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ANALYSIS
DESIGN
ASSESSMENT
CERTIFICATION

STORMWATER MANAGEMENT

STORMWATER MANAGEMENT PLAN

Revision 6 — Council Information Request
Stormwater quantity and quality

134 South Street
Thornlands QLD 4164



Prepared for
Development application
MCU26/0056



Lot 1 on RP124525



Multiple dwelling, 15 units
(12 two-bedroom and 3 three-bedroom)

STORMWATER MANAGEMENT
HYDROLOGY & HYDRAULICS
CIVIL INFRASTRUCTURE
DEVELOPMENT ENGINEERING

South Street

Unnamed

134

132

130

128

Site context only — refer to report for assessment

RPEQ
ENGINEERING



Council ref:
MCU26/0056



Council IR:
Items 2, 3 and 4



Prepared by:
Craig Harris,
RPEQ 13194



15 October 2026
Revision 6
Public issue



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DOCUMENT CONTROL

CURRENT ISSUE	STATUS
STORMWATER MANAGEMENT PLAN	Revision 6 — Public issue

PROJECT PARTICULARS

Site / project	134 South Street, Thornlands QLD 4164
Lot on plan	Lot 1 on RP124525
Land area	1,011 m ² (site area adopted throughout this plan; Revision 2 stated 1,012 m ²)
Development / scope	Multiple dwelling, 15 units (12 two-bedroom and 3 three-bedroom), 29 car parking spaces
Client / Council reference	MCU26/0056 (pre-lodgement meeting PLM25/0045)
Council information request	Redland City Council Information Request MCU26/0056 dated 9 June 2026, Items 2, 3 and 4
Prepared for	Development application MCU26/0056; response to Council Items 2 and 3 (stormwater), with supplementary Item 4 water assessment

ISSUE & AUTHORISATION

Responsibility / control	Current status
Prepared, reviewed and approved for issue by	Craig Harris, RPEQ Civil Engineer, RPEQ 13194
Revision date / status	15 October 2026 — Revision 6, Public issue
Issue restriction	Public issue of the final report for the Council RFI response (Revision 5), including the supplementary water-supply assessment. Not for construction.

REVISION HISTORY

Table 1: Revision history

Rev.	Date	Status	Description
1	January 2026	Not issued	Preparation version, superseded by Revision 2
2	9 January 2026	Issued for DA lodgement	Original Stormwater Management Plan lodged as Appendix E of the application; no on-site detention proposed
3	22 September 2026	Issued for RFI Response	Revised plan responding to Council Information Request Items 2 and 3: ARR ensemble hydrology, on-site detention held at or below pre-development for every event from the 50% to the 1% AEP (44.5 m ³ rear-strip tank), coordinated levels, collection network, bioretention interface, surface escape and blockage route, calculation package and drawings SW-Q04 to SW-Q08. Prepared and certified; superseded by Revision 4 before lodgement.
4	23 September 2026	Issued for RFI Response	Detention criterion re-based on the City Plan design events (10% AEP minor, 1% AEP major, and the 5% and 2% events between them; PSP2 1.2.1.4(1)); tank minimised to the smallest length at the strip width that meets it: 5.10 × 2.225 m clear, 22.7 m ³ , control plate 110 mm and 105 mm. Sections 1.4, 4.1, 4.4, 5.4, 8.1 and Appendix D revised; SW-Q04 to SW-Q08 reissued. Nothing else in the design changes.
5	24 September 2026	Issued for RFI Response	Existing stormwater analysis retained; supplied water-network assessment and WS-01 incorporated into this SWMP. Final report issued for the Council RFI response.
6	15 October 2026	Public issue	Public issue of Revision 5: internal working appendix, file references and notes removed; Appendix F restated in plain engineering terms. No change to engineering content.

Revision 6 is the public issue of Revision 5, the final report issued for the Council RFI response on 24 September 2026; its engineering content is unchanged. It supersedes Revision 4 and is not for construction.

