchment 1**

Author
IZ

Date
5/05/2026

CHECKED
RW

- | | | |
|----|---------------------|--|
| 1. | Data | |
| | Catchment Area | A= 0.03812 km ² (100ha =1km ²) |
| | Runoff curve number | CN= 80.0 (from worksheet 3) |
| | Initial abstraction | Ia= 3.2 mm (from worksheet 3) |

2. Calculate storage, $S = (1000/CN - 10)25.4 = 64 \text{ mm}$

3. Average recurrence interval, ARI

4. 24 hour rainfall depth, P24

WQV	2 (yr)	10 yr	100 yr
27.1339	81.4016	127.104	198.912
6.6	43.2	81.9	147.8
250.20	1645.93 m ³	3123.65 m ³	5633.80 m ³

8. Runoff depth, $Q_{24} = (P_{24} - I_a)^2 / (P_{24} - I_a) + S$

9. Runoff volume, $V_{24} = 1000 \times Q_{24} A$

Worksheet 2: Graphical Peak Flow Rate

 Maven Associates		Job Number 180010	Sheet 5	Rev: A Ver: 1.0
Job Title Te Kowhai East Calc Title Post-development IMPERVIOUS-Volumes Catchment 1		Author IZ	Date 5/05/2026	Checked RW

1. Runoff Curve Number (CN) and initial Abstraction (Ia)

Soil name and classification	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1ha	Product of CN x area
C	Road Reserve Impervious Area	98	0.00	0.00
C	Residential Lots Impervious Area	98	0.00	0.00
C	Open Space Pervious Area	98	0.00	0.00
C	Medium Density Housing	98	0.00	0.00
C	Commercial Lots	98	34.31	3362.18
* from Appendix B		Totals =	34.3080	3362.18

CN (weighted) : $\frac{\text{total product}}{\text{total area}}$

S= $\frac{(1000}{CN} - 10) * 25.4$

Ia= 0.05*S

$\frac{3362.18}{34.308}$

$\frac{(1000}{98.0} - 10) * 25.4$

0.05x 5.2

= 98.0

= 5.2 mm

= 0.3 mm

	Job Title
	Calc Title

**Te Kowhai East
Post-development IMPERVIOUS-Volumes
Catchment 1**

Author
IZ

Date
5/05/2026

CHECKED
RW

- | | | | |
|----|---------------------|-----|---|
| 1. | Data | | |
| | Catchment Area | A= | 0.34308 km ² (100ha =1km ²) (from wrksht 5) |
| | Runoff curve number | CN= | 98.0 (from worksheet 5) |
| | Initial abstraction | Ia= | 0.3 mm (from worksheet 5) |

2. Calculate storage, $S = (1000/CN - 10)25.4$ = 5 mm

3. Average recurrence interval, ARI

4. 24 hour rainfall depth, P24

8. Runoff depth, $Q_{24} = (P_{24} - I_a)^2 / (P_{24} - I_a) + S$

9. Runoff volume, $V_{24} = 1000 \times Q_{24} A$

Combined Post development run off volume

Worksheet 2: Graphical Peak Flow Rate

P24: Obtained from HIRDS using a 3.8 deg Temperate rise adjustment
Data Downloaded on 11/2025



Maven Associates

Job Number
180010

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3

Rev: A
Ver: 1.0

Job Title
Calc Title

Te Kowhai East
Post-development PERVIOUS- Volumes
Catchment 2

Author
IZ

Date
5/05/2026

Checked
RW

1. Runoff Curve Number (CN) and initial Abstraction (Ia)

Soil name and classification	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2= 1ha	Product of CN x area
D	Parkland and green space	80	1.95	156.00
D	Road Reserve Pervious Areas	80	0.00	0.00
D	Pervious Residential Lot Area	80	0.00	0.00
D	Commercial lot	80	0.00	0.00
D	Medium Density Housing	80	0.00	0.00
* from TR2018-2 Table 5-2		Totals =	1.9500	156.00

WQV

$$\text{CN (weighted)} = \frac{\text{total product}}{\text{total area}} = \frac{156.00}{1.950} = \boxed{80.0}$$

$$S = \frac{(1000}{\text{CN}} - 10) * 25.4 = \frac{(1000}{80.0} - 10) * 25.4 = \boxed{63.5} \text{ mm}$$

$$Ia = 0.05 * S = 0.05 * 63.5 = \boxed{3.2} \text{ mm}$$

Job Title
Calc Title

**Te Kowhai East
Post-development PERVIOUS- Volumes
Catchment 2**

Author
IZ

Date
5/05/2026

CHECKED
RW

- | | | |
|----|---------------------|---|
| 1. | Data | |
| | Catchment Area | A= 0.0195 km ² (100ha =1km ²) |
| | Runoff curve number | CN= 80.0 (from worksheet 3) |
| | Initial abstraction | Ia= 3.2 mm (from worksheet 3) |

2. Calculate storage, $S = (1000/CN - 10)25.4 = 64 \text{ mm}$

3. Average recurrence interval, ARI

4. 24 hour rainfall depth, P24

WQV	2 (yr)	10 yr	100 yr
27.1339	81.4016	127.104	198.912
6.6	43.2	81.9	147.8
127.99	841.96 m ³	1597.88 m ³	2881.93 m ³

8. Runoff depth, $Q_{24} = (P_{24} - I_a)^2 / (P_{24} - I_a) + S$

9. Runoff volume, $V_{24} = 1000 \times Q_{24} A$

Worksheet 2: Graphical Peak Flow Rate



Maven Associates

Job Number
180010

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Rev: A
Ver: 1.0

Job Title
Calc Title

Te Kowhai East
Post-development IMPERVIOUS-Volumes
Catchment 2

Author
IZ

Date
5/05/2026

Checked
RW

1. Runoff Curve Number (CN) and initial Abstraction (Ia)

Soil name and classification	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1ha	Product of CN x area
C	Road Reserve Impervious Area	98	0.00	0.00
C	Residential Lots Impervious Area	98	0.00	0.00
C	Open Space Pervious Area	98	0.00	0.00
C	Medium Density Housing	98	0.00	0.00
C	Commercial Lots	98	17.55	1719.90
* from Appendix B		Totals =	17.5500	1719.90

$$\text{CN (weighted)} = \frac{\text{total product}}{\text{total area}} = \frac{1719.90}{17.550} = \boxed{98.0}$$

$$S = \frac{(1000}{\text{CN}} - 10) * 25.4 = \frac{(1000}{98.0} - 10) * 25.4 = \boxed{5.2} \text{ mm}$$

$$Ia = 0.05 * S = 0.05 * 5.2 = \boxed{0.3} \text{ mm}$$



Maven Associates

Job Number
180010

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Job Title
Calc Title

Te Kowhai East
Post-development IMPERVIOUS-Volumes
Catchment 2

Author
IZ

Date
5/05/2026

CHECKED
RW

1. Data
Catchment Area A= 0.1755 km²(100ha =1km²) (from wrksht 5)

Runoff curve number CN= 98.0 (from worksheet 5)

Initial abstraction Ia= 0.3 mm (from worksheet 5)

2. Calculate storage, $S = (1000/CN - 10)25.4$ = 5 mm

3. Average recurrence interval, ARI

WQV	2 (yr)	10 yr	100 yr
-----	--------	-------	--------

4. 24 hour rainfall depth, P₂₄

27.13386667	81.4016	127.104	198.912
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8. Runoff depth, $Q_{24} = (P_{24} - Ia)^2 / (P_{24} - Ia) + S$

22.5	76.3	121.9	193.6
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9. Runoff volume, $V_{24} = 1000 \times Q_{24} \times A$

3953.87	13385.39 m ³	21387.25 m ³	33976.97 m ³
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Combined Post development run off volume

4082	14227 m ³
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22985 m ³

36859 m ³

Worksheet 2: Graphical Peak Flow Rate

P24: Obtained from HIRDS using a 3.8 deg Temperature rise adjustment
Data Downloaded on 11/2025

	Maven Associates		Job Number 180010	Sheet 3	Rev: A Ver: 1.0
Job Title Calc Title	Te Kowhai East Post-development PERVIOUS- Volumes Catchment 3	Author IZ	Date 5/05/2026	Checked RW	

1. Runoff Curve Number (CN) and initial Abstraction (Ia)

Soil name and classification	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2= 1ha	Product of CN x area
D	Parkland and green space	80	1.65	132.00
D	Road Reserve Pervious Areas	80	0.00	0.00
D	Pervious Residential Lot Area	80	0.00	0.00
D	Commercial lot	80	0.00	0.00
D	Medium Density Housing	80	0.00	0.00
Totals =			1.6500	132.00

* from TR2018-2 Table 5-2

WQV

CN (weighted) : $\frac{\text{total product}}{\text{total area}} = \frac{132.00}{1.650} =$
80.0

S= $\frac{(1000}{CN} - 10) * 25.4 = \frac{(1000}{80.0} - 10) * 25.4 =$
63.5 mm

Ia= $0.05 * S = 0.05 * 63.5 =$
3.2 mm

Job Title
Calc Title

**Te Kowhai East
Post-development PERVIOUS- Volumes
Catchment 3**

Author
IZ

Date
5/05/2026

CHECKED
RW

- | | | |
|----|---------------------|---|
| 1. | Data | |
| | Catchment Area | A= 0.0165 km ² (100ha =1km ²) |
| | Runoff curve number | CN= 80.0 (from worksheet 3) |
| | Initial abstraction | Ia= 3.2 mm (from worksheet 3) |

2. Calculate storage, $S = (1000/CN - 10)25.4 = 64 \text{ mm}$

3. Average recurrence interval, ARI

4. 24 hour rainfall depth, P24

WQV	2 (yr)	
27.1	81.4	
6.6	43.2	
108.30	712.43 m ³	

10 yr	
127.1	
81.9	
1352.05 m ³	

100 yr	
198.9	
147.8	
2438.56 m ³	

8. Runoff depth, $Q_{24} = (P_{24} - I_a)^2 / (P_{24} - I_a) + S$

9. Runoff volume, $V_{24} = 1000 \times Q_{24} A$

Worksheet 2: Graphical Peak Flow Rate

	Maven Associates		Job Number 180010	Sheet 5	Rev: A Ver: 1.0
Job Title Calc Title	Te Kowhai East Post-development IMPERVIOUS-Volumes Catchment 3		Author IZ	Date 5/05/2026	Checked RW

1. Runoff Curve Number (CN) and initial Abstraction (Ia)

Soil name and classification	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1ha	Product of CN x area
C	Road Reserve Impervious Area	98	0.00	0.00
C	Residential Lots Impervious Area	98	0.00	0.00
C	Open Space Pervious Area	98	0.00	0.00
C	Medium Density Housing	98	0.00	0.00
C	Commercial Lots	98	14.85	1455.30
* from Appendix B			Totals =	14.8500 1455.30

CN (weighted) : $\frac{\text{total product}}{\text{total area}} = \frac{1455.30}{14.850} =$
98.0

S= $\frac{(1000}{CN} - 10) * 25.4 = \frac{(1000}{98.0} - 10) * 25.4 =$
5.2 mm

Ia= $0.05 * S = 0.05 * 5.2 =$
0.3 mm



Maven Associates

Job Number
180010

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Job Title
Calc Title

Te Kowhai East
Post-development IMPERVIOUS-Volumes
Catchment 3

Author
IZ

Date
5/05/2026

CHECKED
RW

1. Data
Catchment Area $A = 0.1485 \text{ km}^2$ (100ha =1km²) (from wrksht 5)
Runoff curve number $CN = 98.0$ (from worksheet 5)
Initial abstraction $I_a = 0.3 \text{ mm}$ (from worksheet 5)

2. Calculate storage, $S = (1000/CN - 10)25.4 = 5 \text{ mm}$

3. Average recurrence interval, ARI

WQV	2 (yr)	10 yr	100 yr
-----	--------	-------	--------

4. 24 hour rainfall depth, P₂₄

27.1	81.4	127.1	198.9
------	------	-------	-------

8. Runoff depth, $Q_{24} = (P_{24} - I_a)^2 / (P_{24} - I_a) + S$

22.5	76.3	121.9	193.6
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9. Runoff volume, $V_{24} = 1000 \times Q_{24} \times A$

3345.58	11326.10 m ³	18096.90 m ³	28749.74 m ³
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Combined Post development run off volume

3454	12039 m ³	19449 m ³	31188 m ³
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Worksheet 2: Graphical Peak Flow Rate

P24: Obtained from HIRDS using a 3.8 deg Temperature rise adjustment
Data Downloaded on 11/2025

	Maven Associates		Job Number 180010	Sheet 3	Rev: A Ver: 1.0
Job Title Calc Title	Te Kowhai East Post-development PERVIOUS- Volumes Catchment 4		Author IZ	Date 5/05/2026	Checked RW

1. Runoff Curve Number (CN) and initial Abstraction (Ia)

Soil name and classification	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2= 1ha	Product of CN x area
D	Parkland and green space	80	2.97	237.68
D	Road Reserve Pervious Areas	80	0.00	0.00
D	Pervious Residential Lot Area	80	0.00	0.00
D	Commercial lot	80	0.00	0.00
D	Medium Density Housing	80	0.00	0.00
Totals =			2.9710	237.68

* from TR2018-2 Table 5-2

WQV

CN (weighted) : $\frac{\text{total product}}{\text{total area}} = \frac{237.68}{2.971} =$
80.0

S= $\frac{(1000}{\text{CN}} - 10) * 25.4 = \frac{(1000}{80.0} - 10) * 25.4 =$
63.5 mm

Ia= $0.05 * S = 0.05 * 63.5 =$
3.2 mm

[illegible]

**Te Kowhai East
Post-development PERVIOUS- Volumes
Catchment 4**

Author
IZ

Date
5/05/2026

CHECKED
RW

- | | | |
|----|---------------------|--|
| 1. | Data | |
| | Catchment Area | A= 0.02971 km ² (100ha =1km ²) |
| | Runoff curve number | CN= 80.0 (from worksheet 3) |
| | Initial abstraction | Ia= 3.2 mm (from worksheet 3) |

2. Calculate storage, $S = (1000/CN - 10)25.4 = 64 \text{ mm}$

3. Average recurrence interval, ARI

4. 24 hour rainfall depth, P24

WQV	2 (yr)	10 yr
27.1	81.4	127.1
6.6	43.2	81.9
195.00	1282.80 m ³	2434.51 m ³

8. Runoff depth, $Q_{24} = (P_{24} - I_a)^2 / (P_{24} - I_a) + S$

9. Runoff volume, $V_{24} = 1000 \times Q_{24} A$

Worksheet 2: Graphical Peak Flow Rate

	Maven Associates	Job Number 180010	Sheet 5	Rev: A Ver: 1.0
Job Title Calc Title	Te Kowhai East Post-development IMPERVIOUS-Volumes Catchment 4	Author IZ	Date 5/05/2026	Checked RW

1. Runoff Curve Number (CN) and initial Abstraction (Ia)

Soil name and classification	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1ha	Product of CN x area
C	Road Reserve Impervious Area	98	0.00	0.00
C	Residential Lots Impervious Area	98	0.00	0.00
C	Open Space Pervious Area	98	0.00	0.00
C	Medium Density Housing	98	0.00	0.00
C	Commercial Lots	98	26.74	2620.42
* from Appendix B			Totals =	26.7390 2620.42

CN (weighted) : $\frac{\text{total product}}{\text{total area}} = \frac{2620.42}{26.739} =$
98.0

S= $\frac{(1000}{CN} - 10) * 25.4 = \frac{(1000}{98.0} - 10) * 25.4 =$
5.2 mm

Ia= $0.05 * S = 0.05 * 5.2 =$
0.3 mm



Maven Associates

Job Number
180010

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Job Title
Calc Title

Te Kowhai East
Post-development IMPERVIOUS-Volumes
Catchment 4

Author
IZ

Date
5/05/2026

CHECKED
RW

1. Data
Catchment Area $A = 0.26739 \text{ km}^2$ (100ha =1km²) (from wrksht 5)
Runoff curve number $CN = 98.0$ (from worksheet 5)
Initial abstraction $I_a = 0.3 \text{ mm}$ (from worksheet 5)

2. Calculate storage, $S = (1000/CN - 10)25.4 = 5 \text{ mm}$

3. Average recurrence interval, ARI

WQV	2 (yr)	10 yr	100 yr
-----	--------	-------	--------

4. 24 hour rainfall depth, P₂₄

27.1	81.4	127.1	198.9
------	------	-------	-------

8. Runoff depth, $Q_{24} = (P_{24} - I_a)^2 / (P_{24} - I_a) + S$

22.5	76.3	121.9	193.6
------	------	-------	-------

9. Runoff volume, $V_{24} = 1000 \times Q_{24} \times A$

6024.08	20393.84 m ³	32585.39 m ³	51766.96 m ³
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Combined Post development run off volume

6219	21677 m ³	35020 m ³	56158 m ³
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Worksheet 2: Graphical Peak Flow Rate

P24: Obtained from HIRDS using a 3.8 deg Temperature rise adjustment
Data Downloaded on 11/2025

	Maven Associates		Job Number 180010	Sheet 3	Rev: A Ver: 1.0
Job Title Calc Title	Te Kowhai East Post-development PERVIOUS- Volumes Catchment 5		Author IZ	Date 5/05/2026	Checked RW

1. Runoff Curve Number (CN) and initial Abstraction (Ia)

Soil name and classification	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2= 1ha	Product of CN x area
D	Parkland and green space	80	2.09	167.04
D	Road Reserve Pervious Areas	80	0.00	0.00
D	Pervious Residential Lot Area	80	0.00	0.00
D	Commercial lot	80	0.00	0.00
D	Medium Density Housing	80	0.00	0.00
Totals =			2.0880	167.04

* from TR2018-2 Table 5-2

WQV

CN (weighted) : $\frac{\text{total product}}{\text{total area}} = \frac{167.04}{2.088} =$
80.0

S= $\frac{(1000}{CN} - 10) * 25.4 = \frac{(1000}{80.0} - 10) * 25.4 =$
63.5 mm

Ia= $0.05 * S = 0.05 * 63.5 =$
3.2 mm

	Job Title	Calc Title
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**Te Kowhai East
Post-development PERVIOUS- Volumes
Catchment 5**

Author
IZ

Date
5/05/2026

CHECKED
RW

- | | | |
|----|---------------------|--|
| 1. | Data | |
| | Catchment Area | A= 0.02088 km ² (100ha =1km ²) |
| | Runoff curve number | CN= 80.0 (from worksheet 3) |
| | Initial abstraction | Ia= 3.2 mm (from worksheet 3) |

2. Calculate storage, $S = (1000/CN - 10)25.4 = 64 \text{ mm}$

3. Average recurrence interval, ARI

4. 24 hour rainfall depth, P24

WQV	2 (yr)	10 yr
27.1	81.4	127.1
6.6	43.2	81.9
137.04	901.55 m ³	1710.96 m ³

8. Runoff depth, $Q_{24} = (P_{24} - I_a)^2 / (P_{24} - I_a) + S$

9. Runoff volume, $V_{24} = 1000 \times Q_{24} A$

Worksheet 2: Graphical Peak Flow Rate

	Maven Associates		Job Number 180010	Sheet 5	Rev: A Ver: 1.0
Job Title Calc Title	Te Kowhai East Post-development IMPERVIOUS-Volumes Catchment 5		Author IZ	Date 5/05/2026	Checked RW

1. Runoff Curve Number (CN) and initial Abstraction (Ia)

Soil name and classification	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1ha	Product of CN x area
C	Road Reserve Impervious Area	98	0.00	0.00
C	Residential Lots Impervious Area	98	0.00	0.00
C	Open Space Pervious Area	98	0.00	0.00
C	Medium Density Housing	98	0.00	0.00
C	Commercial Lots	98	18.79	1841.62
* from Appendix B		Totals =	18.7920	1841.62

CN (weighted) : $\frac{\text{total product}}{\text{total area}} = \frac{1841.62}{18.792} =$
98.0

S= $\frac{(1000}{CN} - 10) * 25.4 = \frac{(1000}{98.0} - 10) * 25.4 =$
5.2 mm

Ia= $0.05 * S = 0.05 * 5.2 =$
0.3 mm

Job Title
Calc Title

Te Kowhai East
Post-development IMPERVIOUS-Volumes
Catchment 5

Author
IZ

Date
5/05/2026

CHECKED
RW

- | | | | |
|----|---------------------|-----|---|
| 1. | Data | | |
| | Catchment Area | A= | 0.18792 km ² (100ha =1km ²) (from wrksht 5) |
| | Runoff curve number | CN= | 98.0 (from worksheet 5) |
| | Initial abstraction | Ia= | 0.3 mm (from worksheet 5) |

2. Calculate storage, $S = (1000/CN - 10)25.4$ = 5 mm

- | | | | | |
|-------------------------------------|-----|--------|-------|--------|
| 3. Average recurrence interval, ARI | WQV | 2 (yr) | 10 yr | 100 yr |
|-------------------------------------|-----|--------|-------|--------|

- | | | | | | |
|----|-----------------------------|------|------|-------|-------|
| 4. | 24 hour rainfall depth, P24 | 27.1 | 81.4 | 127.1 | 198.9 |
|----|-----------------------------|------|------|-------|-------|

- | | | | | | |
|----|--|------|------|-------|-------|
| 8. | Runoff depth, $Q_{24} = (P_{24}-I_a)^2/(P_{24}-I_a)+S$ | 22.5 | 76.3 | 121.9 | 193.6 |
|----|--|------|------|-------|-------|