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1. INTRODUCTION

Revision 5, reissued unchanged in engineering content as Revision 6, retains the preceding stormwater analysis and adds the assessment made possible by the subsequently supplied water-network data. Section 9 and Appendices F–G present the retained assumed-service method, new network results and civil water concept together in this report. The water-supply model does not establish stormwater pit tailwater, road flooding or survey levels; those existing stormwater qualifications remain. [R1; M1–M7]

1.1 Purpose

This Stormwater Management Plan responds to Items 2 and 3 of the Redland City Council Information Request dated 9 June 2026 for development application MCU26/0056, a 15-unit multiple dwelling at 134 South Street, Thornlands (Lot 1 on RP124525). Item 2 asks for:

1. the location of inlet pits and pipes collecting ground water and roof water, directed to a lawful point of discharge, with the catchment associated with each structure;
2. the size, levels and capacity of the drainage system;
3. surface levels in the parking and landscaping areas showing how stormwater is collected in each inlet pit, and levels in the adjacent properties;
4. how flows above the capacity of the internal system are managed for events up to the 1% AEP;
5. the mechanism that maintains pre-development flow before discharge to the local network;
6. how blocked flows are directed to the front road through the driveway crossover;
7. the location and size of the proposed bioretention system and how it connects to the drainage system;
8. a calculation table, a long section of the stormwater system and cross sections of the treatment device.

Item 3 asks for:

1. a revised site-based stormwater management plan;
2. details of the proposed treatment devices;
3. an electronic copy of the treatment model for Council validation.

Compliance is demonstrated against performance outcomes PO3, PO4 and PO6 (quantity) and PO9 and PO10 (quality) of the Healthy waters code, Redland City Plan 2018 Version 14.

Revision 2, lodged with the application on 9 January 2026, assessed the site with the Rational Method at a five-minute time of concentration, identified South Street as the lawful point of discharge, proposed a bioretention raingarden, and did not propose on-site detention. This revision replaces that assessment. It adopts the submitted architectural layout, the site cadastral area, Australian Rainfall and Runoff ensemble hydrology, and Council mapping of the receiving network. The material changes and their reasons are set out in section 1.4.

1.2 Council request and where it is answered

Table 2: Council Information Request Item 2 and Item 3, response schedule

Council request	Answered in	Evidence
Item 2: location of inlet pits and pipes collecting ground water and roof water, directed to a lawful point of discharge, including the catchment associated with each structure, and the size, levels and capacity of the system	Sections 3.3, 4.5; SW-01, SW-Q04, SW-Q05; Table 9 and Appendix D Table 23	Catchment schedule, pipe schedule with inverts, capacities and hydraulic grade lines

Council request	Answered in	Evidence
Item 2: surface levels in the parking area and landscaping areas demonstrating how stormwater is collected in each inlet pit, and levels in the adjacent properties	Sections 4.6, 5.1; SW-Q04 level schedule; SW-Q07 sections	Level schedule from the RL 31.00 entrance transition; mapped adjoining ground
Item 2: if earthworks are required, how flows above the internal capacity are managed up to the 1% AEP event, the mechanism maintaining pre-development flow before discharge, and how blocked flows are directed to the front road through the driveway crossover	Sections 4.3, 4.4, 5.2, 5.3; SW-Q04, SW-Q05, SW-Q06, SW-Q07	Routed detention with a two-stage control; surface corridors, cross-drain and driveway with depths and velocities; seven blockage scenarios
Item 2: location and size of the proposed bioretention system and how it connects to the above system	Section 6.5; SW-Q08, SW-Q04	Rear bioretention, 16.1 m ² media, inlet, overflow transfer to the tank and filtered transfer to the control chamber
Item 2: calculation table, long section of the stormwater system, and cross sections of the treatment device proposed	Appendix D; SW-Q05; SW-Q08	Calculation package, long section, treatment plan and sections
Item 3: revised site-based stormwater management plan, treatment details and electronic model	Sections 6, 7; SW-Q08; Appendix A	Default treatment area route under Planning Scheme Policy 2 section 1.3.1.1(4); Council acceptance of that route in place of the model is requested in section 6.3