- | | | | | |
|---|----------------|--------------------------------|--------------------------------|--------------------------------|
| 9. Runoff volume, $V_{24} = 1000 \times Q_{24} A$ | 4233.68 | 14332.66 m ³ | 22900.81 m ³ | 36381.49 m ³ |
|---|----------------|--------------------------------|--------------------------------|--------------------------------|

Combined Post development run off volume	4371	15234 m ³	24612 m ³	39467 m ³
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Worksheet 2: Graphical Peak Flow Rate

P24: Obtained from HIRDS using a 3.8 deg Temperate rise adjustment
Data Downloaded on 11/2025

	Maven Associates		Job Number 180010	Sheet 3	Rev: A Ver: 1.0
Job Title Calc Title	Te Kowhai East Post-development PERVIOUS- Volumes Catchment 6		Author IZ	Date 5/05/2026	Checked RW

1. Runoff Curve Number (CN) and initial Abstraction (Ia)

Soil name and classification	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2= 1ha	Product of CN x area
D	Parkland and green space	80	2.76	220.96
D	Road Reserve Pervious Areas	80	0.00	0.00
D	Pervious Residential Lot Area	80	0.00	0.00
D	Commercial lot	80	0.00	0.00
D	Medium Density Housing	80	0.00	0.00
Totals =			2.7620	220.96

* from TR2018-2 Table 5-2

WQV

CN (weighted) : $\frac{\text{total product}}{\text{total area}} = \frac{220.96}{2.762}$

S= $\frac{(1000}{CN} - 10) * 25.4 = \frac{(1000}{80.0} - 10) * 25.4$

Ia= $0.05 * S = 0.05 * 63.5$

= 80.0

= 63.5 mm

= 3.2 mm

Job Title
Calc Title

**Te Kowhai East
Post-development PERVIOUS- Volumes
Catchment 6**

Author
IZ

Date
5/05/2026

CHECKED
RW

- | | | |
|----|---------------------|--|
| 1. | Data | |
| | Catchment Area | A= 0.02762 km ² (100ha =1km ²) |
| | Runoff curve number | CN= 80.0 (from worksheet 3) |
| | Initial abstraction | Ia= 3.2 mm (from worksheet 3) |

2. Calculate storage, $S = (1000/CN - 10)25.4 = 64 \text{ mm}$

3. Average recurrence interval, ARI

4. 24 hour rainfall depth, P24

WQV	2 (yr)	10 yr
27.1	81.4	127.1
6.6	43.2	81.9
181.28	1192.56 m ³	2263.25 m ³

8. Runoff depth, $Q_{24} = (P_{24} - I_a)^2 / (P_{24} - I_a) + S$

9. Runoff volume, $V_{24} = 1000 \times Q_{24} A$

Worksheet 2: Graphical Peak Flow Rate

	Maven Associates		Job Number 180010	Sheet 5	Rev: A Ver: 1.0
Job Title Calc Title	Te Kowhai East Post-development IMPERVIOUS-Volumes Catchment 6	Author IZ	Date 5/05/2026	Checked RW	

1. Runoff Curve Number (CN) and initial Abstraction (Ia)

Soil name and classification	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1ha	Product of CN x area
C	Road Reserve Impervious Area	98	0.00	0.00
C	Residential Lots Impervious Area	98	0.00	0.00
C	Open Space Pervious Area	98	0.00	0.00
C	Medium Density Housing	98	0.00	0.00
C	Commercial Lots	98	24.86	2436.08
* from Appendix B			Totals =	24.8580 2436.08

CN (weighted) : $\frac{\text{total product}}{\text{total area}} = \frac{2436.08}{24.858} =$
98.0

S= $\frac{(1000}{CN} - 10) * 25.4}{98.0} =$
5.2 mm

Ia= $0.05 * S = 0.05 * 5.2 =$
0.3 mm



Maven Associates

Job Number
180010

Sheet
6

Rev: A
Ver: 1.0

Job Title
Calc Title

Te Kowhai East
Post-development IMPERVIOUS-Volumes
Catchment 6

Author
IZ

Date
5/05/2026

CHECKED
RW

1. Data
Catchment Area $A = 0.24858 \text{ km}^2$ (100ha = 1km²) (from wrksht 5)
Runoff curve number $CN = 98.0$ (from worksheet 5)
Initial abstraction $I_a = 0.3 \text{ mm}$ (from worksheet 5)

2. Calculate storage, $S = (1000/CN - 10)25.4 = 5 \text{ mm}$

3. Average recurrence interval, ARI

WQV	2 (yr)	10 yr	100 yr
-----	--------	-------	--------

4. 24 hour rainfall depth, P₂₄

27.1	81.4	127.1	198.9
------	------	-------	-------

8. Runoff depth, $Q_{24} = (P_{24} - I_a)^2 / (P_{24} - I_a) + S$

22.5	76.3	121.9	193.6
------	------	-------	-------

9. Runoff volume, $V_{24} = 1000 \times Q_{24} \times A$

5600.30	18959.20 m ³	30293.12 m ³	48125.33 m ³
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Combined Post development run off volume

5782	20152 m ³	32556 m ³	52207 m ³
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Worksheet 2: Graphical Peak Flow Rate

P24: Obtained from HIRDS using a 3.8 deg Temperature rise adjustment
Data Downloaded on 11/2025

	Maven Associates		Job Number 180010	Sheet 3	Rev: A Ver: 1.0
Job Title Calc Title	Te Kowhai East Post-development PERVIOUS- Volumes Catchment 7		Author IZ	Date 5/05/2026	Checked RW

1. Runoff Curve Number (CN) and initial Abstraction (Ia)

Soil name and classification	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2= 1ha	Product of CN x area
D	Parkland and green space	80	0.98	78.64
D	Road Reserve Pervious Areas	80	0.00	0.00
D	Pervious Residential Lot Area	80	0.00	0.00
D	Commercial lot	80	0.00	0.00
D	Medium Density Housing	80	0.00	0.00
Totals =			0.9830	78.64

* from TR2018-2 Table 5-2

WQV

CN (weighted) : $\frac{\text{total product}}{\text{total area}} = \frac{78.64}{0.983} =$
80.0

S= $\frac{(1000}{CN} - 10) * 25.4 = \frac{(1000}{80.0} - 10) * 25.4 =$
63.5 mm

Ia= $0.05 * S = 0.05 * 63.5 =$
3.2 mm

Job Title
Calc Title

**Te Kowhai East
Post-development PERVIOUS- Volumes
Catchment 7**

Author
IZ

Date
5/05/2026

CHECKED
RW

- | | | |
|----|---------------------|--|
| 1. | Data | |
| | Catchment Area | A= 0.00983 km ² (100ha =1km ²) |
| | Runoff curve number | CN= 80.0 (from worksheet 3) |
| | Initial abstraction | Ia= 3.2 mm (from worksheet 3) |

2. Calculate storage, $S = (1000/CN - 10)25.4 = 64 \text{ mm}$

3. Average recurrence interval, ARI

4. 24 hour rainfall depth, P24

WQV	2 (yr)	10 yr
27.1	81.4	127.1
6.6	43.2	81.9
64.52	424.43 m ³	805.49 m ³

8. Runoff depth, $Q_{24} = (P_{24} - I_a)^2 / (P_{24} - I_a) + S$

9. Runoff volume, $V_{24} = 1000 \times Q_{24} A$

Worksheet 2: Graphical Peak Flow Rate

	Maven Associates		Job Number 180010	Sheet 5	Rev: A Ver: 1.0
Job Title Calc Title	Te Kowhai East Post-development IMPERVIOUS-Volumes Catchment 7	Author IZ	Date 5/05/2026	Checked RW	

1. Runoff Curve Number (CN) and initial Abstraction (Ia)

Soil name and classification	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1ha	Product of CN x area
C	Road Reserve Impervious Area	98	0.00	0.00
C	Residential Lots Impervious Area	98	0.00	0.00
C	Open Space Pervious Area	98	0.00	0.00
C	Medium Density Housing	98	0.00	0.00
C	Commercial Lots	98	8.85	867.01
* from Appendix B			Totals =	8.8470 867.01

CN (weighted) : $\frac{\text{total product}}{\text{total area}} = \frac{867.01}{8.847} =$
98.0

S= $\frac{(1000}{CN} - 10) * 25.4 = \frac{(1000}{98.0} - 10) * 25.4 =$
5.2 mm

Ia= $0.05 * S = 0.05 * 5.2 =$
0.3 mm



Maven Associates

Job Number
180010

Sheet
6

Rev: A
Ver: 1.0

Job Title
Calc Title

Te Kowhai East
Post-development IMPERVIOUS-Volumes
Catchment 7

Author
IZ

Date
5/05/2026

CHECKED
RW

1. Data
Catchment Area $A = 0.08847 \text{ km}^2$ (100ha = 1km²) (from wrksht 5)
Runoff curve number $CN = 98.0$ (from worksheet 5)
Initial abstraction $I_a = 0.3 \text{ mm}$ (from worksheet 5)

2. Calculate storage, $S = (1000/CN - 10)25.4 = 5 \text{ mm}$

3. Average recurrence interval, ARI	WQV	2 (yr)	10 yr	100 yr
4. 24 hour rainfall depth, P24	27.1	81.4	127.1	198.9
8. Runoff depth, $Q_{24} = (P_{24} - I_a)^2 / (P_{24} - I_a) + S$	22.5	76.3	121.9	193.6
9. Runoff volume, $V_{24} = 1000 \times Q_{24} \times A$	1993.16	6747.61 m ³	10781.37 m ³	17127.88 m ³

Combined Post development run off volume **2058** **7172** m³ **11587** m³ **18581** m³

Worksheet 2: Graphical Peak Flow Rate

P24: Obtained from HIRDS using a 3.8 deg Temperature rise adjustment
Data Downloaded on 11/2025

	Maven Associates		Job Number 180010	Sheet 3	Rev: A Ver: 1.0
Job Title Calc Title	Te Kowhai East Post-development PERVIOUS- Volumes Catchment 8		Author IZ	Date 5/05/2026	Checked RW

1. Runoff Curve Number (CN) and initial Abstraction (Ia)

Soil name and classification	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2= 1ha	Product of CN x area
D	Parkland and green space	80	1.73	138.72
D	Road Reserve Pervious Areas	80	0.00	0.00
D	Pervious Residential Lot Area	80	0.00	0.00
D	Commercial lot	80	0.00	0.00
D	Medium Density Housing	80	0.00	0.00
Totals =			1.7340	138.72

* from TR2018-2 Table 5-2

WQV

CN (weighted) : $\frac{\text{total product}}{\text{total area}} =$

$\frac{138.72}{1.734}$

= 80.0

S= $\frac{(1000}{CN} - 10) * 25.4$

$\frac{(1000 - 10)}{80.0} * 25.4$

= 63.5 mm

Ia= 0.05*S

0.05x 63.5

= 3.2 mm

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Job Title
Calc Title

**Te Kowhai East
Post-development PERVIOUS- Volumes
Catchment 8**

Author
IZ

Date
5/05/2026

CHECKED
RW

- | | | |
|----|---------------------|--|
| 1. | Data | |
| | Catchment Area | A= 0.01734 km ² (100ha =1km ²) |
| | Runoff curve number | CN= 80.0 (from worksheet 3) |
| | Initial abstraction | Ia= 3.2 mm (from worksheet 3) |

2. Calculate storage, $S = (1000/CN - 10)25.4 = 64 \text{ mm}$

3. Average recurrence interval, ARI

4. 24 hour rainfall depth, P24

WQV	2 (yr)		
27.1	81.4	10	100
		127.1	198.9
6.6	43.2	81.9	147.8
113.81	748.70 m ³	1420.88 m ³	2562.70 m ³

8. Runoff depth, $Q_{24} = (P_{24} - I_a)^2 / (P_{24} - I_a) + S$

9. Runoff volume, $V_{24} = 1000 \times Q_{24} A$

Worksheet 2: Graphical Peak Flow Rate

	Maven Associates		Job Number 180010	Sheet 5	Rev: A Ver: 1.0
Job Title Calc Title	Te Kowhai East Post-development IMPERVIOUS-Volumes Catchment 8		Author IZ	Date 5/05/2026	Checked RW

1. Runoff Curve Number (CN) and initial Abstraction (Ia)

Soil name and classification	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1ha	Product of CN x area
C	Road Reserve Impervious Area	98	0.00	0.00
C	Residential Lots Impervious Area	98	0.00	0.00
C	Open Space Pervious Area	98	0.00	0.00
C	Medium Density Housing	98	0.00	0.00
C	Commercial Lots	98	15.61	1529.39
* from Appendix B		Totals =	15.6060	1529.39

CN (weighted) : $\frac{\text{total product}}{\text{total area}} = \frac{1529.39}{15.606} =$
98.0

S= $\frac{(1000}{CN} - 10) * 25.4 = \frac{(1000}{98.0} - 10) * 25.4 =$
5.2 mm

Ia= $0.05 * S = 0.05 * 5.2 =$
0.3 mm

Job Title	Calc Title
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Te Kowhai East
Post-development IMPERVIOUS-Volumes
Catchment 8

Author
IZ

Date
5/05/2026

CHECKED
RW

- | | | | |
|----|---------------------|-----|---|
| 1. | Data | | |
| | Catchment Area | A= | 0.15606 km ² (100ha =1km ²) (from wrksht 5) |
| | Runoff curve number | CN= | 98.0 (from worksheet 5) |
| | Initial abstraction | Ia= | 0.3 mm (from worksheet 5) |

2. Calculate storage, $S = (1000/CN - 10)25.4$ = 5 mm

- | | | | | | |
|----|----------------------------------|-----|--------|-------|--------|
| 3. | Average recurrence interval, ARI | WQV | 2 (yr) | 10 yr | 100 yr |
|----|----------------------------------|-----|--------|-------|--------|

- | | | | | | |
|----|-----------------------------|------|------|-------|-------|
| 4. | 24 hour rainfall depth, P24 | 27.1 | 81.4 | 127.1 | 198.9 |
|----|-----------------------------|------|------|-------|-------|

- | | | | | | |
|----|--|------|------|-------|-------|
| 8. | Runoff depth, $Q_{24} = (P_{24}-I_a)^2/(P_{24}-I_a)+S$ | 22.5 | 76.3 | 121.9 | 193.6 |
|----|--|------|------|-------|-------|

- | | | | | | | | | |
|----|--|---------|----------|-------|----------|-------|----------|-------|
| 9. | Runoff volume, $V_{24} = 1000 \times Q_{24} A$ | 3515.90 | 11902.70 | m^3 | 19018.20 | m^3 | 30213.37 | m^3 |
|----|--|---------|----------|-------|----------|-------|----------|-------|

Combined Post development run off volume	3630	12651 m ³	20439 m ³	32776 m ³
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Worksheet 2: Graphical Peak Flow Rate

P24: Obtained from HIRDS using a 3.8 deg Temperate rise adjustment
Data Downloaded on 11/2025



Maven Associates

Job Number
180010

Sheet
1

Rev
A

Job Title
Calc Title
Catchment

Te Kowhai East
Wetland Sizing and dimensions
Catchment and Wetland 1

Author
KY

Date
13/05/2026

Checked
RW

References

WRC 2020/07 8.5.6 Stormwater pond
WRC 2020/07 8.5.7 Wetlands
RITs V2 Constructed Wetlands

Key:

input

Wetland / Pond Sizing and dimensions

Catchment Area	38.12	ha	
Imperviousness <70%?	No		
Percentage of catchment area	4%		
Min. Wetland PWL Surface Area	15248	m ²	per WRC 2020/06 8.5.7
Wetland Length	278	m	From 2D Plans
Wetland Width	54.8		
	5 OK		per RITS min. 5:1 max 8:1

Depth requirements

			Bathymetry Area Targets	Design Area	
Intermediate Deep Pools	10% 0.35-1.20	m	1925 m ²	2239	11.6% OK
Outlet Deep Pools		1.5 m			
Deep Marsh	40% 0.2-0.35	m	7700 m ²	7687	39.9% OK
Shallow Marsh	40% 0-0.2	m	7700 m ²	7418	38.5% OK
Forebay Depth	10% 1.0-1.50	m	1925 m ²	1905	9.9% OK

Info Table

	PostDev	
Q2	-	m ³ /s Designed separately via TMW Hydraulic TUFLOW Model
V2	-	m ³ Designed separately via TMW Hydraulic TUFLOW Model
Q10	-	m ³ /s Designed separately via TMW Hydraulic TUFLOW Model
V10	-	m ³ Designed separately via TMW Hydraulic TUFLOW Model
Q100	-	m ³ /s Designed separately via TMW Hydraulic TUFLOW Model
V100	-	m ³ Designed separately via TMW Hydraulic TUFLOW Model
Min. WQV	7979.51	m ³ 1/3 of the 2-year storm 24hr ARI rainfall event.
Min. EDV	9575.41	m ³ 1.2*WQV
Min. WQV within PSZ (EDD inclusion)	3989.75	m ³ 50% reduction from EDD inclusion

SUMMARY TABLE

Min. Wetland PWL Surface Area (4% catchment)	15,248	m ²	
Design Wetland PWL Surface Area	19,250	m ²	From 3D Model
Min. WQV within PSZ (EDD inclusion)	3990	m ³	50% reduction from EDD inclusion
Min. EDV	9575	m ³	1.2WQV 2872.623 0.149227179
Min. PSZ	13565	m ³	
Design WQV	8057	m ³	From 3D Bathymetry
Design EDD Depth		m	To be provided via HMS routing to max height of 0.35m and
Design EDD Volume		m ³	removal of EDV volume within 24-hours during detailed (EPA) design
Design PSZ	8057	m ³	75% of PSZ > 50% WQV? 6042.75 OK
Min. Forebay Volume (15% of min. WQV)	=	1196.93 m ³	Per WRC
Design Forebay Volume	=	1513 m ³	OK From 3D Model
Min. Forebay Area (10% of Design Wetland Area)	=	1925.00 m ²	per RITS
Design Forebay Area (+/- 2% Tolerance)	=	1905 m ²	9.9% OK From 2D Plan



Maven Associates

Job Number
180010

Sheet
1

Rev
A

Job Title
Calc Title
Catchment

Te Kowhai East
Wetland Sizing and dimensions
Catchment and Wetland 2

Author
KY

Date
13/05/2026

Checked
RW

References

WRC 2020/07 8.5.6 Stormwater pond
WRC 2020/07 8.5.7 Wetlands
RITs V2 Constructed Wetlands

Key:

input

Wetland / Pond Sizing and dimensions

Catchment Area	19.50 ha	
Imperviousness <70%?	No	
Percentage of catchment area	4%	
Min. Wetland PWL Surface Area	7800 m ²	per WRC 2020/06 8.5.7
Wetland Length	186 m	From 2D Plans
Wetland Width	41.9	
	4 TBD	per RITS min. 5:1 max 8:1

Increase ratio via additional EDD bunds, for detailed EPA design

Depth requirements

			Bathymetry Area Targets	Design Area	
Intermediate Deep Pools	10% 0.35-1.20 m		1381.2 m ²	1381	10.0% OK
Outlet Deep Pools	1.5 m				
Deep Marsh	40% 0.2-0.35 m		5524.8 m ²	5519	40.0% OK
Shallow Marsh	40% 0-0.2 m		5524.8 m ²	5531	40.0% OK
Forebay Depth	10% 1.0-1.50 m		1381.2 m ²	1382	10.0% OK

Info Table

	PostDev	
Q2	- m ³ /s	Designed separately via TMW Hydraulic TUFLOW Model
V2	- m ³	Designed separately via TMW Hydraulic TUFLOW Model
Q10	- m ³ /s	Designed separately via TMW Hydraulic TUFLOW Model
V10	- m ³	Designed separately via TMW Hydraulic TUFLOW Model
Q100	- m ³ /s	Designed separately via TMW Hydraulic TUFLOW Model
V100	- m ³	Designed separately via TMW Hydraulic TUFLOW Model
Min. WQV	4081.86 m ³	1/3 of the 2-year storm 24hr ARI rainfall event.
Min. EDV	4898.23 m ³	1.2*WQV
Min. WQV within PSZ (EDD inclusion)	2040.93 m ³	50% reduction from EDD inclusion

SUMMARY TABLE

Min. Wetland PWL Surface Area (4% catchment)	7,800 m ²	
Design Wetland PWL Surface Area	13,812 m ²	From 3D Model
Min. WQV within PSZ (EDD inclusion)	2041 m ³	50% reduction from EDD inclusion
Min. EDV	4898 m ³	1.2WQV 1469.469 0.106390736
Min. PSZ	6939 m ³	
Design WQV	5783 m ³	OK From 3D Bathymetry
Design EDD Depth	m	To be provided via HMS routing to max height of 0.35m and
Design EDD Volume	m ³	removal of EDV volume within 24-hours during detailed (EPA) design
Design PSZ	5783 m ³	75% of PSZ > 50% WQV?
		4337.25 OK
Min. Forebay Volume (15% of min. WQV)	= 612.28 m ³	Per WRC
Design Forebay Volume	= 1306 m ³	OK From 3D Model
Min. Forebay Area (10% of Design Wetland Area)	= 1381.20 m ²	per RITS
Design Forebay Area (+/- 2% Tolerance)	= 1382 m ²	10.0% OK From 2D Plan



Maven Associates

Job Number
180010

Sheet
1

Rev
A

Job Title
Calc Title
Catchment

Te Kowhai East
Wetland Sizing and dimensions
Catchment and Wetland 3

Author
KY

Date
13/05/2026

Checked
RW

References

WRC 2020/07 8.5.6 Stormwater pond
WRC 2020/07 8.5.7 Wetlands
RITs V2 Constructed Wetlands

Key:

input

Wetland / Pond Sizing and dimensions

Catchment Area	16.50 ha	
Imperviousness <70%?	No	
Percentage of catchment area	4%	
Min. Wetland PWL Surface Area	6600 m ²	per WRC 2020/06 8.5.7
Wetland Length	219 m	From 2D Plans
Wetland Width	30.1	
	7 OK	per RITS min. 5:1 max 8:1

Depth requirements

		Bathymetry Area Targets	Design Area	
Intermediate Deep Pools	10% 0.35-1.20 m	888.6 m ²	889	10.0% OK
Outlet Deep Pools	1.5 m			
Deep Marsh	40% 0.2-0.35 m	3554.4 m ²	3507	39.5% OK
Shallow Marsh	40% 0-0.2 m	3554.4 m ²	3597	40.5% OK
Forebay Depth	10% 1.0-1.50 m	888.6 m ²	893	10.0% OK

Info Table

	PostDev	
Q2	- m ³ /s	Designed separately via TMW Hydraulic TUFLOW Model
V2	- m ³	Designed separately via TMW Hydraulic TUFLOW Model
Q10	- m ³ /s	Designed separately via TMW Hydraulic TUFLOW Model
V10	- m ³	Designed separately via TMW Hydraulic TUFLOW Model
Q100	- m ³ /s	Designed separately via TMW Hydraulic TUFLOW Model
V100	- m ³	Designed separately via TMW Hydraulic TUFLOW Model
Min. WQV	3453.88 m ³	1/3 of the 2-year storm 24hr ARI rainfall event.
Min. EDV	4144.66 m ³	1.2*WQV
Min. WQV within PSZ (EDD inclusion)	1726.94 m ³	50% reduction from EDD inclusion

SUMMARY TABLE

Min. Wetland PWL Surface Area (4% catchment)	6,600 m ²	
Design Wetland PWL Surface Area	8,886 m ²	From 3D Model
Min. WQV within PSZ (EDD inclusion)	1727 m ³	50% reduction from EDD inclusion
Min. EDV	4145 m ³	1.2WQV 1243.397 0.139927607
Min. PSZ	5872 m ³	
Design WQV	3633 m ³	OK From 3D Bathymetry
Design EDD Depth	m	To be provided via HMS routing to max height of 0.35m and
Design EDD Volume	m ³	removal of EDV volume within 24-hours during detailed (EPA) design
Design PSZ	3633 m ³	75% of PSZ > 50% WQV?
		2724.75 OK
Min. Forebay Volume (15% of min. WQV)	= 518.08 m ³	Per WRC
Design Forebay Volume	= 798 m ³	OK From 3D Model
Min. Forebay Area (10% of Design Wetland Area)	= 888.60 m ²	per RITS
Design Forebay Area (+/- 2% Tolerance)	= 893 m ²	10.0% OK From 2D Plan



Maven Associates

Job Number
180010

Sheet
1

Rev
A

Job Title
Calc Title
Catchment

Te Kowhai East
Wetland Sizing and dimensions
Catchment and Wetland 4

Author
KY

Date
13/05/2026

Checked
RW

References

WRC 2020/07 8.5.6 Stormwater pond
WRC 2020/07 8.5.7 Wetlands
RITs V2 Constructed Wetlands

Key:

input

Wetland / Pond Sizing and dimensions

Catchment Area	29.71 ha	
Imperviousness <70%?	No	
Percentage of catchment area	4%	
Min. Wetland PWL Surface Area	11884 m ²	per WRC 2020/06 8.5.7
Wetland Length	256 m	From 2D Plans
Wetland Width	46.4	
	6 OK	per RITS min. 5:1 max 8:1

Depth requirements

		Bathymetry Area Targets	Design Area	
Intermediate Deep Pools	10% 0.35-1.20 m	1398.9 m ²	1469	10.5% OK
Outlet Deep Pools	1.5 m			
Deep Marsh	40% 0.2-0.35 m	5595.6 m ²	5317	38.0% OK
Shallow Marsh	40% 0-0.2 m	5595.6 m ²	5680	40.6% OK
Forebay Depth	10% 1.0-1.50 m	1398.9 m ²	1295	9.3% OK

Info Table

	PostDev	
Q2	- m ³ /s	Designed separately via TMW Hydraulic TUFLOW Model
V2	- m ³	Designed separately via TMW Hydraulic TUFLOW Model
Q10	- m ³ /s	Designed separately via TMW Hydraulic TUFLOW Model
V10	- m ³	Designed separately via TMW Hydraulic TUFLOW Model
Q100	- m ³ /s	Designed separately via TMW Hydraulic TUFLOW Model
V100	- m ³	Designed separately via TMW Hydraulic TUFLOW Model
Min. WQV	6219.08 m ³	1/3 of the 2-year storm 24hr ARI rainfall event.
Min. EDV	7462.89 m ³	1.2*WQV
Min. WQV within PSZ (EDD inclusion)	3109.54 m ³	50% reduction from EDD inclusion

SUMMARY TABLE

Min. Wetland PWL Surface Area (4% catchment)	11,884 m ²	
Design Wetland PWL Surface Area	13,989 m ²	From 3D Model
Min. WQV within PSZ (EDD inclusion)	3110 m ³	50% reduction from EDD inclusion
Min. EDV	7463 m ³	1.2WQV 2238.868 0.160044868
Min. PSZ	10572 m ³	
Design WQV	5390 m ³	OK From 3D Bathymetry
Design EDD Depth	m	To be provided via HMS routing to max height of 0.35m and
Design EDD Volume	m ³	removal of EDV volume within 24-hours during detailed (EPA) design
Design PSZ	5390 m ³	75% of PSZ > 50% WQV?
		4042.5 OK
Min. Forebay Volume (15% of min. WQV)	= 932.86 m ³	Per WRC
Design Forebay Volume	= 902 m ³	ERROR From 3D Model
Min. Forebay Area (10% of Design Wetland Area)	= 1398.90 m ²	per RITS
Design Forebay Area (+/- 2% Tolerance)	= 1295 m ²	9.3% OK From 2D Plan



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A

Job Title
Calc Title
Catchment

Te Kowhai East
Wetland Sizing and dimensions
Catchment and Wetland 5

Author
KY

Date
13/05/2026

Checked
RW

References

WRC 2020/07 8.5.6 Stormwater pond
WRC 2020/07 8.5.7 Wetlands
RITs V2 Constructed Wetlands

Key:

input

Wetland / Pond Sizing and dimensions

Catchment Area	20.88 ha	
Imperviousness <70%?	No	
Percentage of catchment area	4%	
Min. Wetland PWL Surface Area	8352 m ²	per WRC 2020/06 8.5.7
Wetland Length	257 m	From 2D Plans
Wetland Width	32.5	
	8 OK	per RITS min. 5:1 max 8:1

Depth requirements

			Bathymetry Area Targets	Design Area	
Intermediate Deep Pools	10% 0.35-1.20 m		1262.18 m ²	1262	10.0% OK
Outlet Deep Pools	1.5 m				
Deep Marsh	40% 0.2-0.35 m		5048.72 m ²	5188	41.1% OK
Shallow Marsh	40% 0-0.2 m		5048.72 m ²	4910	38.9% OK
Forebay Depth	10% 1.0-1.50 m		1262.18 m ²	1261	10.0% OK

Info Table

	PostDev	
Q2	- m ³ /s	Designed separately via TMW Hydraulic TUFLOW Model
V2	- m ³	Designed separately via TMW Hydraulic TUFLOW Model
Q10	- m ³ /s	Designed separately via TMW Hydraulic TUFLOW Model
V10	- m ³	Designed separately via TMW Hydraulic TUFLOW Model
Q100	- m ³ /s	Designed separately via TMW Hydraulic TUFLOW Model
V100	- m ³	Designed separately via TMW Hydraulic TUFLOW Model
Min. WQV	4370.73 m ³	1/3 of the 2-year storm 24hr ARI rainfall event.
Min. EDV	5244.87 m ³	1.2*WQV
Min. WQV within PSZ (EDD inclusion)	2185.36 m ³	50% reduction from EDD inclusion

SUMMARY TABLE

Min. Wetland PWL Surface Area (4% catchment)	8,352 m ²	
Design Wetland PWL Surface Area	12,622 m ²	From 3D Model
Min. WQV within PSZ (EDD inclusion)	2185 m ³	50% reduction from EDD inclusion
Min. EDV	5245 m ³	1.2WQV 1573.462 0.124662253
Min. PSZ	7430 m ³	
Design WQV	4984 m ³	OK From 3D Bathymetry
Design EDD Depth	m	To be provided via HMS routing to max height of 0.35m and
Design EDD Volume	m ³	removal of EDV volume within 24-hours during detailed (EPA) design
Design PSZ	4984 m ³	75% of PSZ > 50% WQV? 3738 OK
Min. Forebay Volume (15% of min. WQV)	= 655.61 m ³	Per WRC
Design Forebay Volume	= 911 m ³	OK From 3D Model
Min. Forebay Area (10% of Design Wetland Area)	= 1262.18 m ²	per RITS
Design Forebay Area (+/- 2% Tolerance)	= 1261 m ²	10.0% OK From 2D Plan



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A

Job Title
Calc Title
Catchment

Te Kowhai East
Wetland Sizing and dimensions
Catchment and Wetland 6

Author
KY

Date
13/05/2026

Checked
RW

References

WRC 2020/07 8.5.6 Stormwater pond
WRC 2020/07 8.5.7 Wetlands
RITs V2 Constructed Wetlands

Key:

input

Wetland / Pond Sizing and dimensions

Catchment Area	27.62 ha	
Imperviousness <70%?	No	
Percentage of catchment area	4%	
Min. Wetland PWL Surface Area	11048 m ²	per WRC 2020/06 8.5.7
Wetland Length	248 m	From 2D Plans
Wetland Width	44.5	
	6 OK	per RITS min. 5:1 max 8:1

Depth requirements

Bathymetry Area Targets

Design Area

Intermediate Deep Pools	10% 0.35-1.20 m	1441.1 m ²	1441	10.0% OK
Outlet Deep Pools	1.5 m			
Deep Marsh	40% 0.2-0.35 m	5764.4 m ²	5612	38.9% OK
Shallow Marsh	40% 0-0.2 m	5764.4 m ²	5898	40.9% OK
Forebay Depth	10% 1.0-1.50 m	1441.1 m ²	1449	10.1% OK

Info Table

PostDev

Q2	- m ³ /s	Designed separately via TMW Hydraulic TUFLOW Model
V2	- m ³	Designed separately via TMW Hydraulic TUFLOW Model
Q10	- m ³ /s	Designed separately via TMW Hydraulic TUFLOW Model
V10	- m ³	Designed separately via TMW Hydraulic TUFLOW Model
Q100	- m ³ /s	Designed separately via TMW Hydraulic TUFLOW Model
V100	- m ³	Designed separately via TMW Hydraulic TUFLOW Model
Min. WQV	5781.59 m ³	1/3 of the 2-year storm 24hr ARI rainfall event.
Min. EDV	6937.90 m ³	1.2*WQV
Min. WQV within PSZ (EDD inclusion)	2890.79 m ³	50% reduction from EDD inclusion

SUMMARY TABLE

Min. Wetland PWL Surface Area (4% catchment)	11,048 m ²	
Design Wetland PWL Surface Area	14,411 m ²	From 3D Model
Min. WQV within PSZ (EDD inclusion)	2891 m ³	50% reduction from EDD inclusion
Min. EDV	6938 m ³	1.2WQV 2081.371 0.144429307
Min. PSZ	9829 m ³	
Design WQV	5887 m ³	OK From 3D Bathymetry
Design EDD Depth	m	To be provided via HMS routing to max height of 0.35m and
Design EDD Volume	m ³	removal of EDV volume within 24-hours during detailed (EPA) design
Design PSZ	5887 m ³	75% of PSZ > 50% WQV? 4415.25 OK
Min. Forebay Volume (15% of min. WQV)	= 867.24 m ³	Per WRC
Design Forebay Volume	= 1270 m ³	OK From 3D Model
Min. Forebay Area (10% of Design Wetland Area)	= 1441.10 m ²	per RITS
Design Forebay Area (+/- 2% Tolerance)	= 1449 m ²	10.1% OK From 2D Plan



Maven Associates

Job Number
180010

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Job Title
Calc Title
Catchment

Te Kowhai East
Wetland Sizing and dimensions
Catchment and Wetland 7

Author
KY

Date
13/05/2026

Checked
RW

References

WRC 2020/07 8.5.6 Stormwater pond
WRC 2020/07 8.5.7 Wetlands
RITs V2 Constructed Wetlands

Key:

input

Wetland / Pond Sizing and dimensions

Catchment Area	9.83 ha	
Imperviousness <70%?	No	
Percentage of catchment area	4%	
Min. Wetland PWL Surface Area	3932 m ²	per WRC 2020/06 8.5.7
Wetland Length	168 m	From 2D Plans
Wetland Width	23.4	
	7 OK	per RITS min. 5:1 max 8:1

Depth requirements

		Bathymetry Area Targets	Design Area	
Intermediate Deep Pools	10% 0.35-1.20 m	725.3 m ²	725	10.0% OK
Outlet Deep Pools	1.5 m			
Deep Marsh	40% 0.2-0.35 m	2901.2 m ²	2740	37.8% OK
Shallow Marsh	40% 0-0.2 m	2901.2 m ²	2888	39.8% OK
Forebay Depth	10% 1.0-1.50 m	725.3 m ²	735	10.1% OK

Info Table

	PostDev	
Q2	- m ³ /s	Designed separately via TMW Hydraulic TUFLOW Model
V2	- m ³	Designed separately via TMW Hydraulic TUFLOW Model
Q10	- m ³ /s	Designed separately via TMW Hydraulic TUFLOW Model
V10	- m ³	Designed separately via TMW Hydraulic TUFLOW Model
Q100	- m ³ /s	Designed separately via TMW Hydraulic TUFLOW Model
V100	- m ³	Designed separately via TMW Hydraulic TUFLOW Model
Min. WQV	2057.68 m ³	1/3 of the 2-year storm 24hr ARI rainfall event.
Min. EDV	2469.21 m ³	1.2*WQV
Min. WQV within PSZ (EDD inclusion)	1028.84 m ³	50% reduction from EDD inclusion

SUMMARY TABLE

Min. Wetland PWL Surface Area (4% catchment)	3,932 m ²	
Design Wetland PWL Surface Area	7,253 m ²	From 3D Model
Min. WQV within PSZ (EDD inclusion)	1029 m ³	50% reduction from EDD inclusion
Min. EDV	2469 m ³	1.2WQV 740.763 0.102131947
Min. PSZ	3498 m ³	
Design WQV	2813 m ³	OK From 3D Bathymetry
Design EDD Depth	m	To be provided via HMS routing to max height of 0.35m and
Design EDD Volume	m ³	removal of EDV volume within 24-hours during detailed (EPA) design
Design PSZ	2813 m ³	75% of PSZ > 50% WQV?
		2109.75 OK
Min. Forebay Volume (15% of min. WQV)	= 308.65 m ³	Per WRC
Design Forebay Volume	= 529 m ³	OK From 3D Model
Min. Forebay Area (10% of Design Wetland Area)	= 725.30 m ²	per RITS
Design Forebay Area (+/- 2% Tolerance)	= 735 m ²	10.1% OK From 2D Plan



Maven Associates

Job Number
180010

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Rev
A

Job Title
Calc Title
Catchment

Te Kowhai East
Wetland Sizing and dimensions
Catchment and Wetland 8

Author
KY

Date
13/05/2026

Checked
RW

References

WRC 2020/07 8.5.6 Stormwater pond
WRC 2020/07 8.5.7 Wetlands
RITs V2 Constructed Wetlands

Key:

input

Wetland / Pond Sizing and dimensions

Catchment Area	17.34 ha	
Imperviousness <70%?	No	
Percentage of catchment area	4%	
Min. Wetland PWL Surface Area	6936 m ²	per WRC 2020/06 8.5.7
Wetland Length	195 m	From 2D Plans
Wetland Width	35.6	
	5 OK	per RITS min. 5:1 max 8:1

Depth requirements

Bathymetry Area Targets

Design Area

Intermediate Deep Pools	10% 0.35-1.20 m	1121.8 m ²	1239	11.0% OK
Outlet Deep Pools	1.5 m			
Deep Marsh	40% 0.2-0.35 m	4487.2 m ²	4376	39.0% OK
Shallow Marsh	40% 0-0.2 m	4487.2 m ²	4478	39.9% OK
Forebay Depth	10% 1.0-1.50 m	1121.8 m ²	1127	10.0% OK

Info Table

PostDev

Q2	- m ³ /s	Designed separately via TMW Hydraulic TUFLOW Model
V2	- m ³	Designed separately via TMW Hydraulic TUFLOW Model
Q10	- m ³ /s	Designed separately via TMW Hydraulic TUFLOW Model
V10	- m ³	Designed separately via TMW Hydraulic TUFLOW Model
Q100	- m ³ /s	Designed separately via TMW Hydraulic TUFLOW Model
V100	- m ³	Designed separately via TMW Hydraulic TUFLOW Model
Min. WQV	3629.71 m ³	1/3 of the 2-year storm 24hr ARI rainfall event.
Min. EDV	4355.66 m ³	1.2*WQV
Min. WQV within PSZ (EDD inclusion)	1814.86 m ³	50% reduction from EDD inclusion

SUMMARY TABLE

Min. Wetland PWL Surface Area (4% catchment)	6,936 m ²	
Design Wetland PWL Surface Area	11,218 m ²	From 3D Model
Min. WQV within PSZ (EDD inclusion)	1815 m ³	50% reduction from EDD inclusion
Min. EDV	4356 m ³	1.2WQV 1306.697 0.116482164
Min. PSZ	6171 m ³	
Design WQV	4546 m ³	OK From 3D Bathymetry
Design EDD Depth	m	To be provided via HMS routing to max height of 0.35m and
Design EDD Volume	m ³	removal of EDV volume within 24-hours during detailed (EPA) design
Design PSZ	4546 m ³	75% of PSZ > 50% WQV? 3409.5 OK
Min. Forebay Volume (15% of min. WQV)	= 544.46 m ³	Per WRC
Design Forebay Volume	= 933 m ³	OK From 3D Model
Min. Forebay Area (10% of Design Wetland Area)	= 1121.80 m ²	per RITS
Design Forebay Area (+/- 2% Tolerance)	= 1127 m ²	10.0% OK From 2D Plan



MAVEN ASSOCIATES

Job Number
180010

Sheet
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Rev
A

Job Title
Calc Title

Te Kowhai East
Wetland 1 Outlet Controls
EDD Orifice & Emergency Spillway

Author
KY

Date
1-Mar

Checked

Wetland 1

VOLUMES

Min. WQV	6838 m ³	From 'Wetland Sizing' Spreadsheet
Min. EDV	8206 m ³	1.2WQV

LEVELS

PWL	23.50 m	Design based on 3D Model
EDD	23.95 m	0.45 (m) EDD Depth
2YR Water Level	24.67 m	Based on TMW TUFLOW Modelling
10YR Water Level	25.13 m	Based on TMW TUFLOW Modelling
100YR Water Level	25.53 m	Based on TMW TUFLOW Modelling
Spillway Invert	25.70 m	(Min.) 100mm above 100YR WL
Spillway Water Level	25.95 m	Design based on 3D Model
Top of Wetland	26.25 m	0.30 (m) Freeboard

FLOWS

Q ₂ post	m ³ /s	
Q ₁₀ post	m ³ /s	
Q ₁₀₀ post	5.28 m ³ /s	From TMW Hydrograph

CALC 1: Extended Detention Orifice

Outlet to be sized to release the EDV over a 24-hour period

$$Q_{\text{EDD}} = \frac{8206 \text{ m}^3 \text{ over } 24 \text{ hours}}{0.09 \text{ m}^3/\text{s}}$$

At full EDV elev., maximum release rate is assumed to be $Q_{\text{max}} = 2 * Q_{\text{EDD}}$

$$Q_{\text{max}} = 0.1899 \text{ m}^3/\text{s}$$

Vol. of storage savings can be realised if edv is determined by routing flows through the pond

$$Q_i = n * 0.62 * A * (2 * g * h_i)^{0.5} \quad \text{where } A = \text{area of EDD orifice}$$

$$n = 5 \text{ No. of orifices}$$

$$\text{orifice dia} = 0.16 \text{ m}$$

$$A = 0.0201062 \text{ m}^2$$

$$h_i = 0.37 \text{ m}$$

Concept only. Provide routing via HMS
to limit EDD depth to 0.35m at EPA design

$$Q_i = 0.1679 \text{ m}^3/\text{s} \quad \text{OK}$$

Use **5** x **160** mm orifice

CALC 2: Emergency Spillway

$$Q_{\text{spill}} = 0.57 (2g)^{1/2} (2/3 L h^{3/2} + 8/15 Z h^{5/2})$$

Where Q= discharge through the spillway m^3/s
 L = horizontal bottom width of the spillway m
 h = depth of design flow m
 z = horizontal/vertical side slope (recommended to be 3) m

$$Q_{100 \text{ post}} = 5.28 \text{ m}^3/\text{s}$$

$$Q_{\text{spill}} = 6.44 \text{ m}^3/\text{s}$$

$$h = 0.25 \text{ m}$$

$$z = 3 \quad 1V:3H \text{ slope}$$

$$L = 30.0 \text{ m} \quad \text{trial and error} \quad \text{OK}$$

$$\text{Weir width} = L = 30 \text{ m}$$



MAVEN ASSOCIATES

Job Number
180010

Sheet
1

Rev
A

Job Title
Calc Title

Te Kowhai East
Wetland 2 Outlet Controls
EDD Orifice & Emergency Spillway

Author
KY

Date
1-Mar

Checked

Wetland 2

VOLUMES

Min. WQV	3608 m ³	From 'Wetland Sizing' Spreadsheet
Min. EDV	4329 m ³	1.2WQV

LEVELS

PWL	24.50 m	Design based on 3D Model
EDD	24.90 m	0.4 (m) EDD Depth
2YR Water Level	25.43 m	Based on TMW TUFLOW Modelling
10YR Water Level	25.78 m	Based on TMW TUFLOW Modelling
100YR Water Level	26.54 m	Based on TMW TUFLOW Modelling
Spillway Invert	26.65 m	(Min.) 100mm above 100YR WL
Spillway Water Level	26.90 m	Design based on 3D Model
Top of Wetland	27.00 m	0.10 (m) Freeboard