1.3 Design references

1. Redland City Plan 2018 Version 14, Healthy waters code section 9.3.1.3: Table 9.3.1.3.1 performance outcomes and Table 9.3.1.3.2 minor drainage design storms (the assessment instrument).
2. Planning Scheme Policy 2, Infrastructure works: section 1.2 governs drainage design and section 1.3 the stormwater quality management plan.
3. Queensland Urban Drainage Manual, 2016 fourth edition (called up by PSP2 as the current edition).
4. Australian Rainfall and Runoff 2019, Book 9 version 4.2 and Book 2 chapter 5 (called up by PSP2 as the current edition).
5. Water by Design Bioretention Technical Design Guidelines, version 1.1, 2014, and standard drawings DS-070, DS-072 and DS-078 (deemed-to-satisfy bioretention details).
6. Bureau of Meteorology 2016 IFD design rainfall at the site grid; ARR Data Hub temporal patterns, losses and pre-burst depths for the site.
7. Council pre-lodgement minutes PLM25/0045, 8 December 2025 (South Street identified as the lawful discharge destination).
8. Council stormwater asset mapping and 0.25 m contours, checked 21 September 2026 (receiving pit 164208 and terrain).
9. Interworks Architects, submitted Issue A set dated 30 October 2025: drawings 2.1, 2.2, 2.6 and 5.1 (the governing layout).

1.4 What changed from Revision 2 and Revision 3, and why

Matter	Revision 2 (lodged January 2026)	This revision	Reason
Hydrology	Rational Method, $C = 0.9$, 5 minute time of concentration, $1,012 \text{ m}^2$	ARR 2019 ensemble method (ten Data Hub patterns per duration) for pre and post development, Rational Method retained as the cross-check; $1,011 \text{ m}^2$ site area	Detention sizing needs routed hydrographs; the Rational Method cannot size storage (QUDM 5.6.2)
Detention	Not proposed; downstream capacity asserted	Tank in the rear landscape strip west of the bioretention, 11.35 m^2 clear plan, 2.0 m working depth, 22.7 m^3 to the normal maximum, two-stage orifice control	PSP2 1.2.1.4(1) requires on-site mitigation unless downstream capacity is demonstrated; the Council asset records do not demonstrate it
Receiving asset	Existing pit at the corner of South Street and the unnamed road to the west	Council pit 164208 east of the site (mapped base RL 26.30, cover RL 28.01)	Live Council asset mapping locates the receiving gully pit east of the site
Levels	Not designed	One schedule from the owner-stated carpark entrance transition RL 31.00	Council asked for surface and inlet levels and adjoining levels
Treatment	Raingarden in the landscaped area	Rear sealed bioretention, 16.1 m^2 finished media, media RL 30.45, beside the detention tank	Default treatment area route (1.5% of the contributing catchment) with the connected hydraulics designed
Blockage route	Not addressed	Graded surface corridors, lined cross-drain and driveway to South Street, outside the building line	Council Item 2 asks for the blockage route through the crossover

Revision 3 of this plan (22 September 2026) was prepared and certified for this response but not lodged. Revision 4 changed one thing in it, the detention basis, and everything that follows from it:

- Design criterion. Revision 3 held the controlled peak at or below the pre-development peak for every event between the 50% and the 1% AEP. This revision applies that test to the design events the planning scheme sets, the 10% AEP minor storm (Healthy waters code Table 9.3.1.3.2) and the 1% AEP major storm (AO4.1; PSP2 1.2.1.4(1)), and to the 5% and 2% AEP events between them. The more frequent 20%, 50% and 63.2% AEP events lie below the minor design storm; their results are reported in section 4.4 but are not design criteria.
- Tank. $5.10 \times 2.225 \text{ m}$ clear (11.35 m^2 , 22.7 m^3 to the normal maximum); Revision 3 provided $10.00 \times 2.225 \text{ m}$ (22.25 m^2 , 44.5 m^3). The floor, normal maximum, bypass crest, roof and cover levels are unchanged. The tank keeps its east end beside the overflow and filtered-water pits, so the bioretention, the pits, the control chamber and the outlet route are unchanged and the 4.90 m of strip west of the tank returns to landscape.
- Control plate. 110 mm low orifice at the floor and 105 mm high orifice with its lower edge at RL 29.50; Revision 3 provided 80 mm and 100 mm at RL 29.00.
- Result. The controlled 1% AEP peak is 50.3 L/s against the pre-development 51.5 L/s (Revision 3 released 36.2 L/s, leaving 15.4 L/s of the allowance unused); the 1% AEP design storage is 21.0 m^3 (Revision 3: 36.5 m^3). The tank is now the smallest length at the strip width that meets the criteria (section 4.4).
- Why. The Revision 3 basis was stricter than the planning scheme requires and the Revision 3 tank used the whole length of the strip rather than the length the criteria need. A tank of that size is a material cost to the development with no corresponding requirement; this revision sizes the tank to the requirement and states the requirement.