Raise Maintenance Track RL at detailed design EPA
to achieve freeboard

FLOWS

Q ₂ post	m ³ /s	
Q ₁₀ post	m ³ /s	
Q ₁₀₀ post	2.42 m ³ /s	From TMW Hydrograph

CALC 1: Extended Detention Orifice

Outlet to be sized to release the EDV over a 24-hour period

$$Q_{\text{EDD}} = \frac{4329 \text{ m}^3 \text{ over } 24 \text{ hours}}{0.05 \text{ m}^3/\text{s}}$$

At full EDV elev., maximum release rate is assumed to be $Q_{\text{max}} = 2 * Q_{\text{EDD}}$

$$Q_{\text{max}} = 0.1002 \text{ m}^3/\text{s}$$

Vol. of storage savings can be realised if edv is determined by routing flows through the pond

$$Q_i = n * 0.62 * A * (2 * g * h_i)^{0.5} \quad \text{where } A = \text{area of EDD orifice}$$

$$n = 4 \text{ No. of orifices}$$

$$\text{orifice dia} = 0.14 \text{ m}$$

$$A = 0.0153938 \text{ m}^2$$

$$h_i = 0.33 \text{ m}$$

Concept only. Provide routing via HMS
to limit EDD depth to 0.35m at EPA design

$$Q_i = 0.0971 \text{ m}^3/\text{s} \quad \text{OK}$$

Use **4** x **140** mm orifice

CALC 2: Emergency Spillway

$$Q_{\text{spill}} = 0.57 (2g)^{1/2} (2/3 L h^{3/2} + 8/15 Z h^{5/2})$$

Where Q= discharge through the spillway m^3/s
 L = horizontal bottom width of the spillway m
 h = depth of design flow m
 z = horizontal/vertical side slope (recommended to be 3) m

$$Q_{100 \text{ post}} = 2.42 \text{ m}^3/\text{s}$$

$$Q_{\text{spill}} = 3.28 \text{ m}^3/\text{s}$$

$$h = 0.25 \text{ m}$$

$$z = 3 \quad 1V:3H \text{ slope}$$

$$L = 15.0 \text{ m} \quad \text{trial and error} \quad \text{OK}$$

$$\text{Weir width} = L = 15 \text{ m}$$



MAVEN ASSOCIATES

Job Number
180010

Sheet
1

Rev
A

Job Title
Calc Title

Te Kowhai East
Wetland 3 Outlet Controls
EDD Orifice & Emergency Spillway

Author
KY

Date
1-Mar

Checked

Wetland 3

VOLUMES

Min. WQV	2977 m ³	From 'Wetland Sizing' Spreadsheet
Min. EDV	3572 m ³	1.2WQV

LEVELS

PWL	26.20 m	Design based on 3D Model
EDD	26.70 m	0.5 (m) EDD Depth
2YR Water Level	27.31 m	Based on TMW TUFLOW Modelling
10YR Water Level	27.61 m	Based on TMW TUFLOW Modelling
100YR Water Level	28.06 m	Based on TMW TUFLOW Modelling
Spillway Invert	28.20 m	(Min.) 100mm above 100YR WL
Spillway Water Level	28.40 m	Design based on 3D Model
Top of Wetland	28.70 m	0.30 (m) Freeboard

FLOWS

Q ₂ post	m ³ /s	
Q ₁₀ post	m ³ /s	
Q ₁₀₀ post	1.89 m ³ /s	From TMW Hydrograph

CALC 1: Extended Detention Orifice

Outlet to be sized to release the EDV over a 24-hour period

$$Q_{\text{EDD}} = \frac{3572 \text{ m}^3 \text{ over } 24 \text{ hours}}{0.04 \text{ m}^3/\text{s}}$$

At full EDV elev., maximum release rate is assumed to be $Q_{\text{max}} = 2 \cdot Q_{\text{EDD}}$

$$Q_{\text{max}} = 0.0827 \text{ m}^3/\text{s}$$

Vol. of storage savings can be realised if edv is determined by routing flows through the pond

$$Q_i = n \cdot 0.62 \cdot A \cdot (2 \cdot g \cdot h_i)^{0.5} \quad \text{where } A = \text{area of EDD orifice}$$

$$n = 4 \text{ No. of orifices}$$

$$\text{orifice dia} = 0.12 \text{ m}$$

$$A = 0.0113097 \text{ m}^2$$

$$h_i = 0.44 \text{ m}$$

Concept only. Provide routing via HMS
to limit EDD depth to 0.35m at EPA design

$$Q_i = 0.0824 \text{ m}^3/\text{s} \quad \text{OK}$$

Use **4** x **120** mm orifice

CALC 2: Emergency Spillway

$$Q_{\text{spill}} = 0.57 (2g)^{1/2} (2/3 L h^{3/2} + 8/15 Z h^{5/2})$$

Where Q= discharge through the spillway m^3/s
 L = horizontal bottom width of the spillway m
 h = depth of design flow m
 z = horizontal/vertical side slope (recommended to be 3) m

$$Q_{100 \text{ post}} = 1.89 \text{ m}^3/\text{s}$$

$$Q_{\text{spill}} = 2.33 \text{ m}^3/\text{s}$$

$$h = 0.20 \text{ m}$$

$$z = 3 \quad 1V:3H \text{ slope}$$

$$L = 15.0 \text{ m} \quad \text{trial and error} \quad \text{OK}$$

$$\text{Weir width} = L = 15 \text{ m}$$



MAVEN ASSOCIATES

Job Number
180010

Sheet
1

Rev
A

Job Title
Calc Title

Te Kowhai East
Wetland 4 Outlet Controls
EDD Orifice & Emergency Spillway

Author
KY

Date
1-Mar

Checked

Wetland 4

VOLUMES

Min. WQV	5474 m ³	From 'Wetland Sizing' Spreadsheet
Min. EDV	6569 m ³	1.2WQV

LEVELS

PWL	23.55 m	Design based on 3D Model
EDD	24.00 m	0.45 (m) EDD Depth
2YR Water Level	24.95 m	Based on TMW TUFLOW Modelling
10YR Water Level	25.37 m	Based on TMW TUFLOW Modelling
100YR Water Level	25.90 m	Based on TMW TUFLOW Modelling
Spillway Invert	26.00 m	(Min.) 100mm above 100YR WL
Spillway Water Level	26.25 m	Design based on 3D Model
Top of Wetland	26.25 m	0.00 (m) Freeboard

Raise Maintenance Track RL at detailed design EPA to achieve freeboard

FLOWS

Q ₂ post	m ³ /s	
Q ₁₀ post	m ³ /s	
Q ₁₀₀ post	6.64 m ³ /s	From TMW Hydrograph

CALC 1: Extended Detention Orifice

Outlet to be sized to release the EDV over a 24-hour period

$$Q_{\text{EDD}} = \frac{6569 \text{ m}^3 \text{ over } 24 \text{ hours}}{0.08 \text{ m}^3/\text{s}}$$

At full EDV elev., maximum release rate is assumed to be $Q_{\text{max}} = 2 * Q_{\text{EDD}}$

$$Q_{\text{max}} = 0.1521 \text{ m}^3/\text{s}$$

Vol. of storage savings can be realised if edv is determined by routing flows through the pond

$$Q_i = n * 0.62 * A * (2 * g * h_i)^{0.5} \quad \text{where } A = \text{area of EDD orifice}$$

$$n = 5 \text{ No. of orifices}$$

$$\text{orifice dia} = 0.14 \text{ m}$$

$$A = 0.0153938 \text{ m}^2$$

$$h_i = 0.38 \text{ m}$$

Concept only. Provide routing via HMS to limit EDD depth to 0.35m at EPA design

$$Q_i = 0.1303 \text{ m}^3/\text{s} \quad \text{OK}$$

Use **5** x **140** mm orifice

CALC 2: Emergency Spillway

$$Q_{\text{spill}} = 0.57 (2g)^{1/2} (2/3 L h^{3/2} + 8/15 Z h^{5/2})$$

Where Q= discharge through the spillway m^3/s
 L = horizontal bottom width of the spillway m
 h = depth of design flow m
 z = horizontal/vertical side slope (recommended to be 3) m

$$Q_{100 \text{ post}} = 6.64 \text{ m}^3/\text{s}$$

$$Q_{\text{spill}} = 7.49 \text{ m}^3/\text{s}$$

$$h = 0.25 \text{ m}$$

$$z = 3 \quad 1V:3H \text{ slope}$$

$$L = 35.0 \text{ m} \quad \text{trial and error} \quad \text{OK}$$

$$\text{Weir width} = L = 35 \text{ m}$$



MAVEN ASSOCIATES

Job Number
180010

Sheet
1

Rev
A

Job Title
Calc Title

Te Kowhai East
Wetland 5 Outlet Controls
EDD Orifice & Emergency Spillway

Author
KY

Date
1-Mar

Checked

Wetland 5

VOLUMES

Min. WQV	3928 m ³	From 'Wetland Sizing' Spreadsheet
Min. EDV	4714 m ³	1.2WQV

LEVELS

PWL	25.10 m	Design based on 3D Model
EDD	25.60 m	0.5 (m) EDD Depth
2YR Water Level	26.24 m	Based on TMW TUFLOW Modelling
10YR Water Level	26.65 m	Based on TMW TUFLOW Modelling
100YR Water Level	27.03 m	Based on TMW TUFLOW Modelling
Spillway Invert	27.30 m	(Min.) 100mm above 100YR WL
Spillway Water Level	27.55 m	Design based on 3D Model
Top of Wetland	27.85 m	0.30 (m) Freeboard

FLOWS

Q ₂ post	m ³ /s	
Q ₁₀ post	m ³ /s	
Q ₁₀₀ post	2.49 m ³ /s	From TMW Hydrograph

CALC 1: Extended Detention Orifice

Outlet to be sized to release the EDV over a 24-hour period

$$Q_{\text{EDD}} = \frac{4714 \text{ m}^3 \text{ over } 24 \text{ hours}}{0.05 \text{ m}^3/\text{s}}$$

At full EDV elev., maximum release rate is assumed to be $Q_{\text{max}} = 2 \cdot Q_{\text{EDD}}$

$$Q_{\text{max}} = 0.1091 \text{ m}^3/\text{s}$$

Vol. of storage savings can be realised if edv is determined by routing flows through the pond

$$Q_i = n \cdot 0.62 \cdot A \cdot (2 \cdot g \cdot h_i)^{0.5} \quad \text{where } A = \text{area of EDD orifice}$$

$$n = 4 \text{ No. of orifices}$$

$$\text{orifice dia} = 0.13 \text{ m}$$

$$A = 0.0132732 \text{ m}^2$$

$$h_i = 0.435 \text{ m}$$

Concept only. Provide routing via HMS
to limit EDD depth to 0.35m at EPA design

$$Q_i = 0.0962 \text{ m}^3/\text{s} \quad \text{OK}$$

Use **4** x **130** mm orifice

CALC 2: Emergency Spillway

$$Q_{\text{spill}} = 0.57 (2g)^{1/2} (2/3 L h^{3/2} + 8/15 Z h^{5/2})$$

Where Q= discharge through the spillway m^3/s
 L = horizontal bottom width of the spillway m
 h = depth of design flow m
 z = horizontal/vertical side slope (recommended to be 3) m

$$Q_{100 \text{ post}} = 2.49 \text{ m}^3/\text{s}$$

$$Q_{\text{spill}} = 3.28 \text{ m}^3/\text{s}$$

$$h = 0.25 \text{ m}$$

$$z = 3 \quad 1V:3H \text{ slope}$$

$$L = 15.0 \text{ m} \quad \text{trial and error} \quad \text{OK}$$

$$\text{Weir width} = L = 15 \text{ m}$$



MAVEN ASSOCIATES

Job Number
180010

Sheet
1

Rev
A

Job Title
Calc Title

Te Kowhai East
Wetland 6 Outlet Controls
EDD Orifice & Emergency Spillway

Author
KY

Date
1-Mar

Checked

Wetland 6

VOLUMES

Min. WQV	5123 m ³	From 'Wetland Sizing' Spreadsheet
Min. EDV	6148 m ³	1.2WQV

LEVELS

PWL	25.40 m	Design based on 3D Model
EDD	25.90 m	0.5 (m) EDD Depth
2YR Water Level	26.59 m	Based on TMW TUFLOW Modelling
10YR Water Level	26.93 m	Based on TMW TUFLOW Modelling
100YR Water Level	27.32 m	Based on TMW TUFLOW Modelling
Spillway Invert	27.45 m	(Min.) 100mm above 100YR WL
Spillway Water Level	27.70 m	Design based on 3D Model
Top of Wetland	28.00 m	0.30 (m) Freeboard

FLOWS

Q ₂ post	m ³ /s	
Q ₁₀ post	m ³ /s	
Q ₁₀₀ post	4.74 m ³ /s	From TMW Hydrograph

CALC 1: Extended Detention Orifice

Outlet to be sized to release the EDV over a 24-hour period

$$Q_{\text{EDD}} = \frac{6148 \text{ m}^3 \text{ over } 24 \text{ hours}}{0.07 \text{ m}^3/\text{s}}$$

At full EDV elev., maximum release rate is assumed to be $Q_{\text{max}} = 2 \cdot Q_{\text{EDD}}$

$$Q_{\text{max}} = 0.1423 \text{ m}^3/\text{s}$$

Vol. of storage savings can be realised if edv is determined by routing flows through the pond

$$Q_i = n \cdot 0.62 \cdot A \cdot (2 \cdot g \cdot h_i)^{0.5} \quad \text{where } A = \text{area of EDD orifice}$$

$$n = 4 \text{ No. of orifices}$$

$$\text{orifice dia} = 0.15 \text{ m}$$

$$A = 0.0176715 \text{ m}^2$$

$$h_i = 0.425 \text{ m}$$

Concept only. Provide routing via HMS
to limit EDD depth to 0.35m at EPA design

$$Q_i = 0.1266 \text{ m}^3/\text{s} \quad \text{OK}$$

Use **4** x **150** mm orifice

CALC 2: Emergency Spillway

$$Q_{\text{spill}} = 0.57 (2g)^{1/2} (2/3 L h^{3/2} + 8/15 Z h^{5/2})$$

Where Q= discharge through the spillway m^3/s
 L = horizontal bottom width of the spillway m
 h = depth of design flow m
 z = horizontal/vertical side slope (recommended to be 3) m

$$Q_{100 \text{ post}} = 4.74 \text{ m}^3/\text{s}$$

$$Q_{\text{spill}} = 5.39 \text{ m}^3/\text{s}$$

$$h = 0.25 \text{ m}$$

$$z = 3 \quad 1V:3H \text{ slope}$$

$$L = 25.0 \text{ m} \quad \text{trial and error} \quad \text{OK}$$

$$\text{Weir width} = L = 25 \text{ m}$$



MAVEN ASSOCIATES

Job Number
180010

Sheet
1

Rev
A

Job Title
Calc Title

Te Kowhai East
Wetland 7 Outlet Controls
EDD Orifice & Emergency Spillway

Author
KY

Date
1-Mar

Checked

Wetland 7

VOLUMES

Min. WQV	1793 m ³	From 'Wetland Sizing' Spreadsheet
Min. EDV	2152 m ³	1.2WQV

LEVELS

PWL	28.20 m	Design based on 3D Model
EDD	28.50 m	0.3 (m) EDD Depth
2YR Water Level	28.96 m	Based on TMW TUFLOW Modelling
10YR Water Level	29.20 m	Based on TMW TUFLOW Modelling
100YR Water Level	29.48 m	Based on TMW TUFLOW Modelling
Spillway Invert	29.60 m	(Min.) 100mm above 100YR WL
Spillway Water Level	29.75 m	Design based on 3D Model
Top of Wetland	30.10 m	0.35 (m) Freeboard

FLOWS

Q ₂ post	m ³ /s	
Q ₁₀ post	m ³ /s	
Q ₁₀₀ post	1.40 m ³ /s	From TMW Hydrograph

CALC 1: Extended Detention Orifice

Outlet to be sized to release the EDV over a 24-hour period

$$Q_{\text{EDD}} = \frac{2152 \text{ m}^3 \text{ over } 24 \text{ hours}}{0.02 \text{ m}^3/\text{s}}$$

At full EDV elev., maximum release rate is assumed to be $Q_{\text{max}} = 2 \cdot Q_{\text{EDD}}$

$$Q_{\text{max}} = 0.0498 \text{ m}^3/\text{s}$$

Vol. of storage savings can be realised if edv is determined by routing flows through the pond

$$Q_i = n \cdot 0.62 \cdot A \cdot (2 \cdot g \cdot h_i)^{0.5} \quad \text{where } A = \text{area of EDD orifice}$$

$$n = 4 \text{ No. of orifices}$$

$$\text{orifice dia} = 0.1 \text{ m}$$

$$A = 0.007854 \text{ m}^2$$

$$h_i = 0.25 \text{ m}$$

Concept only. Provide routing via HMS
to limit EDD depth to 0.35m at EPA design

$$Q_i = 0.0431 \text{ m}^3/\text{s} \quad \text{OK}$$

Use **4** x **100** mm orifice

CALC 2: Emergency Spillway

$$Q_{\text{spill}} = 0.57 (2g)^{1/2} (2/3 L h^{3/2} + 8/15 Z h^{5/2})$$

Where Q= discharge through the spillway m^3/s
 L = horizontal bottom width of the spillway m
 h = depth of design flow m
 z = horizontal/vertical side slope (recommended to be 3) m

$$Q_{100 \text{ post}} = 1.40 \text{ m}^3/\text{s}$$

$$Q_{\text{spill}} = 1.50 \text{ m}^3/\text{s}$$

$$h = 0.15 \text{ m}$$

$$z = 3 \quad 1\text{V}:3\text{H slope}$$

$$L = 15.0 \text{ m} \quad \text{trial and error} \quad \text{OK}$$

$$\text{Weir width} = L = 15 \text{ m}$$



MAVEN ASSOCIATES

Job Number
180010

Sheet
1

Rev
A

Job Title
Calc Title

Te Kowhai East
Wetland 8 Outlet Controls
EDD Orifice & Emergency Spillway

Author
KY

Date
1-Mar

Checked

Wetland 8

VOLUMES

Min. WQV	3190 m ³	From 'Wetland Sizing' Spreadsheet
Min. EDV	3829 m ³	1.2WQV

LEVELS

PWL	26.35 m	Design based on 3D Model
EDD	26.80 m	0.45 (m) EDD Depth
2YR Water Level	27.42 m	Based on TMW TUFLOW Modelling
10YR Water Level	27.76 m	Based on TMW TUFLOW Modelling
100YR Water Level	28.17 m	Based on TMW TUFLOW Modelling
Spillway Invert	28.35 m	(Min.) 100mm above 100YR WL
Spillway Water Level	28.55 m	Design based on 3D Model
Top of Wetland	28.90 m	0.35 (m) Freeboard

FLOWS

Q ₂ post	m ³ /s	
Q ₁₀ post	m ³ /s	
Q ₁₀₀ post	2.13 m ³ /s	From TMW Hydrograph

CALC 1: Extended Detention Orifice

Outlet to be sized to release the EDV over a 24-hour period

$$Q_{\text{EDD}} = \frac{3829 \text{ m}^3 \text{ over } 24 \text{ hours}}{0.04 \text{ m}^3/\text{s}}$$