2. SITE AND PROPOSED DEVELOPMENT

2.1 Site

The site is Lot 1 on RP124525 at 134 South Street, Thornlands, area 1,011 m² (the digital cadastral database area of 1,010.79 m² is rounded to 1,011 m² throughout this plan; the RCC land record lists 1,012 m²). South Street adjoins the northern frontage. The western neighbour is 4 Clayton Court (Lot 0 on SP216860); 132 South Street (Lot 5 on SP117647) adjoins the east and south. The existing use is a dwelling with 196 m² of roof and 43 m² of paving.

2.2 Architectural basis

The governing architectural reference is the submitted Issue A set: ground floor drawing 2.1, upper floor 2.2, roof 2.6 and sections 5.1. The adopted roof outline is 19.125 × 32.695 m (625.3 m²). Ground level is a covered carpark with the lift and stair core, the bin store and the vehicle entry from South Street; 15 dwellings occupy the upper levels.

Table 3: Architectural dimensions and area interpretation

Architectural measure	Area or dimension	Share of the 1,011 m ² site	Interpretation
Site area on plan	1,011 m ²	100%	Rounded architectural figure adopted throughout this plan (section 2.1)
Corrected roof outline	19.125 × 32.695 m	61.8%	625.3 m ² calculated plan area
Ground site cover	640 m ²	63.3%	Site-cover measure
Upper level site cover	621 m ²	61.4%	Site-cover measure
Landscape	264 m ²	26.1%	Architectural schedule only; exposed pervious ground is 207.2 m ² in section 3
Rear common open space	120 m ²	11.9%	Within the landscape schedule; not added again

Architectural site cover, the scheduled landscape and the runoff catchment are separate measures. Section 3 allocates each rainfall-exposed surface once, excluding covered ground below roof or upper structure; the 264 m² architectural landscape schedule is an architectural quantity, not the pervious runoff area.

2.3 Terrain and level evidence

Council 0.25 m contours and QSpatial 1 m contours show a west-to-east fall of about 0.9 m across the frontage (mapped ground about RL 31.22 at the north-west corner and RL 30.34 at the north-east corner) with the frontage and the rear at similar levels. Council contours place about RL 31 at the front boundary near the centre of the frontage, beside the driveway, and the mapped road axis at about RL 30.5. These are mapping values, not a site survey. At this stage that level of accuracy is sufficient: the plan is a concept design for the development application, its levels are set relative to one control point on the site, and the receiving pit, kerb and road levels are read from Council asset data. Kerb, driveway, wall and adjoining spot levels are confirmed by survey before detailed design at operational works, and the level schedule is adjusted from that survey without changing the design logic. The design levels in this plan are set from the owner-stated carpark entrance transition of RL 31.00 and fall toward South Street.

2.4 Existing drainage

The existing dwelling drains by sheet flow across the lot toward the east and the frontage. No piped connection to the Council network is recorded. Zero external inflow is adopted for the assessment: neighbouring runoff is assumed to be contained on neighbouring land, and the proposed boundary walls make that assumption physical (section 5.4). This is an adopted design basis, not a field verification.

3. CATCHMENT AND DISCHARGE BASIS

3.1 Existing and proposed surfaces

The pre-development baseline is 196 m² of roof, 43 m² of paving and 771.8 m² of pervious ground (impervious fraction 0.236). The proposed development has 625.3 m² of roof, 130.1 m² of exposed upper surface, 48.2 m² of exposed ground hardstand (including a 0.8 m² cadastral allowance) and 207.2 m² of exposed ground landscape (impervious fraction 0.795). Both conditions total 1,011 m².