At full EDV elev., maximum release rate is assumed to be $Q_{\text{max}} = 2 \cdot Q_{\text{EDD}}$

$$Q_{\text{max}} = 0.0886 \text{ m}^3/\text{s}$$

Vol. of storage savings can be realised if edv is determined by routing flows through the pond

$$Q_i = n \cdot 0.62 \cdot A \cdot (2 \cdot g \cdot h_i)^{0.5} \quad \text{where } A = \text{area of EDD orifice}$$

$$n = 4 \text{ No. of orifices}$$

$$\text{orifice dia} = 0.12 \text{ m}$$

$$A = 0.0113097 \text{ m}^2$$

$$h_i = 0.39 \text{ m}$$

Concept only. Provide routing via HMS
to limit EDD depth to 0.35m at EPA design

$$Q_i = 0.0776 \text{ m}^3/\text{s} \quad \text{OK}$$

Use **4** x **120** mm orifice

CALC 2: Emergency Spillway

$$Q_{\text{spill}} = 0.57 (2g)^{1/2} (2/3 L h^{3/2} + 8/15 Z h^{5/2})$$

Where Q= discharge through the spillway m^3/s
 L = horizontal bottom width of the spillway m
 h = depth of design flow m
 z = horizontal/vertical side slope (recommended to be 3) m

$$Q_{100 \text{ post}} = 2.13 \text{ m}^3/\text{s}$$

$$Q_{\text{spill}} = 2.33 \text{ m}^3/\text{s}$$

$$h = 0.20 \text{ m}$$

$$z = 3 \quad 1V:3H \text{ slope}$$

$$L = 15.0 \text{ m} \quad \text{trial and error} \quad \text{OK}$$

$$\text{Weir width} = L = 15 \text{ m}$$

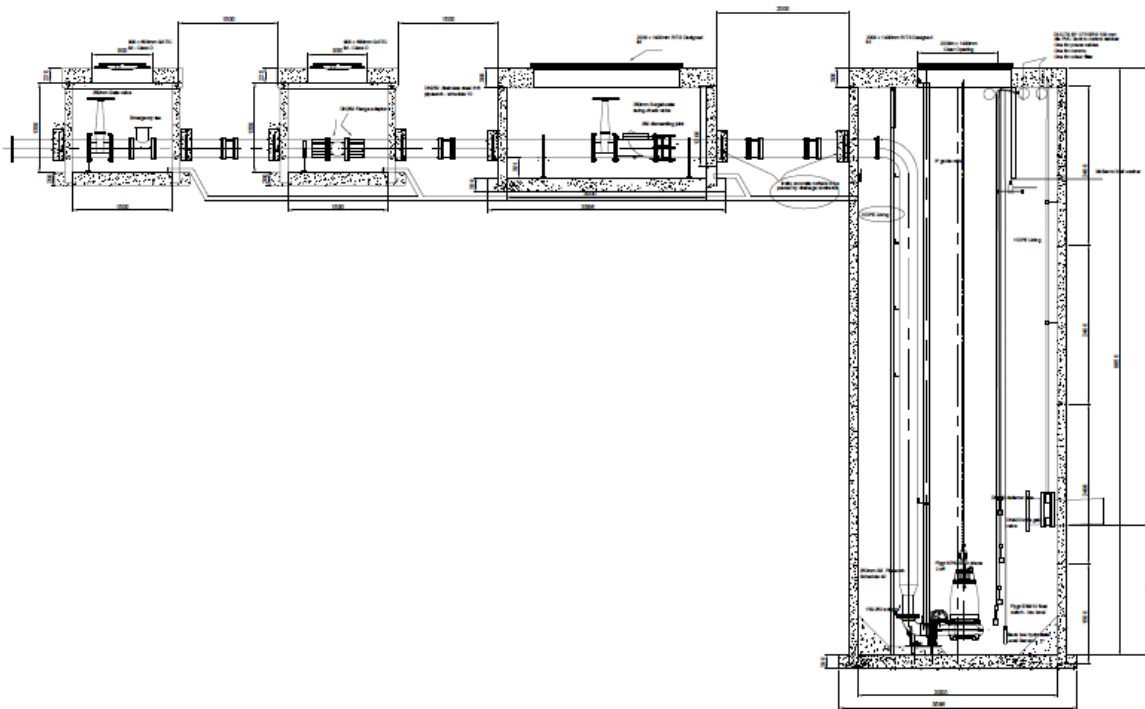
APPENDIX C.6 – WW PUMPSTATION SIZING

 Maven Associates		Job Number 180010	Sheet 1	Rev A
Job Title Calc Title	TKE Development Wastewater Demand Calc	Author JD	Date 5/06/2026	Checked RW
As per Waikato Local Authority RITS standards Domestic Average Daily Flow (Water Consumption)= 200 <i>l/person/day</i> RITS V2 Infiltration Allowance= 0 <i>l/Ha/day</i> RITS V2 Surface Water Ingress= 0 <i>l/Ha/day</i>				
Development Area using Gravity/Pumpstation Industrial Catchment area = 125.00 Ha Population Equivalent as per Table 91= 45 <i>persons per hectare</i> Total Population 5625 Wastewater Peaking Factor (RITS V2)= 6.7 Average Daily Flow (ADF)= 1125.00 <i>m³/day</i> 13.02 <i>L/sec</i> Peak Daily Flow (PDF)= TBC Peak Wet Weather Flow (PWWF)= 87.24 <i>L/sec</i>				
PWW Flow <i>l/s</i> Proposed Gravity Pipe dia <i>m</i> Gradient % Capacity <i>l/s</i> Velocity <i>m/s</i> Check OK 87.24 0.3 1.00 98.35 1.39 OK				
Emergency Storage is 9 hours of ADF 421.9 m3				

1 Technical Specifications

1.1 Storage Chamber

Tank Volume:	60.85 m ³
Effective volume:	53.7 m ³
Storage tank vol:	371.8 m ³
Tank material:	Concrete
Manufacturing Std:	AS/NZS 4058:2007 and NZS 3101
Loading:	The standard lid is suitable for pedestrian loading only.
Inlet Connections:	the inlet pipe to be installed on site and sealed by the drainage contractor.
Outlet connection:	A flanged stainless steel is installed for connection to the outlet pressure pipe
Venting:	External venting required (see section 1.6 for venting details if required)
Dimensions:	see drawing below



1.2 Pumps/Pump Performance

See full specifications in the Appendices

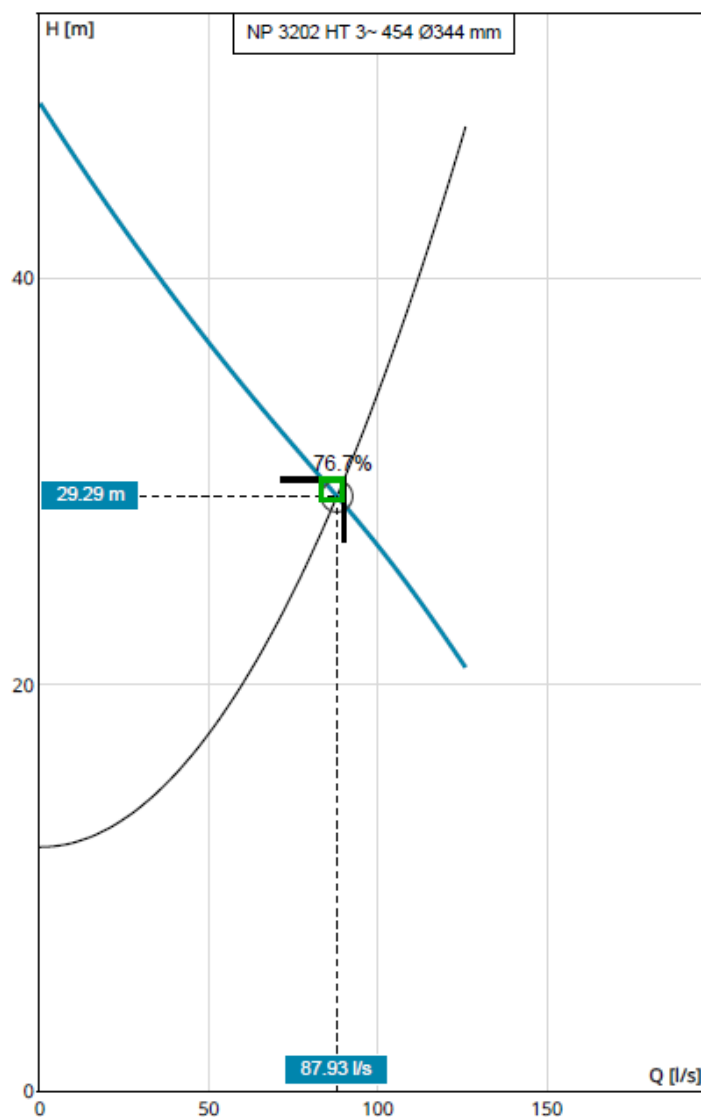
Manufacturer: Xylem Pumps

Type: Flygt NP3202 HT3 454

Connection: 415 V, 50 Hz

Power: 45 kW

Phases: Three



1.3 Design Information

1.3.1 Storage Calculations

The total Average Dry Weather flow has been calculated from the expected flow rate:

Average dry weather flow= 1125 m³/day

Total emergency storage required is 9 hours = 421.9 m³

Total emergency storage in wet-well and emergency storage tank is 125.5 m³ so 9.1 hours storage has been provided.

1.3.2 Pump Design

The system above has been designed based on the information provided below.

Rising main length : 4170 m

Static Lift : – 12 m

Rising main – PE100 (HDPE) PE 4710 / SDR 11 (PN 16) / DN 400 (400x36.3 mm)

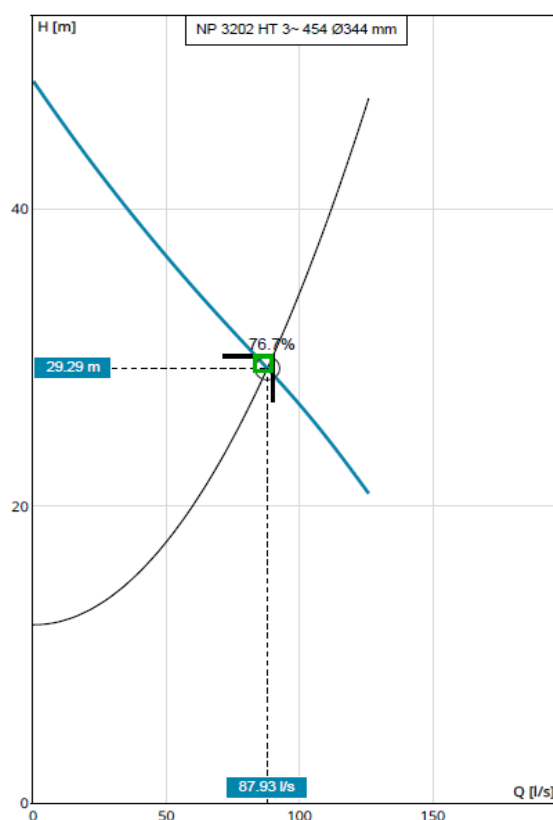
Pipe OD = 400mm , Pipe ID = 327.4 mm

$K = 0.6$

Flygt NP3202 - Flow rate = 87.9 l/s with a total head loss of 29.3 m (single pump).

Pumps to operate in a duty / assist / standby function

Velocity in PE pipe = 1.1 m/s



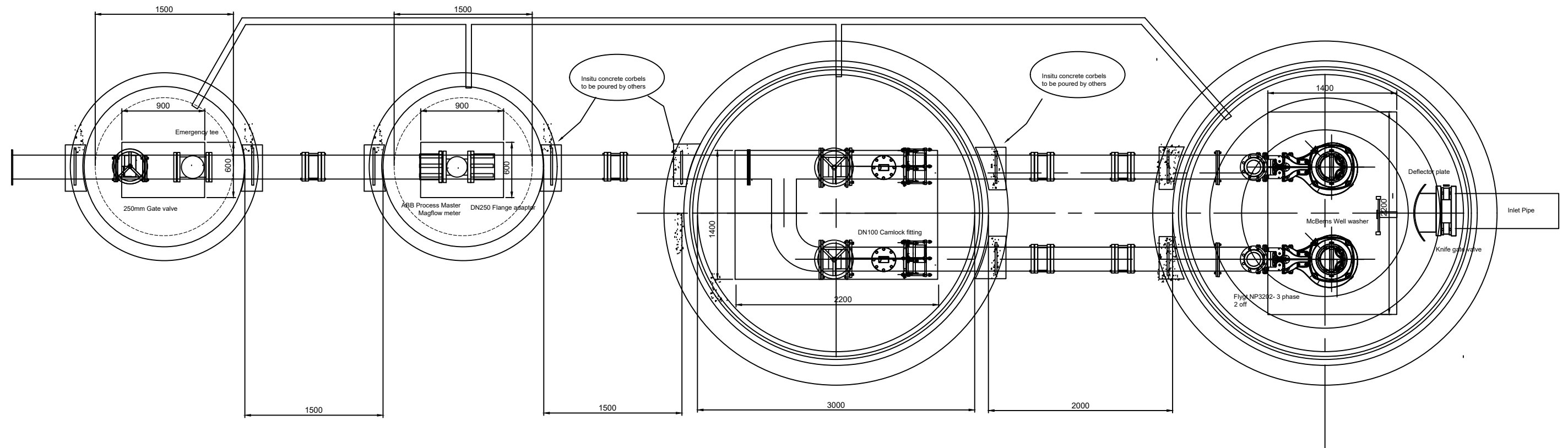
1.3.3 Pump Operational Levels

The operational levels will be set as follows (dimensions taken from floor of wet-well) :

- High level alarm - 1150mm
- Assist Pump Start – 1050mm
- Duty Pump Start – 1000mm
- Both Pumps Stop – 350mm
- Low Level alarm : 300mm

1.3.4 Pump Stop/Start Calculations

Expected daily flow:	1124928	litres
Infiltration (l/day)	0	litres
SWI (l/day)	0	litres
Peaking factor :	6.7	
ADDWF	13.02	l/s
PDDWF	34.79	l/s
PWWF	87.23	l/s
Pump Station diameter	3	m
Pump station depth (total)	8.6	m
Pump station depth (effective)	7.6	m
Volume (effective)	53.69	m ³
Pump Stop Level	0.35	m
Pump Start Level	1	m
Volume to pump:	4592	litres
Discharge rate:	87.9	l/s
Time to empty: (PDDWF)	1.4	minutes
Time to empty: (PWWF)	114.9	minutes
Time to fill (PDDWF)	2.2	minutes
Time to fill (PWWF)	0.9	minutes
No starts/ hour (PDDWF)	16.5	
No starts/ hour (PWWF)	0.5	
No starts per pump (PDDWF)	8.2	
No starts per pump (PWWF)	0.3	
Max Start/Hour permissible	30	Okay



NOTES:

1.

REVISIONS

REV #	REVISION DESCRIPTION	DATE	DRAWN
A	Prelim design	8/6/26	PC

PO Box 58142, Botany, Auckland, 2316
Tel: 09-274 0316
Fax: 09-272 7485
email: hwwsupport@hynds.co.nz

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ISO 9001 CERTIFIED MANAGEMENT SYSTEM



PROJECT:
Te Kowhai Pump Station
Te Kowhai East Industrial project
Te Kowhai

DRAWING TITLE:
Plan View

REFERENCE/QUOTE NUMBER:	AQ2291 - 1	
DRAWN: PC	DESIGN: PC	CHECKED: JB
SIGNATURE:	SIGNATURE:	SIGNATURE:
SCALE: As Shown	Note: Do not scale drawing if in doubt ASK!!!	DATE: 23/6/25
PAPER SIZE: A3		
DRAWING NUMBER: AQ2291 - 13	SHEET NUMBER: of 4	REVISION NUMBER: A

APPENDIX G – SERVICE PROVIDER CORRESPONDENCE

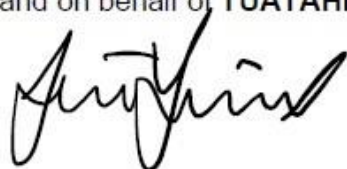
7 November 2025

**CONDITIONAL ACCEPTANCE BY TUATAHI FIRST FIBRE LIMITED AS
TELECOMMUNICATIONS OPERATOR****Development:** Te Kowhai East Development**Legal Description:** LOTS 2 & 3 DP 523063, PT ALLOT 215 PSH OF PUKETE, LOT 1 DP
546273, SEC 1 & 2 SO 473918, SEC 2 SO 577981, SEC 2 SO 473918,
SEC 25 & 26 SO 417000 & SEC 13 SO 416124

1. Tuatahi First Fibre Limited (TFF) confirms that a TFF telecommunications connection will be made available for each site in the development, **providing the developer was to sign a TFF Installation Agreement**. Upon approval of this agreement, TFF will undertake to become the telecommunications operator of the telecommunications reticulation in the proposed development and any proposed public roads at Te Kowhai East (the "**Subdivision**"), to provide network connections to all lots / units in the Subdivision (the "**Reticulation**").
2. The Reticulation will be installed in accordance with:
 - (a) the requirements and standards set by the Waikato District Council and advised to TFF via the Council's website; and
 - (b) the requirements of the Telecommunications Act 2001 and all other applicable laws, regulations and codes (as amended).
3. The Reticulation will be installed by our preferred provider to TFF's satisfaction.
4. TFF will be the owner, operator and maintainer of the Reticulation.
5. One or more retail service providers will be available to supply telecommunications services over the completed Reticulation when service is available, provided that TFF shall not be responsible if the retail service provider's offer to supply such telecommunications services or the number of such providers varies from time to time.

SIGNED for and on behalf of **TUATAHI FIRST FIBRE LIMITED** by:

Signature:



Name:

Dan Fenwick

2 April 2026

Jon Crooks
Te Kowahi East LP
5 Owens Rd
AUCKLAND

Dear Jon

RE: PROPOSED SUBDIVISIONS – Te Kowhai East LP - Te Kowhai fast track application

Thank you for your enquiry regarding the power availability for the proposed development of the Te Kowhai area.

We have investigated the electricity supply requirements for the above proposed subdivision and we are able to supply the electrical reticulation.

Te Kowhai East (Green) – 188 ha of Light Industrial Land

Available Capacity: 9 MVA sufficient to cover approximately 70-75 hectares depending on individual load requirements of the expected commercial/industrial customers

Infrastructure Requirements: Installation of approximately 8.6 km of (2 x11 kV) cables up to customers' first RMU.

Note: The above load and neighbouring developments would be served on first come first served bases. The rest of the load would be supplied from New Exelby zone substation which is currently planned for installation by 2033. Alternatively, this can be brought forward as development progresses.

We thank you for your enquiry. If you have any further queries or require additional information, please do not hesitate to contact me.

Yours faithfully

A handwritten signature in black ink, appearing to read 'Miranda McLean'.

Miranda McLean
PROJECT MANAGER



APPENDIX H – WATER SUPPLY MODELLING MEMO

WATER SUPPLY NETWORK MODELLING REPORT

TE KOWHAI EAST DEVELOPMENT TE KOWHAI, HAMILTON

 M A E N		Job Number 180010		Rev A
Job Title Title	Te Kowhai East Development Water Supply Network Modelling Report	Author AYC	Date 14/08/2026	Checked JZ

1. INTRODUCTION

1.1 PURPOSE

The purpose of this report is to present the hydraulic assessment of the proposed pressurised water supply network for the Te Kowhai East (TKE) Development. The assessment has been undertaken to demonstrate that the proposed internal reticulation network is capable of supplying:

- domestic peak water demand in accordance with the **Regional Infrastructure Technical Specifications (RITS)**; and
- firefighting demand in accordance with **SNZ PAS 4509** New Zealand Fire Service Firefighting Water Supplies Code of Practice.

The hydraulic assessment has been completed using EPANET and considers the Ultimate development scenario.

1.2 PROJECT BACKGROUND

The Te Kowhai East (TKE) Development is located northwest of Hamilton City, adjacent to Te Kowhai Road. The site currently lies outside the Hamilton City Council boundary but has been identified as a strategic growth area. Hamilton City Council is assessing the concept development plan to determine whether the proposed development can be integrated into the future urban expansion of the city.

The proposed development comprises approximately 125 ha of net commercial and industrial land and will be delivered in several stages. The overall development layout and staging plan are presented in **Figure 1**. The proposed water supply network has therefore been assessed for both:

- Ultimate development (approximately 125 ha net developable area).

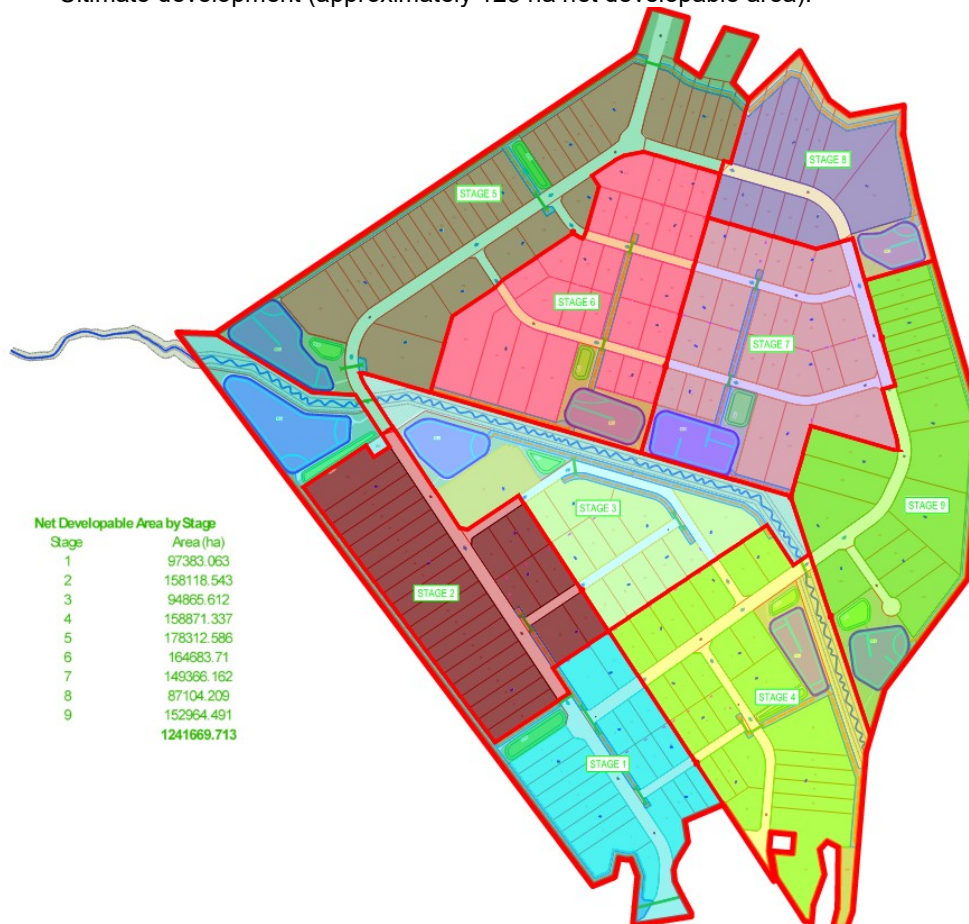


Figure 1: Development Plan

2. PROPOSED INTERNAL WATER SUPPLY NETWORK

2.1 PROPOSED WATER SUPPLY LAYOUT

The proposed internal water supply network has been designed to provide adequate capacity for both domestic water demand and firefighting requirements corresponding to Fire Water Classification **FW3**.