Table 4: Existing and proposed surface areas

Surface	Pre-development (m ²)	Share of site	Post-development (m ²)	Share of site
Roof	196.0	19.4%	625.3	61.8%
Exposed upper surface	–	–	130.1	12.9%
Ground paving / hardstand	43.0	4.3%	48.2	4.8%
Pervious ground	771.8	76.3%	207.2	20.5%
Total impervious	239.0	23.6%	803.6	79.5%
Total site	1,011	100%	1,011	100%

Components are measured on the cadastral area and rounded to one decimal; percentages are of the 1,011 m² site.

3.2 External and upstream catchments

Zero external inflow applies to both conditions. The western neighbour is higher (mapped RL 31.2 to 31.4 at the boundary); a boundary upstand to RL 31.40 along the west boundary and retaining walls on the east and rear boundaries keep neighbouring runoff off the site and site runoff on the site. The western neighbour is the townhouse development at 4 Clayton Court (lots 1 to 16 on SP216860 with a common lot 0; Council parent land area 3,478 m²). It is a relatively recent development: Council received the plumbing application PD222172 on 5 March 2008, the building application BD145815 was decided on 25 September 2008, and unit construction is reported as 2009. The aerial below shows its sixteen attached dwellings on the Clayton Court frontage with several stormwater pits within the lot, and Council mapping records a private 300 mm roofwater line (asset 164239) inside the complex. That development collects its design storms in its own drainage system within its own lot; no part of it discharges onto the site, which is the basis of the zero-external-inflow assumption on the west boundary. Its downstream connection is not examined here and is not needed for this assessment.



Figure 1: Western neighbour: 4 Clayton Court townhouse development with Council 0.25 m contours and cadastre (owner-supplied Council aerial)

3.3 Proposed catchments and collection points

Drawing SW-01 (Appendix C) allocates eight disjoint catchments to roofwater collectors RW1 and RW2, upper-surface outlets PD1 and PD2, and ground inlets GP1 (front), GP2 (west), GP3 (east)

and GP4 (rear). All eight drain by pipe to the treatment inlet pit; the overflow path then reaches the detention tank and the controlled outlet to South Street, and the filtered path reaches the control chamber downstream of the plate.

Table 5: Proposed catchment and collection point schedule

Catchment	Collection point	Surface	Impervious (m ²)	Pervious (m ²)	Total (m ²)	Share of site
R-W	RW1	Roof west	312.6	0.0	312.6	30.9%
R-E	RW2	Roof east	312.6	0.0	312.6	30.9%
P-W	PD1	Upper surface west	80.1	0.0	80.1	7.9%
P-E	PD2	Upper surface east	50.0	0.0	50.0	4.9%
G-N	GP1	Front ground	48.1	42.8	90.8	9.0%
G-W	GP2	West corridor	0.0	23.0	23.0	2.3%
G-E	GP3	East corridor	0.1	46.0	46.1	4.6%
G-S	GP4	Rear landscape	0.0	95.5	95.5	9.4%
Total			803.6	207.2	1,011	100%

3.4 Lawful point of discharge and receiving infrastructure

Council pre-lodgement advice identifies South Street as the lawful discharge destination. Council asset mapping identifies gully pit 164208 in South Street, about 42.5 m east of the site's north-east corner (mapped base RL 26.30, cover RL 28.01, kerb inlet 2.4 m), as the receiving asset, draining through 450 mm and 600 mm pipes to pit 164220; the pit and its outlet pipe are shown on the aerial in section 6.1. The proposed private connection enters pit 164208 at invert RL 26.80 through a 300 mm pipe 94.7 m long at 1.32%. Connection detail and the design water level in the pit are proposed for verification at operational works; the design uses the mapped pipe invert as the tailwater with sensitivities to RL 28.70 (section 4.4).



Figure 2: Site (Lot 1 on RP124525) with the direction to the lawful point of discharge, Council gully pit 164208 in South Street (owner-supplied aerial, annotated)

Table 6: Mapped receiving drainage assets

Asset	Recorded value	Use in this assessment
Pit 164208	Base 26.30 m; cover 28.01 m	Concept receiving asset; connection invert to verify
Drain 164197	450 mm; recorded length 53.7 m	Downstream link; material and capacity to verify
Drain invert levels	Upstream 26.30 m; downstream 22.92 m	Recorded asset values, subject to field confirmation
Pit 164209	Base 22.90 m; cover 23.91 m	Downstream asset

4. STORMWATER QUANTITY MANAGEMENT

4.1 Design events and criteria

The minor drainage system is designed for the 10% AEP event (Healthy waters code Table 9.3.1.3.2, medium density residential) and the major system for the 1% AEP event (AO4.1; PSP2 1.2.1.3). Increased runoff is mitigated on site (PSP2 1.2.1.4(1)): the controlled post-development peak discharge at the site boundary does not exceed the pre-development peak at the minor design event, at the major design event and at the events between them, that is at 10%, 5%, 2% and 1% AEP (QUDM section 5.5.2, standard (iii), applied over the range of frequencies the planning scheme designs for). The more frequent 20%, 50% and 63.2% AEP events lie below the minor design storm and are not design criteria; their controlled peaks are reported in section 4.4 so that the full behaviour of the control is visible. Revision 3 applied the same test down to the 50% AEP event, which is more than the scheme requires and roughly doubled the tank (section 1.4). Downstream duration-matched restrictions under QUDM sections 5.5.3 and 5.5.5 are not adopted because the receiving system is a piped Council network with no identified undersizing and no downstream creek-flooding concern was identified; the controlled discharge is small relative to the traced pipe capacities.

4.2 Hydrological method

Pre-development and post-development runoff are modelled the same way so the comparison is like for like. Design bursts use the Bureau 2016 IFD intensities at the site (Appendix B) and the ARR Data Hub East Coast North point temporal patterns, ten patterns for each of the 24 standard durations from 10 minutes to 168 hours, in the frequent, intermediate and rare bands. Losses follow ARR Book 9 section 6.4.2: impervious surfaces initial loss 1.0 mm and no continuing loss; pervious surfaces the Data Hub storm initial loss of 26 mm reduced by the median pre-burst depth for the duration and AEP, and continuing loss 1.7 mm/h. Runoff is translated by a uniform time-area with a 15 minute inlet time before development and 5 minutes after (QUDM Table 4.6.2). For each duration the design value is the mean of the ten patterns; for each AEP it is the maximum of those means over the durations (ARR Book 2 chapter 5). Every-pattern maxima are reported as a sensitivity. Rainfall is not climate-adjusted for the design case because neither the planning scheme nor PSP2 prescribes a horizon; a +2.4 °C sensitivity is reported in section 4.4.

4.3 Pre-development and post-development discharge

Table 7: Design peak discharge at the site boundary by AEP (ensemble mean at the critical duration)

AEP	Pre-development (L/s)	Critical duration (min)	Post-development uncontrolled (L/s)	Critical duration (min)	Increase without detention (L/s)	QUDM Rational ceiling (L/s)
63.2%	11.7	90	23.3	10	11.6	13.0
50%	14.0	90	26.3	10	12.2	14.7
20%	23.8	30	37.4	15	13.7	21.2
10%	30.6	30	47.7	10	17.1	29.4
5%	37.5	20	57.5	10	20.0	35.1
2%	46.3	25	66.3	10	20.0	44.8
1%	51.5	25	73.8	10	22.2	51.8

Without detention the 1% AEP peak rises from 51.5 to 73.8 L/s and the 10% AEP peak from 30.6 to 47.7 L/s, so detention is required. The pre-development ensemble peaks agree with the QUDM Rational Method ceilings calculated independently ($C_{10} = 0.719$ at the 15 minute inlet time): 51.5 against 51.8 L/s at 1% AEP. Runoff volume also increases (the 1% AEP one-hour burst produces about 101 m³ after development against 91 m³ before); section 5.4 addresses PO3.