The network comprises PE100 pressure mains supplied from a proposed bulk water supply connection located on Te Kowhai Road. The proposed reticulation has been designed to:

- provide sufficient capacity during peak domestic demand;
- maintain adequate residual pressure throughout the network;
- provide a looped reticulation layout where practicable to improve operational reliability; and
- satisfy the firefighting requirements specified in **SNZ PAS 4509**.

The proposed water supply network layout and pipe sizes are shown in **Figure 2** and on Drawings **C600 to C600-5**, included in **Appendix A**.

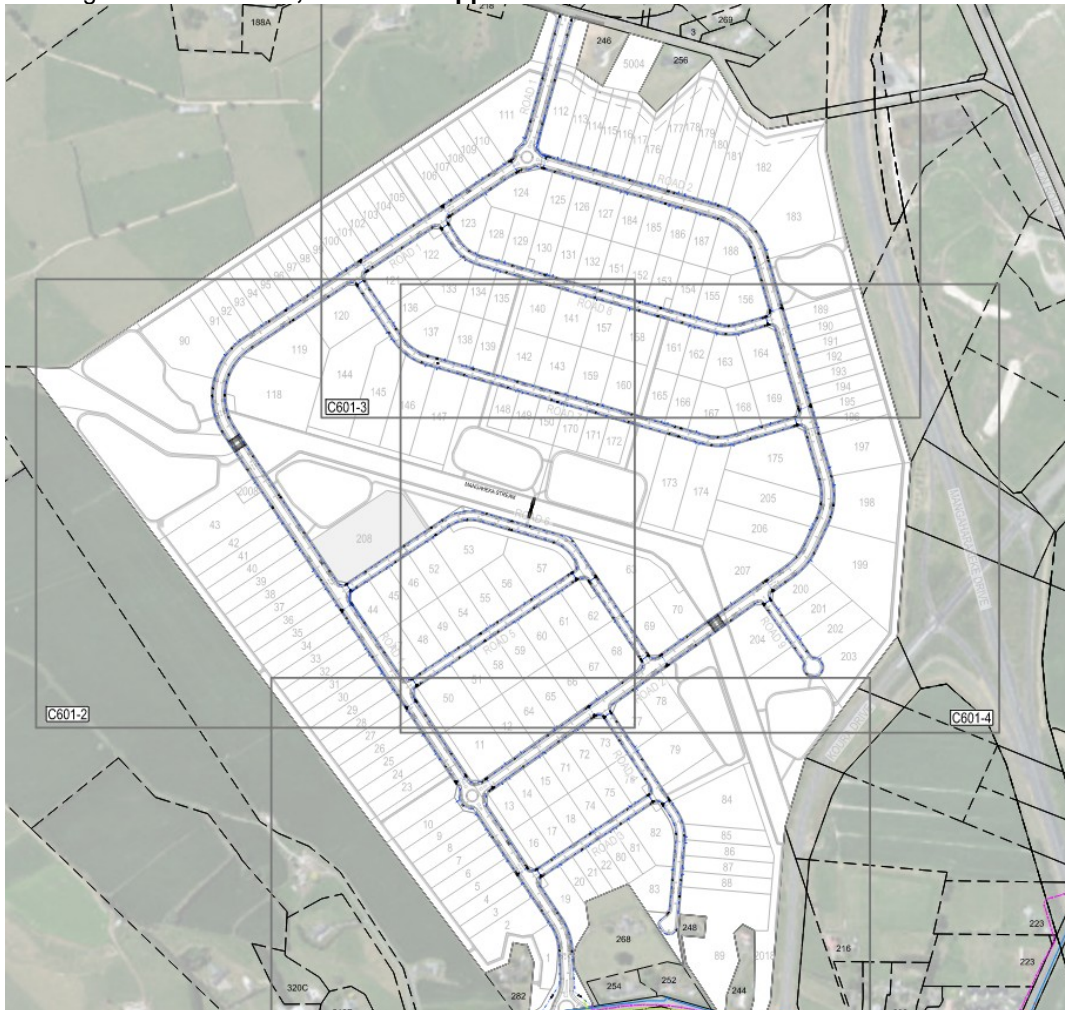


Figure 2: Proposed water supply layout

3. EPANET MODELLING

3.1 MODELLING METHODOLOGY

The proposed water supply network was modelled using EPANET to assess hydraulic performance under both normal operating conditions and firefighting conditions.

The development scenario was assessed:

- Ultimate development (125 ha).

The network was analysed under:

- peak domestic demand; and
- firefighting demand in accordance with **SNZ PAS 4509**.

The objective of the modelling was to determine the minimum residual pressure required at the bulk water supply connection to satisfy the required pressure throughout the internal reticulation network.

3.2 WATER DEMAND ASSESSMENT

Catchment areas and peak demand calculations have been undertaken in accordance with the **RITS**. The catchment plans and detailed demand calculations are included in **Appendix B**. A summary of the adopted design demand is presented in **Table 1**.

Table 1-Water demand assessment

Water Demand	260 Litre/Person/Day
Peaking factor	5
Peaking demand	1300 Litre/Person/Day
Population equivalent	45 persons/ha (Industrial Zone – RITS Table 5-3)
Ultimate developable area	~125 ha
Peak demand (Ultimate)	84.2 L/s

3.3 WATER PIPE SIZES

For hydraulic modelling, the internal pipe diameters were adopted as shown in **Table 2**.

Table 2-Water pipe size used in EAPNET

Pipe	Mean Bore (mm)	Internal Diameter Adopted in EPANET (mm)
180mm (OD) PE100 PN12.5	152.8	150
250mm (OD) PE100 PN12.5	212.4	212

3.4 BULK WATER SUPPLY

The proposed point of supply is located at the entrance to the site from Te Kowhai Road. The bulk supply connection has a ground level of RL 30.852 m. Within the hydraulic model, this location has been represented by a reservoir with a variable total head. The reservoir head was adjusted during the analysis to determine the minimum residual pressure required at the point of supply while maintaining the required pressures throughout the internal reticulation network.

3.5 FIREFIGHTING DEMAND ASSESSMENT

Firefighting demand has been assessed in accordance with **SNZ PAS 4509**. The Code recommends that firefighting assessments be undertaken with domestic demand equal to two-thirds of the annual peak demand. Accordingly, the following domestic demand was adopted:

- Ultimate: 56.1 L/s

To satisfy the requirements of **Fire Water Classification FW3**, an additional firefighting demand of 50 L/s was applied, representing two simultaneous fire hydrants each operating at 25 L/s.

The hydraulic model represents the firefighting demand by applying two simultaneous 25 L/s nodal demands within the network. While individual fire hydrants shown on the engineering drawings were not explicitly modelled, each network junction is considered representative of a

potential hydrant connection. Given the relatively short spacing between hydrants and adjacent junctions, this modelling approach provides an appropriate representation of the hydraulic loading imposed on the reticulation network and is considered sufficient for assessing overall network performance under firefighting conditions.

3.6 GENERAL MODEL PARAMETERS

The hydraulic model was developed using the following assumptions.

Table 3-General model parameters

Parameter	Value
Flow Units	L/s
Elevation Units	m
Pipe Diameter (mm)	Internal Diameter (Table 2)
Headloss Formula	Hazen-Williams Formula
Roughness Coefficient	140 (PE Pipe)
Reservoir head (m)	Variable (determined for each scenario)

3.7 HYDRAULIC MODELLING RESULTS

The EPANET modelling demonstrates that the proposed water supply network can meet the required pressure criteria for both domestic demand and firefighting demand. The minimum residual pressure required at the bulk water supply connection is summarised in **Table 4**.

Table 4-Model Results Summary

	Residual pressure requirement at point of supply (Kpa)	Scenario 1 – Peak domestic demand		Scenario 2 - Firefighting	
		Peak demand (L/s)	Peak Demand Compliance	2/3 of peak flow rate + 50 L/s (L/s)	Firefighting Compliance
Ultimate development (125 ha)	250	84.2	Minimum 200 kPa achieved at all nodes	56.1 + 50	Minimum 100 kPa achieved at all nodes

PEAK DEMAND ASSESSMENT (Under peak domestic demand conditions:)

- all network junctions maintain a residual pressure greater than 200 kPa;
- no significant pressure deficiencies were identified.

The model outputs for peak demand are presented in **Figures 3 and 4** for the ultimate development.

FIREFIGHTING ASSESSMENT (Under firefighting conditions:)

- domestic demand was reduced to two-thirds of peak demand in accordance with **SNZ PAS 4509**;
- two simultaneous nodal demands of 25 L/s each were applied to represent the required **FW3** firefighting demand;
- all junctions maintained a minimum residual pressure of 100 kPa; and
- the proposed network satisfies the hydraulic requirements for **FW3**.

The model outputs for the firefighting scenarios are presented in **Figures 5 and 6** for the ultimate development.

Ultimate development (net area: 125 ha) model results are shown in figure below:

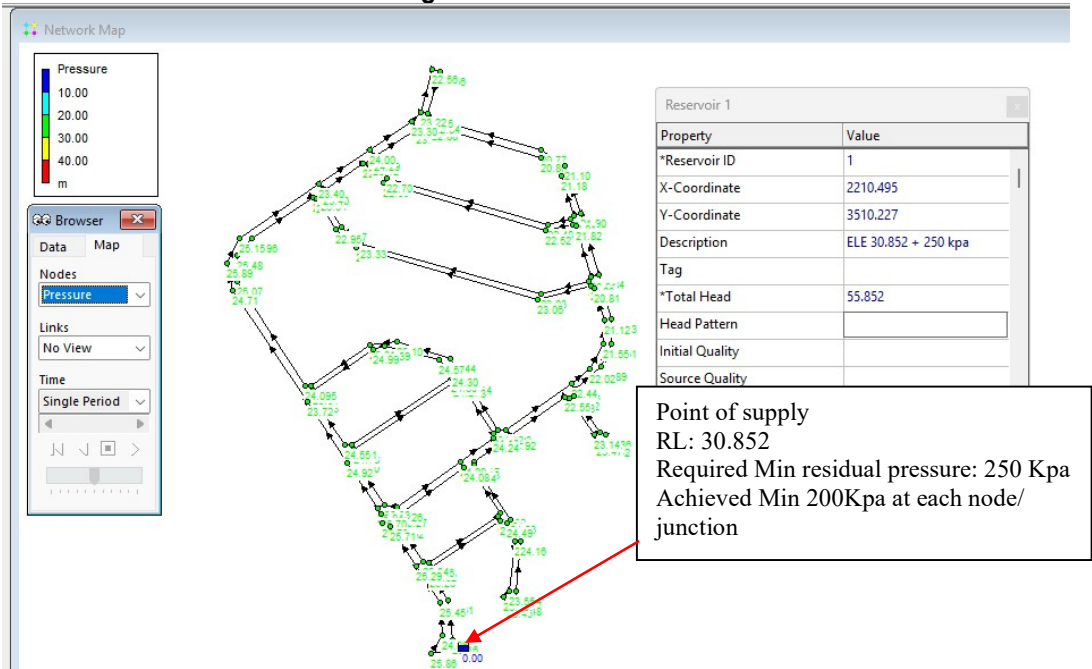
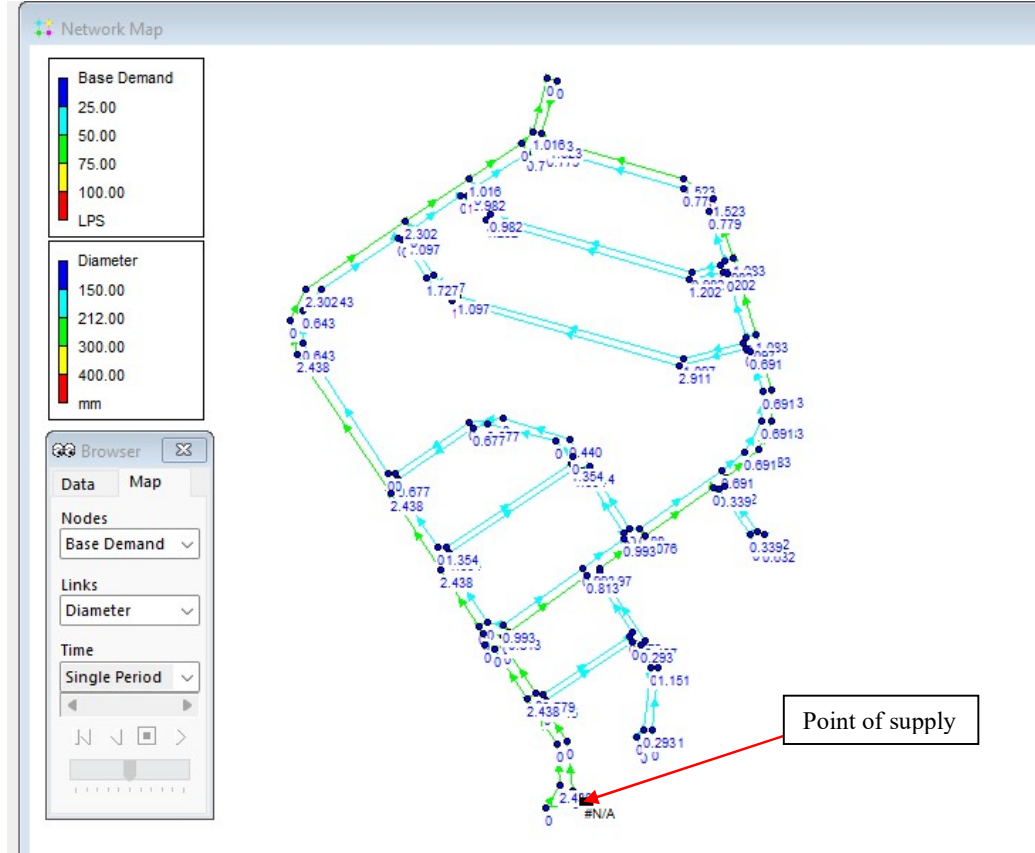


Figure 4: Min 250 Kpa residual pressure for the entire internal network achieved Min 200 Kpa at each node/junction

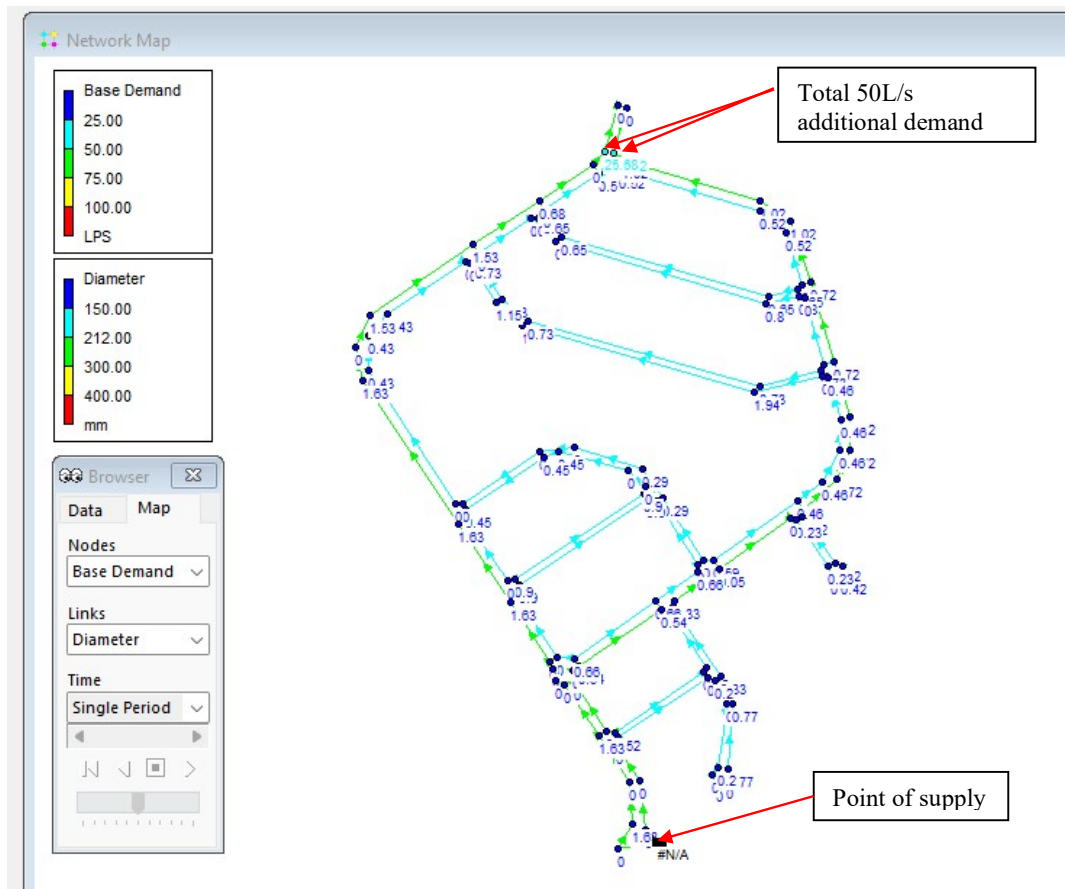


Figure 5: Peak demand at firefighting condition (FW3: 2/3 peak flow rate + additional 50 L/s)

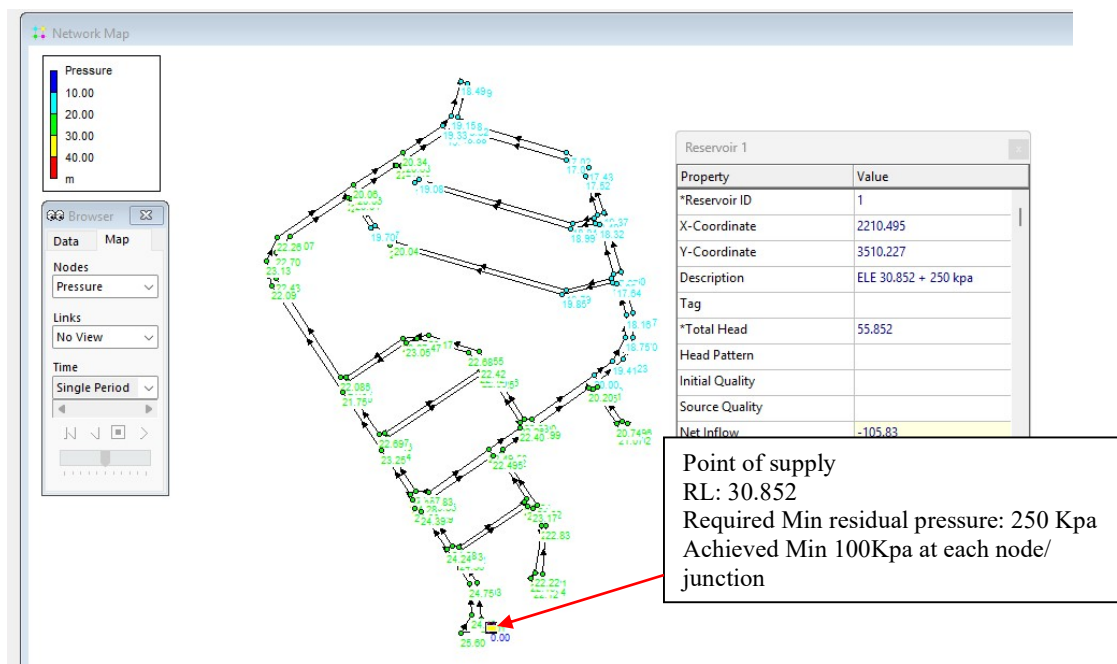


Figure 6: Min 250 Kpa residual pressure for the entire internal network achieved Min 100 Kpa at each node/junction at firefighting condition (FW3: 2/3 peak demand + additional 50 L/s)

3.8 MODEL LIMITATIONS

The EPANET model has been developed to assess the hydraulic performance of the proposed internal water supply network based on the proposed reticulation layout and pipe sizes shown on the engineering drawings refer to **Appendix A**.

For the firefighting assessment, individual fire hydrants were not explicitly included as separate model elements. Instead, firefighting demand was represented by equivalent nodal demands applied at selected junctions within the network. This approach is considered appropriate for hydraulic analysis because the objective is to assess the capacity of the reticulation network to supply the required firefighting flows while maintaining the minimum residual pressures specified by **SNZ PAS 4509**. The modelling methodology provides a representative assessment of network performance and does not materially affect the calculated pressure distribution or headlosses within the system.

The assessment further assumes that the external bulk water supply is capable of providing the minimum residual pressures identified at the proposed point of supply.

4. CONCLUSION

A hydraulic assessment of the proposed Te Kowhai East internal water supply network has been completed using EPANET for the Ultimate development.

The assessment demonstrates that:

- the proposed reticulation network is capable of supplying the calculated peak domestic demand in accordance with the Regional Infrastructure Technical Specifications (RITS);
- the network satisfies the firefighting requirements for Fire Water Classification FW3 in accordance with SNZ PAS 4509;
- for the Ultimate development, a minimum residual pressure of approximately **250 kPa** is required at the bulk supply connection for the Ultimate development to maintain at least **200 kPa** throughout the network during peak domestic demand and **100 kPa** during firefighting conditions

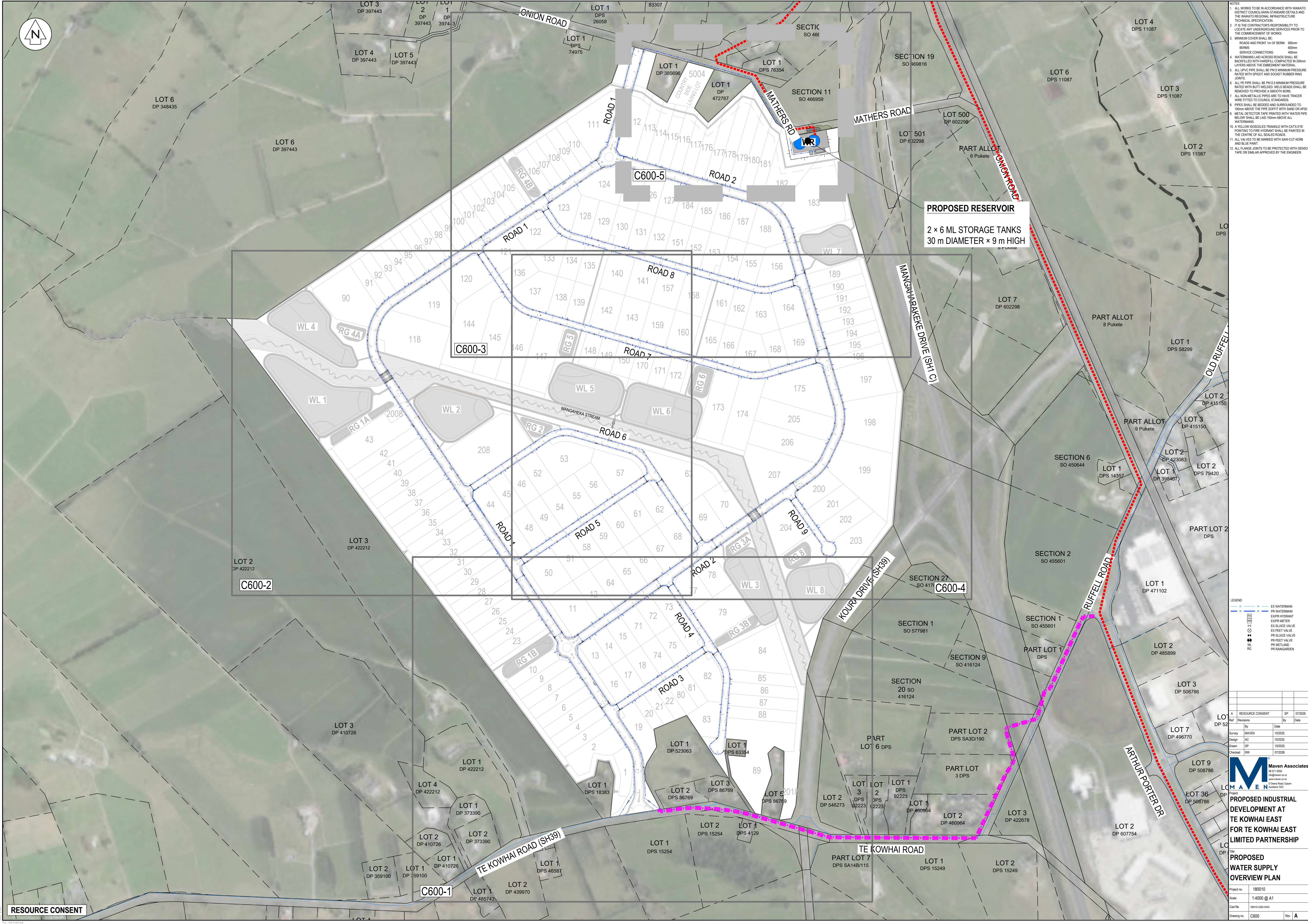
Subject to the bulk water supply being capable of providing the required residual pressures, the proposed internal water supply network is considered hydraulically adequate to service both the staged development and the Ultimate development while meeting the requirements of RITS and SNZ PAS 4509.

Appendices

Appendix A: Proposed water supply network

Appendix B: Catchment plan and detailed calculations

APPENDIX A



- NOTES
1. ALL WORKS TO BE IN ACCORDANCE WITH WAKATO DISTRICT COUNCIL STANDARD DETAILS AND THE WAKATO REGIONAL INFRASTRUCTURE TECHNICAL SPECIFICATION.
 2. IT IS THE CONTRACTOR'S RESPONSIBILITY TO LOCATE ANY UNDERGROUND SERVICES PRIOR TO THE COMMENCEMENT OF WORKS.
 3. MINIMUM COVER SHALL BE:
ROADS AND FRONT 1m OF BERM: 800mm
BERMS: 450mm
SERVICE CONNECTIONS: 450mm
 4. WATERMANS LAD ACROSS ROADS SHALL BE BACKFILLED WITH HESFILL COMPACTED IN 200mm LAYERS ABOVE THE EMBEDEDMENT MATERIAL.
 5. ALL LUPIC PIPE SHALL BE PN10 MINIMUM PRESSURE RATED WITH SPIGOT AND SOCKET RUBBER RING JOINTS.
 6. ALL PE PIPE SHALL BE PN10.5 MINIMUM PRESSURE RATED WITH BUTT WELDED. WELD BEADS SHALL BE REMOVED TO PROVIDE A SMOOTH BORE.
 7. ALL NON-METALLIC PIPES ARE TO HAVE TRACER WIRE FITTED TO COUNCIL STANDARDS.
 8. PIPES SHALL BE BEDDED AND SURROUNDED TO 150mm ABOVE THE PIPE SPOUT WITH SAND OR APPROX. METAL DETECTOR TAPE PRINTED WITH WATER PIPE BELOW SHALL BE LAID 150mm ABOVE ALL WATERMANS.
 9. A YELLOW ISOCELES TRIANGLE WITH CAT'S EYE POINTING TO FIRE HYDRANT SHALL BE PAINTED IN THE CENTRE OF ALL SCALED ROADS.
 11. ALL VALVES TO BE MARKED WITH SAW-CUT KERB AND BLUE PAINT.
 12. ALL FLANGE JOINTS TO BE PROTECTED WITH DENSOTAPE OR SIMILAR APPROVED BY THE ENGINEER.

- LEGEND
- EX WATERMAIN
 - PR WATERMAIN
 - EX/PR HYDRANT
 - EX/PR METER
 - EX SLUICE VALVE
 - PR SLUICE VALVE
 - PR PEET VALVE
 - PR PEET VALVE
 - PR WETLAND
 - PR BANGARDEN

A RESOURCE CONSENT				SP	07/2026
Ref	Revisions	By	Date	By	Date
Survey	MAVEN		10/02/25		
Design	AC		10/02/25		
Drawn	SP		10/02/25		
Checked	RM		07/02/26		

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PROPOSED INDUSTRIAL DEVELOPMENT AT TE KOWHAI EAST FOR TE KOWHAI EAST LIMITED PARTNERSHIP

PROPOSED WATER SUPPLY OVERVIEW PLAN

Project no: 180010
Scale: 1:4000 @ A1
Cad file: 180010-CADD.DWG
Drawing no: C600 Rev: A

RESOURCE CONSENT



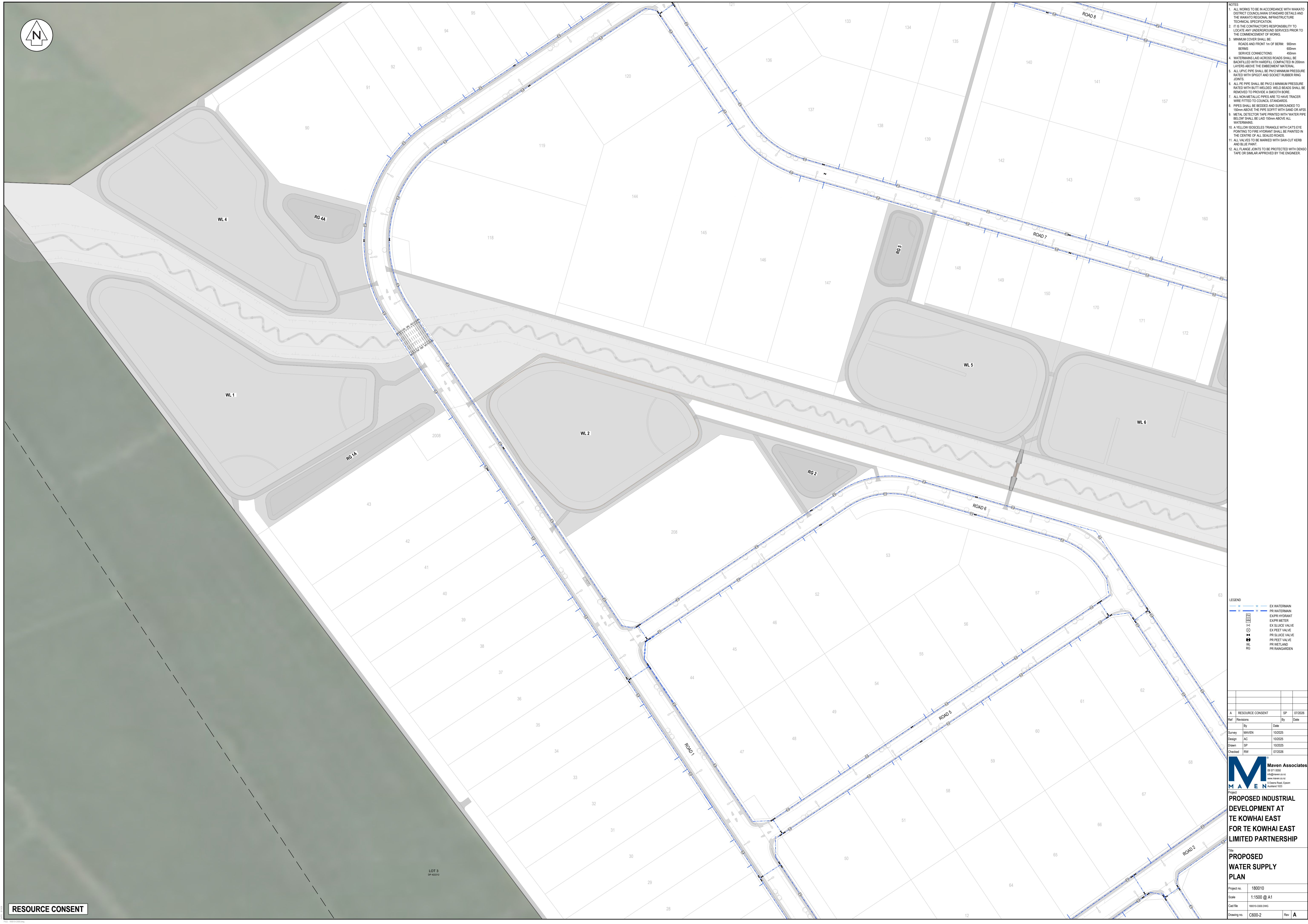
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		W	W		EXN HYDRANT
		W			EXPR METER
		W	W		EX SLUDGE VALVE
		W			EX FEET VALVE
		W	W		PR SLUDGE VALVE
		W			PR FEET VALVE
		W	W		PR PRETREAT
		W			PR PRETAND
		W	W		PR RAINGARDEN



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 Auckland 1023

**PROPOSED INDUSTRIAL
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 FOR TE KOWHAI EAST
 LIMITED PARTNERSHIP**

Title	
PROPOSED WATER SUPPLY PLAN	
Project no.	180010
Scale	1:1500 @ A1
Drawing no.	180010-03-CH02-DWG2
Drawing no.	C600-1
Rev	A



- LEGEND**
- | | |
|---|-----------------|
| | EX WATERMAIN |
|  | PR WATERMAIN |
|  | EX/PR HYDRANT |
|  | EX/PR METER |
|  | EX SLUICE VALVE |
|  | EX PEET VALVE |
|  | PR SLUICE VALVE |
|  | PR PEET VALVE |
|  | WL |
|  | RG |



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**PROPOSED INDUSTRIAL
 DEVELOPMENT AT
 TE KOWHAI EAST
 FOR TE KOWHAI EAST
 LIMITED PARTNERSHIP**

Title
**PROPOSED
 WATER SUPPLY
 PLAN**

Project no. 180110
 Scale 1:1500 @ A1
 Cad file 180110-CADD.DWG
 Drawing no. C600-2

Rev. **A**



- LEGEND
- | | |
|--|----------------|
| | EX WATERMAIN |
| | PR WATERMAIN |
| | EX/PR HYDRANT |
| | EX/PR METER |
| | EX SLUIE VALVE |
| | EX PEET VALVE |
| | PR SLUIE VALVE |
| | PR PEET VALVE |
| | PR WETLAND |
| | PR RAINGARDEN |

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Auckland 1023

Title

**PROPOSED
WATER SUPPLY
PLAN**

Project no.	180010		
Scale	1:1500 @ A1		
Cad file	180010-CB00.DWG		
Drawing no.	C600-3	Rev	A



LEGEND

— W —	EX WATERMAIN
— W —	PR WATERMAIN
— W —	EX/PR HYDRANT
FW	EX/PR METER
⊗	EX SLUICE VALVE
⊗	EX PEET VALVE
⊗	PR SLUICE VALVE
⊗	PR PEET VALVE
WL	PR WETLAND
RG	PR RAINGARDEN

A	RESOURCE CONSENT	SP	07/2026
Ref	Revisions	By	Date
Survey	MAVEN		10/2025
Design	AC		10/2025
Drawn	SP		10/2025
Checked	RW		07/2026



**PROPOSED INDUSTRIAL
DEVELOPMENT AT
TE KOWHAI EAST
FOR TE KOWHAI EAST
LIMITED PARTNERSHIP**

PROPOSED WATER SUPPLY PLAN

Project no.	180010		
Scale	1:1500 @ A1		
Cad file	180010-C600.DWG		
Drawing no.	C600-4	Rev	A



- NOTES
1. ALL WORKS TO BE IN ACCORDANCE WITH WAKATO DISTRICT COUNCIL STANDARD DETAILS AND THE WAKATO REGIONAL INFRASTRUCTURE TECHNICAL SPECIFICATION.
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ROADS AND FRONT 1m OF BERM: 800mm
BERMS: 400mm
SERVICE CONNECTIONS: 450mm
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LEGEND

EX WATERMAIN	PR WATERMAIN
EXR HYDRANT	PRR HYDRANT
EXR METER	PRR METER
EX SLUICE VALVE	PR SLUICE VALVE
EX PEET VALVE	PR PEET VALVE
PR SLUICE VALVE	PRR SLUICE VALVE
PR PEET VALVE	PRR PEET VALVE
PR WETLAND	PRR WETLAND
PR RANGARDEN	PRR RANGARDEN

A	RESOURCE CONSENT	SP	07/2026
Ref	Revisions	By	Date
Survey	MAVEN	10/2025	
Design	AC	10/2025	
Drawn	SP	10/2025	
Checked	RW	07/2026	

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Project
PROPOSED INDUSTRIAL DEVELOPMENT AT TE KOWHAI EAST FOR TE KOWHAI EAST LIMITED PARTNERSHIP

Title
PROPOSED WATER SUPPLY PLAN

Project no.	180010
Scale	1:750 @ A1
Cad file	180010-C600-01.DWG
Drawing no.	C600-5
Rev	A

APPENDIX B



FOR INFORMATION

FOR INFORMATION			
Ref	Revisions	By	Date
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Survey	MAVEN	10/2025	
Design	RW	10/2025	
Drawn	SP	10/2025	
Checked	RW	07/2026	
Maven Associates 19 571 0050 info@maven.co.nz www.maven.co.nz 5 Deans Road, Epsom Auckland 10523			
Project PROPOSED INDUSTRIAL DEVELOPMENT AT TE KOWHAI EAST FOR TE KOWHAI EAST LIMITED PARTNERSHIP			
Title PROPOSED WATER SUPPLY CATCHMENT PLAN			
Project no.	180010		
Scale	1:4000 @ A1		
Cad file	WATER LAYOUT.DWG		
Drawing no.	C590	Rev	A

 Maven Associates		Job Number 180010	Sheet 1	Rev A
Job Title Calc Title	TKE Development Water Demand Calc	Author AYC	Date 1/07/2026	Checked JZ
As per Waikato Local Authority RITS standards Peak Flow Rate of Five Times Domestic Demand = 1300 <i>l/person/day</i> Domestic Demand = 260 <i>l/person/day</i>				
Development Area using Gravity/Pumpstation				
Industrial Catchment area = 124.30 Ha Ultimate Catchment area = 68.80 Ha Stages 1-5 Population Equivalent as per Table 91= 45 <i>persons per hectare</i> Total Population 5594 Ultimate Total Population 3096 Stages 1-5				
Demand Supply (Peak Flow Rate)= 7271.55 <i>m³/day</i> Ultimate Demand Supply (Peak Flow Rate)= 84.161 <i>L/s</i> Ultimate 2/3 Demand Supply (Peak Flow Rate)= 56.108 <i>L/s</i> Ultimate				
Demand Supply (Peak Flow Rate)= 4024.80 <i>m³/day</i> Stages 1-5 Demand Supply (Peak Flow Rate)= 46.583 <i>L/s</i> Stages 1-5 2/3 Demand Supply (Peak Flow Rate)= 31.056 <i>L/s</i> Stages 1-5				

 Maven Associates		Job Number 180010	Sheet 2	Rev A
Job Title Calc Title	TKE Development Water Demand Calc	Author AYC	Date 1/07/2026	Checked JZ
As per Waikato Local Authority RITS standards Peak Flow Rate of Five Times Domestic Demand = 1300 <i>l/person/day</i> Domestic Demand = 260 <i>l/person/day</i>				
Development Area using Gravity/Pumpstation				
Industrial Catchment area = 18.00 Ha CATCHMENT #1 Population Equivalent as per Table 91= 45 <i>persons per hectare</i> Total Population 810				
Demand Supply (Peak Flow Rate)= 1053.00 <i>m³/day</i> Demand Supply (Peak Flow Rate)= 12.188 <i>L/s</i>				
Number of EPANET JUNCTIONS 5 (#101,97,89,86,82) Base Demand 2.438 L/s 2/3 OF Base Demand 1.63 L/s				

 Maven Associates		Job Number 180010	Sheet 3	Rev A
Job Title Calc Title	TKE Development Water Demand Calc	Author AYC	Date 1/07/2026	Checked JZ
As per Waikato Local Authority RITS standards Peak Flow Rate of Five Times Domestic Demand = 1300 <i>l/person/day</i> Domestic Demand = 260 <i>l/person/day</i>				
Development Area using Gravity/Pumpstation				
Industrial Catchment area = 2.20 <i>Ha</i> CATCHMENT #2 Population Equivalent as per Table 91= 45 <i>persons per hectare</i> Total Population 99				
Demand Supply (Peak Flow Rate)= 128.70 <i>m³/day</i> Demand Supply (Peak Flow Rate)= 1.490 <i>L/s</i>				
Number of EPANET JUNCTIONS 2 (#8,185) Base Demand 0.745 <i>L/s</i> 2/3 OF Base Demand 0.50 <i>L/s</i>				

 Maven Associates		Job Number 180010	Sheet 4	Rev A
Job Title Calc Title	TKE Development Water Demand Calc	Author AYC	Date 1/07/2026	Checked JZ
As per Waikato Local Authority RITS standards Peak Flow Rate of Five Times Domestic Demand = 1300 <i>l/person/day</i> Domestic Demand = 260 <i>l/person/day</i>				
Development Area using Gravity/Pumpstation				
Industrial Catchment area = 1.30 <i>Ha</i> CATCHMENT #3 Population Equivalent as per Table 91= 45 <i>persons per hectare</i> Total Population 59				
Demand Supply (Peak Flow Rate)= 76.05 <i>m³/day</i> Demand Supply (Peak Flow Rate)= 0.880 <i>L/s</i>				
Number of EPANET JUNCTIONS 3 (#188,190,198) Base Demand 0.293 <i>L/s</i> 2/3 OF Base Demand 0.20 <i>L/s</i>				

 Maven Associates		Job Number 180010	Sheet 5	Rev A
Job Title Calc Title	TKE Development Water Demand Calc	Author AYC	Date 1/07/2026	Checked JZ
As per Waikato Local Authority RITS standards Peak Flow Rate of Five Times Domestic Demand = 1300 <i>l/person/day</i> Domestic Demand = 260 <i>l/person/day</i>				
Development Area using Gravity/Pumpstation				
Industrial Catchment area = 3.40 Ha CATCHMENT #4 Population Equivalent as per Table 91= 45 <i>persons per hectare</i> Total Population 153				
Demand Supply (Peak Flow Rate)= 198.90 <i>m³/day</i> Demand Supply (Peak Flow Rate)= 2.302 <i>L/s</i>				
Number of EPANET JUNCTIONS 2 (#193,194) Base Demand 1.151 L/s 2/3 OF Base Demand 0.77 L/s				