4.4 Detention: necessity, sizing and control

Storage was sized by routing the same ensembles through a level-pool tank with a two-stage orifice control rated by QUDM equation 5.3 (C_d 0.60, part-full openings integrated), the outlet pipe loss in series, the mapped receiving invert RL 26.30 as tailwater, and the bioretention's filtered flow (2.0 L/s, its full filtration capacity) added to the controlled discharge because the underdrain joins the outlet downstream of the plate. The tank sits in the rear landscape strip west of the bioretention, where no slab limits its height: the floor stays at RL 28.10, above the receiving pit cover so the tank drains empty, and the normal maximum is RL 30.10, giving 2.0 m of working depth. The strip fixes the internal width at 2.225 m; the tank length was then searched from 4.5 to 6.5 m in 0.1 m steps, and at each length the control plate was swept over low orifices of 40 to 200 mm in 5 mm steps, high orifices of 40 to 200 mm in 5 mm steps and high-orifice lower edges 0.3 to 1.4 m above the floor. A length passes when some plate holds the controlled peak at or below the pre-development peak at 10%, 5%, 2% and 1% AEP with the design water level below the normal maximum and every individual pattern below the bypass crest for bursts to six hours. Lengths of 4.5 to 5.0 m fail; 5.1 m is the smallest length that passes, and the plate with the largest minimum margin at that length is adopted (Appendix D, Table 21).

The adopted tank is therefore 5.10 × 2.225 m clear (11.35 m²), normal storage 22.7 m³ and 24.4 m³ to the bypass weir crest RL 30.25, at the east end of the strip beside the overflow and filtered-water pits. The margin over the design events is at least 0.6 L/s (at 5% AEP). The control plate carries a 110 mm orifice with its invert at the floor and a 105 mm orifice with its invert at RL 29.50. The plate is a bolted, replaceable element: its openings are verified against the as-built tank and the measured receiving level at commissioning, so the discharge control is tuned in the field while the tank, which cannot be changed later, is built to the minimum the criteria need.

Table 8: Routed detention performance by AEP (ensemble mean; every-pattern maximum in brackets)

AEP	Pre-development peak (L/s)	Controlled peak (L/s)	Margin (L/s)	Critical duration (min)	Peak storage (m ³)	Peak level RL (m)	Result
63.2%	11.7	17.3 (19.3)	-5.6	15	4.8 (6.0)	28.52 (28.63)	Reported; not a criterion
50%	14.0	19.1 (21.8)	-5.0	30	5.8 (7.6)	28.62 (28.77)	Reported; not a criterion
20%	23.8	25.5 (28.5)	-1.8	20	10.5 (13.2)	29.03 (29.26)	Reported; not a criterion
10%	30.6	29.7 (33.2)	0.9	20	14.2 (16.4)	29.35 (29.55)	Pass
5%	37.5	36.9 (44.7)	0.6	20	17.0 (18.9)	29.60 (29.77)	Pass
2%	46.3	45.4 (55.9)	0.8	25	19.3 (23.5)	29.80 (30.17)	Pass
1%	51.5	50.3 (58.9)	1.2	15	21.0 (24.5)	29.95 (30.26)	Pass

In summary:

- the controlled peak is below the pre-development peak at every design event from 10% to 1% AEP, with the smallest margin, 0.6 L/s, at 5% AEP;
- at the more frequent events below the minor design storm the controlled peaks exceed the pre-development peaks: by 1.8 L/s at 20% AEP, 5.0 L/s at 50% AEP and 5.6 L/s at 63.2% AEP. These events are not design criteria (section 4.1); the excess is released by pipe to the Council network at a fraction of the traced pipe capacities and reaches no adjoining land (section 5.4);
- the 1% AEP design storage is 21.0 m³ at the 15 minute critical duration, peak level RL 29.95, 146 mm below the normal maximum;
- the 10% AEP event sets the low orifice and the rarer events the second stage; the margins are deliberately small because the tank is sized to the criteria, not beyond them;

- across the ten patterns of every duration, one 12-hour pattern at 1% AEP fills the tank to the bypass weir (worst single pattern 24.5 m³, RL 30.26); the weir in the control chamber passes that pattern to the same outlet pipe, which is what it is for. This is the embedded-burst behaviour of long-duration point patterns described in ARR Book 2 chapter 5; the ensemble-mean design values remain below the criteria and no surface discharge occurs.