 Maven Associates		Job Number 180010	Sheet 6	Rev A
Job Title Calc Title	TKE Development Water Demand Calc	Author AYC	Date 1/07/2026	Checked JZ
As per Waikato Local Authority RITS standards Peak Flow Rate of Five Times Domestic Demand = 1300 <i>l/person/day</i> Domestic Demand = 260 <i>l/person/day</i>				
Development Area using Gravity/Pumpstation				
Industrial Catchment area = 2.20 Ha CATCHMENT #5 Population Equivalent as per Table 91= 45 <i>persons per hectare</i> Total Population 99				
Demand Supply (Peak Flow Rate)= 128.70 <i>m³/day</i> Demand Supply (Peak Flow Rate)= 1.490 <i>L/s</i>				
Number of EPANET JUNCTIONS 3 (#187,172,164) Base Demand 0.497 L/s 2/3 OF Base Demand 0.33 L/s				

 Maven Associates		Job Number 180010	Sheet 7	Rev A
Job Title Calc Title	TKE Development Water Demand Calc	Author AYC	Date 1/07/2026	Checked JZ
As per Waikato Local Authority RITS standards Peak Flow Rate of Five Times Domestic Demand = 1300 <i>l/person/day</i> Domestic Demand = 260 <i>l/person/day</i>				
Development Area using Gravity/Pumpstation				
Industrial Catchment area = 2.30 Ha Population Equivalent as per Table 91= 45 persons per hectare Total Population 104 CATCHMENT #6				
Demand Supply (Peak Flow Rate)= 134.55 <i>m³/day</i> Demand Supply (Peak Flow Rate)= 1.557 <i>L/s</i>				
Number of EPANET JUNCTIONS 2 (#9,184) Base Demand 0.779 <i>L/s</i> 2/3 OF Base Demand 0.52 <i>L/s</i>				

 Maven Associates		Job Number 180010	Sheet 8	Rev A
Job Title Calc Title	TKE Development Water Demand Calc	Author AYC	Date 1/07/2026	Checked JZ
As per Waikato Local Authority RITS standards Peak Flow Rate of Five Times Domestic Demand = 1300 <i>l/person/day</i> Domestic Demand = 260 <i>l/person/day</i>				
Development Area using Gravity/Pumpstation				
Industrial Catchment area = 2.40 <i>Ha</i> CATCHMENT #7 Population Equivalent as per Table 91= 45 <i>persons per hectare</i> Total Population 108				
Demand Supply (Peak Flow Rate)= 140.40 <i>m³/day</i> Demand Supply (Peak Flow Rate)= 1.625 <i>L/s</i>				
Number of EPANET JUNCTIONS 2 (#16,36) Base Demand 0.813 <i>L/s</i> 2/3 OF Base Demand 0.54 <i>L/s</i>				

 Maven Associates		Job Number 180010	Sheet 9	Rev A
Job Title Calc Title	TKE Development Water Demand Calc	Author AYC	Date 1/07/2026	Checked JZ
As per Waikato Local Authority RITS standards Peak Flow Rate of Five Times Domestic Demand = 1300 <i>l/person/day</i> Domestic Demand = 260 <i>l/person/day</i>				
Development Area using Gravity/Pumpstation				
Industrial Catchment area = 4.40 <i>Ha</i> CATCHMENT #8 Population Equivalent as per Table 91= 45 <i>persons per hectare</i> Total Population 198				
Demand Supply (Peak Flow Rate)= 257.40 <i>m³/day</i> Demand Supply (Peak Flow Rate)= 2.979 <i>L/s</i>				
Number of EPANET JUNCTIONS 3 (#6,17,176) Base Demand 0.993 <i>L/s</i> 2/3 OF Base Demand 0.66 <i>L/s</i>				

 Maven Associates		Job Number 180010	Sheet 10	Rev A
Job Title Calc Title	TKE Development Water Demand Calc	Author AYC	Date 1/07/2026	Checked JZ
As per Waikato Local Authority RITS standards Peak Flow Rate of Five Times Domestic Demand = 1300 <i>l/person/day</i> Domestic Demand = 260 <i>l/person/day</i>				
Development Area using Gravity/Pumpstation				
Industrial Catchment area = 4.00 <i>Ha</i> Population Equivalent as per Table 91= 45 <i>persons per hectare</i> Total Population 180 CATCHMENT #9				
Demand Supply (Peak Flow Rate)= 234.00 <i>m³/day</i> Demand Supply (Peak Flow Rate)= 2.708 <i>L/s</i>				
Number of EPANET JUNCTIONS 2 (#24,215) Base Demand 1.354 <i>L/s</i> 2/3 OF Base Demand 0.90 <i>L/s</i>				

 Maven Associates		Job Number 180010	Sheet 11	Rev A
Job Title Calc Title	TKE Development Water Demand Calc	Author AYC	Date 1/07/2026	Checked JZ
As per Waikato Local Authority RITS standards Peak Flow Rate of Five Times Domestic Demand = 1300 <i>l/person/day</i> Domestic Demand = 260 <i>l/person/day</i>				
Development Area using Gravity/Pumpstation				
Industrial Catchment area = 1.30 <i>Ha</i> CATCHMENT #10 Population Equivalent as per Table 91= 45 <i>persons per hectare</i> Total Population 59				
Demand Supply (Peak Flow Rate)= 76.05 <i>m³/day</i> Demand Supply (Peak Flow Rate)= 0.880 <i>L/s</i>				
Number of EPANET JUNCTIONS 1 (#163) Base Demand 0.880 <i>L/s</i> 2/3 OF Base Demand 0.59 <i>L/s</i>				

 Maven Associates		Job Number 180010	Sheet 12	Rev A
Job Title Calc Title	TKE Development Water Demand Calc	Author AYC	Date 1/07/2026	Checked JZ
As per Waikato Local Authority RITS standards Peak Flow Rate of Five Times Domestic Demand = 1300 <i>l/person/day</i> Domestic Demand = 260 <i>l/person/day</i>				
Development Area using Gravity/Pumpstation				
Industrial Catchment area = 1.30 <i>Ha</i> CATCHMENT #11 Population Equivalent as per Table 91= 45 <i>persons per hectare</i> Total Population 59				
Demand Supply (Peak Flow Rate)= 76.05 <i>m³/day</i> Demand Supply (Peak Flow Rate)= 0.880 <i>L/s</i>				
Number of EPANET JUNCTIONS 2 (#213,227) Base Demand 0.440 <i>L/s</i> 2/3 OF Base Demand 0.29 <i>L/s</i>				

 Maven Associates		Job Number 180010	Sheet 13	Rev A
Job Title Calc Title	TKE Development Water Demand Calc	Author AYC	Date 1/07/2026	Checked JZ
As per Waikato Local Authority RITS standards Peak Flow Rate of Five Times Domestic Demand = 1300 <i>l/person/day</i> Domestic Demand = 260 <i>l/person/day</i>				
Development Area using Gravity/Pumpstation				
Industrial Catchment area = 4.00 <i>Ha</i> Population Equivalent as per Table 91= 45 <i>persons per hectare</i> Total Population 180 CATCHMENT #12				
Demand Supply (Peak Flow Rate)= 234.00 <i>m³/day</i> Demand Supply (Peak Flow Rate)= 2.708 <i>L/s</i>				
Number of EPANET JUNCTIONS 2 (#25,218) Base Demand 1.354 <i>L/s</i> 2/3 OF Base Demand 0.90 <i>L/s</i>				

 Maven Associates		Job Number 180010	Sheet 14	Rev A
Job Title Calc Title	TKE Development Water Demand Calc	Author AYC	Date 1/07/2026	Checked JZ
As per Waikato Local Authority RITS standards Peak Flow Rate of Five Times Domestic Demand = 1300 <i>l/person/day</i> Domestic Demand = 260 <i>l/person/day</i>				
Development Area using Gravity/Pumpstation				
Industrial Catchment area = 3.00 <i>Ha</i> CATCHMENT #13 Population Equivalent as per Table 91= 45 <i>persons per hectare</i> Total Population 135				
Demand Supply (Peak Flow Rate)= 175.50 <i>m³/day</i> Demand Supply (Peak Flow Rate)= 2.031 <i>L/s</i>				
Number of EPANET JUNCTIONS 3 (#30,234,240) Base Demand 0.677 <i>L/s</i> 2/3 OF Base Demand 0.45 <i>L/s</i>				

	Maven Associates	Job Number 180010	Sheet 15	Rev A
Job Title Calc Title	TKE Development Water Demand Calc	Author AYC	Date 1/07/2026	Checked JZ
As per Waikato Local Authority RITS standards Peak Flow Rate of Five Times Domestic Demand = 1300 <i>l/person/day</i> Domestic Demand = 260 <i>l/person/day</i>				
Development Area using Gravity/Pumpstation Industrial Catchment area = 6.80 Ha CATCHMENT #14 Population Equivalent as per Table 91= 45 <i>persons per hectare</i> Total Population 306 Demand Supply (Peak Flow Rate)= 397.80 <i>m³/day</i> Demand Supply (Peak Flow Rate)= 4.604 <i>L/s</i> Number of EPANET JUNCTIONS 2 (#77,80) Base Demand 2.302 L/s 2/3 OF Base Demand 1.53 L/s				

 Maven Associates		Job Number 180010	Sheet 16	Rev A
Job Title Calc Title	TKE Development Water Demand Calc	Author AYC	Date 1/07/2026	Checked JZ
As per Waikato Local Authority RITS standards Peak Flow Rate of Five Times Domestic Demand = 1300 <i>l/person/day</i> Domestic Demand = 260 <i>l/person/day</i>				
Development Area using Gravity/Pumpstation				
Industrial Catchment area = 3.80 <i>Ha</i> CATCHMENT #15 Population Equivalent as per Table 91= 45 <i>persons per hectare</i> Total Population 171				
Demand Supply (Peak Flow Rate)= 222.30 <i>m³/day</i> Demand Supply (Peak Flow Rate)= 2.573 <i>L/s</i>				
Number of EPANET JUNCTIONS 4 (#37,38,39,42) Base Demand 0.643 <i>L/s</i> 2/3 OF Base Demand 0.43 <i>L/s</i>				

 Maven Associates		Job Number 180010	Sheet 17	Rev A
Job Title Calc Title	TKE Development Water Demand Calc	Author AYC	Date 1/07/2026	Checked JZ
As per Waikato Local Authority RITS standards Peak Flow Rate of Five Times Domestic Demand = 1300 <i>l/person/day</i> Domestic Demand = 260 <i>l/person/day</i>				
Development Area using Gravity/Pumpstation				
Industrial Catchment area = 5.10 <i>Ha</i> Population Equivalent as per Table 91= 45 <i>persons per hectare</i> Total Population 230 CATCHMENT #16				
Demand Supply (Peak Flow Rate)= 298.35 <i>m³/day</i> Demand Supply (Peak Flow Rate)= 3.453 <i>L/s</i>				
Number of EPANET JUNCTIONS 2 (#244,246) Base Demand 1.727 <i>L/s</i> 2/3 OF Base Demand 1.15 <i>L/s</i>				

 Maven Associates		Job Number 180010	Sheet 18	Rev A
Job Title Calc Title	TKE Development Water Demand Calc	Author AYC	Date 1/07/2026	Checked JZ
As per Waikato Local Authority RITS standards Peak Flow Rate of Five Times Domestic Demand = 1300 <i>l/person/day</i> Domestic Demand = 260 <i>l/person/day</i>				
Development Area using Gravity/Pumpstation				
Industrial Catchment area = 4.30 Ha Population Equivalent as per Table 91= 45 persons per hectare Total Population 194 CATCHMENT #17+#18				
Demand Supply (Peak Flow Rate)= 251.55 m³/day Demand Supply (Peak Flow Rate)= 2.911 L/s				
Number of EPANET JUNCTIONS 1 (#260) Base Demand 2.911 L/s 2/3 OF Base Demand 1.94 L/s				

 Maven Associates		Job Number 180010	Sheet 19	Rev A
Job Title Calc Title	TKE Development Water Demand Calc	Author AYC	Date 1/07/2026	Checked JZ
As per Waikato Local Authority RITS standards Peak Flow Rate of Five Times Domestic Demand = 1300 <i>l/person/day</i> Domestic Demand = 260 <i>l/person/day</i>				
Development Area using Gravity/Pumpstation				
Industrial Catchment area = 5.10 <i>Ha</i> Population Equivalent as per Table 91= 45 <i>persons per hectare</i> Total Population 230 CATCHMENT #19				
Demand Supply (Peak Flow Rate)= 298.35 <i>m³/day</i> Demand Supply (Peak Flow Rate)= 3.453 <i>L/s</i>				
Number of EPANET JUNCTIONS 5 (#134,138,140,142,145) Base Demand 0.691 <i>L/s</i> 2/3 OF Base Demand 0.46 <i>L/s</i>				

 Maven Associates		Job Number 180010	Sheet 20	Rev A
Job Title Calc Title	TKE Development Water Demand Calc	Author AYC	Date 1/07/2026	Checked JZ
As per Waikato Local Authority RITS standards Peak Flow Rate of Five Times Domestic Demand = 1300 <i>l/person/day</i> Domestic Demand = 260 <i>l/person/day</i>				
Development Area using Gravity/Pumpstation				
Industrial Catchment area = 1.00 <i>Ha</i> CATCHMENT #20 Population Equivalent as per Table 91= 45 <i>persons per hectare</i> Total Population 45				
Demand Supply (Peak Flow Rate)= 58.50 <i>m³/day</i> Demand Supply (Peak Flow Rate)= 0.677 <i>L/s</i>				
Number of EPANET JUNCTIONS 2 (#154,157) Base Demand 0.339 <i>L/s</i> 2/3 OF Base Demand 0.23 <i>L/s</i>				

 Maven Associates		Job Number 180010	Sheet 21	Rev A
Job Title Calc Title	TKE Development Water Demand Calc	Author AYC	Date 1/07/2026	Checked JZ
As per Waikato Local Authority RITS standards Peak Flow Rate of Five Times Domestic Demand = 1300 <i>l/person/day</i> Domestic Demand = 260 <i>l/person/day</i>				
Development Area using Gravity/Pumpstation				
Industrial Catchment area = 2.80 <i>Ha</i> CATCHMENT #21 Population Equivalent as per Table 91= 45 <i>persons per hectare</i> Total Population 126				
Demand Supply (Peak Flow Rate)= 163.80 <i>m³/day</i> Demand Supply (Peak Flow Rate)= 1.896 <i>L/s</i>				
Number of EPANET JUNCTIONS 3 (#147,150,152) Base Demand 0.632 <i>L/s</i> 2/3 OF Base Demand 0.42 <i>L/s</i>				

 Maven Associates		Job Number 180010	Sheet 22	Rev A
Job Title Calc Title	TKE Development Water Demand Calc	Author AYC	Date 1/07/2026	Checked JZ
As per Waikato Local Authority RITS standards Peak Flow Rate of Five Times Domestic Demand = 1300 <i>l/person/day</i> Domestic Demand = 260 <i>l/person/day</i>				
Development Area using Gravity/Pumpstation				
Industrial Catchment area = 8.00 <i>Ha</i> CATCHMENT #22 Population Equivalent as per Table 91= 45 <i>persons per hectare</i> Total Population 360				
Demand Supply (Peak Flow Rate)= 468.00 <i>m³/day</i> Demand Supply (Peak Flow Rate)= 5.417 <i>L/s</i>				
Number of EPANET JUNCTIONS 5 (#119,130,137,139,141) Base Demand 1.083 <i>L/s</i> 2/3 OF Base Demand 0.72 <i>L/s</i>				

 Maven Associates		Job Number 180010	Sheet 23	Rev A
Job Title Calc Title	TKE Development Water Demand Calc	Author AYC	Date 1/07/2026	Checked JZ
As per Waikato Local Authority RITS standards Peak Flow Rate of Five Times Domestic Demand = 1300 <i>l/person/day</i> Domestic Demand = 260 <i>l/person/day</i>				
Development Area using Gravity/Pumpstation				
Industrial Catchment area = 8.10 <i>Ha</i> Population Equivalent as per Table 91= 45 <i>persons per hectare</i> Total Population 365 CATCHMENT #23				
Demand Supply (Peak Flow Rate)= 473.85 <i>m³/day</i> Demand Supply (Peak Flow Rate)= 5.484 <i>L/s</i>				
Number of EPANET JUNCTIONS 5 (#44,132,241,245,259) Base Demand 1.097 <i>L/s</i> 2/3 OF Base Demand 0.73 <i>L/s</i>				

	Maven Associates	Job Number 180010	Sheet 24	Rev A
Job Title Calc Title	TKE Development Water Demand Calc	Author AYC	Date 1/07/2026	Checked JZ
As per Waikato Local Authority RITS standards Peak Flow Rate of Five Times Domestic Demand = 1300 <i>l/person/day</i> Domestic Demand = 260 <i>l/person/day</i>				
Development Area using Gravity/Pumpstation Industrial Catchment area = 7.10 Ha CATCHMENT #24 Population Equivalent as per Table 91= 45 <i>persons per hectare</i> Total Population 320 Demand Supply (Peak Flow Rate)= 415.35 <i>m³/day</i> Demand Supply (Peak Flow Rate)= 4.807 <i>L/s</i> Number of EPANET JUNCTIONS 4 (#49,124,265,276) Base Demand 1.202 L/s 2/3 OF Base Demand 0.80 L/s				

 Maven Associates		Job Number 180010	Sheet 25	Rev A
Job Title Calc Title	TKE Development Water Demand Calc	Author AYC	Date 1/07/2026	Checked JZ
As per Waikato Local Authority RITS standards Peak Flow Rate of Five Times Domestic Demand = 1300 <i>l/person/day</i> Domestic Demand = 260 <i>l/person/day</i>				
Development Area using Gravity/Pumpstation				
Industrial Catchment area = 5.80 <i>Ha</i> Population Equivalent as per Table 91= 45 <i>persons per hectare</i> Total Population 261 CATCHMENT #25				
Demand Supply (Peak Flow Rate)= 339.30 <i>m³/day</i> Demand Supply (Peak Flow Rate)= 3.927 <i>L/s</i>				
Number of EPANET JUNCTIONS 4 (#50,121,264,277) Base Demand 0.982 <i>L/s</i> 2/3 OF Base Demand 0.65 <i>L/s</i>				

 Maven Associates		Job Number 180010	Sheet 26	Rev A
Job Title Calc Title	TKE Development Water Demand Calc	Author AYC	Date 1/07/2026	Checked JZ
As per Waikato Local Authority RITS standards Peak Flow Rate of Five Times Domestic Demand = 1300 <i>l/person/day</i> Domestic Demand = 260 <i>l/person/day</i>				
Development Area using Gravity/Pumpstation				
Industrial Catchment area = 4.60 <i>Ha</i> CATCHMENT #26 Population Equivalent as per Table 91= 45 <i>persons per hectare</i> Total Population 207				
Demand Supply (Peak Flow Rate)= 269.10 <i>m³/day</i> Demand Supply (Peak Flow Rate)= 3.115 <i>L/s</i>				
Number of EPANET JUNCTIONS 4 (#54,57,113,115) Base Demand 0.779 <i>L/s</i> 2/3 OF Base Demand 0.52 <i>L/s</i>				

	Maven Associates	Job Number 180010	Sheet 27	Rev A
Job Title Calc Title	TKE Development Water Demand Calc	Author AYC	Date 1/07/2026	Checked JZ
As per Waikato Local Authority RITS standards Peak Flow Rate of Five Times Domestic Demand = 1300 <i>l/person/day</i> Domestic Demand = 260 <i>l/person/day</i>				
Development Area using Gravity/Pumpstation Industrial Catchment area = 9.00 Ha CATCHMENT #27 Population Equivalent as per Table 91= 45 <i>persons per hectare</i> Total Population 405 Demand Supply (Peak Flow Rate)= 526.50 <i>m³/day</i> Demand Supply (Peak Flow Rate)= 6.094 <i>L/s</i> Number of EPANET JUNCTIONS 4 (#58,60,112,114) Base Demand 1.523 L/s 2/3 OF Base Demand 1.02 L/s				

 Maven Associates		Job Number 180010	Sheet 28	Rev A
Job Title Calc Title	TKE Development Water Demand Calc	Author AYC	Date 1/07/2026	Checked JZ
As per Waikato Local Authority RITS standards Peak Flow Rate of Five Times Domestic Demand = 1300 <i>l/person/day</i> Domestic Demand = 260 <i>l/person/day</i>				
Development Area using Gravity/Pumpstation				
Industrial Catchment area = 3.00 <i>Ha</i> CATCHMENT #28 Population Equivalent as per Table 91= 45 <i>persons per hectare</i> Total Population 135				
Demand Supply (Peak Flow Rate)= 175.50 <i>m³/day</i> Demand Supply (Peak Flow Rate)= 2.031 <i>L/s</i>				
Number of EPANET JUNCTIONS 2 (#69,74) Base Demand 1.016 <i>L/s</i> 2/3 OF Base Demand 0.68 <i>L/s</i>				