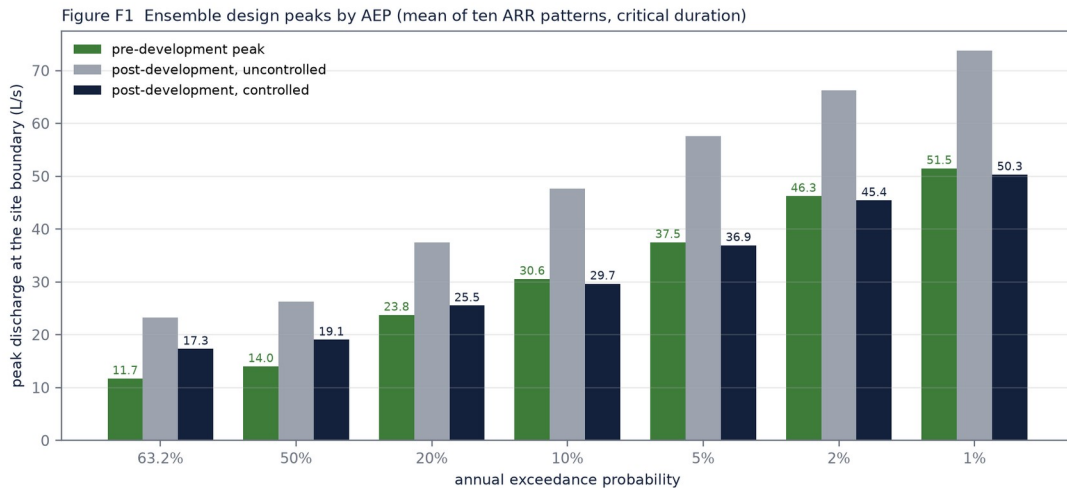


Figure 3: Ensemble design peaks by AEP: pre-development, post-development uncontrolled and controlled

Sensitivities on the adopted geometry (Appendix D Table 20), each reported with its consequence rather than stacked into the design:

- tailwater at the pit cover RL 28.01: the criterion is met at every AEP.
- tailwater RL 28.40 (the declared upper screening boundary): exceeded at 10%, 5% AEP, worst 39.1 against 37.5 L/s at 5%.
- tailwater RL 28.70: exceeded at 10%, 5% AEP, worst 34.2 against 30.6 L/s at 10%.
- discharge coefficient 0.55: the criterion is met at every AEP.
- discharge coefficient 0.65: exceeded at 10%, 1% AEP, worst 51.8 against 51.5 L/s at 1%.
- low orifice 50% blocked: the criterion is met at every AEP.
- QUDM Table 5.6.1 near-dry losses in place of the ARR losses: exceeded at 10%, 5%, 2%, 1% AEP, worst 43.2 against 37.9 L/s at 2%.
- +2.4 °C rainfall on both conditions: exceeded at 10% AEP, worst 48.1 against 47.0 L/s at 10%.
- +2.4 °C rainfall on the developed condition only, against today's pre-development benchmark: exceeded at 10%, 5%, 2%, 1% AEP, worst 48.1 against 30.6 L/s at 10%.

Reading the sensitivities. The tank is sized to the criteria, so the sensitivities that add outflow or drown the low orifice exceed the pre-development peak by small amounts where Revision 3's larger tank absorbed them. Receiving levels at RL 28.40 and 28.70 lie above the pit cover (RL 28.01), that is with South Street itself surcharged; at those levels the low orifice is drowned, the tank fills to the second stage earlier and the controlled peak exceeds the pre-development peak by up to 3.6 L/s (RL 28.7, 10% AEP), while the tank still holds the 1% and 2% AEP events. The pit level in design storms is one of the operational-works verifications (section 8.1); if Council's data places it above the cover the plate is retuned to that level. A discharge coefficient of 0.65 in place of the QUDM 0.60 raises the controlled peaks by up to 0.3 L/s (1% AEP); the plate openings are set at commissioning against the measured tank and outlet, which is the correct response to coefficient uncertainty. The QUDM near-dry loss case reduces the pre-development benchmark more than the developed flows, so its exceedance at 10%, 5%, 2% and 1% AEP reflects the benchmark, not the control; the developed-only climate case is a stress reading, not a like-for-like comparison, and is reported for information. None of the sensitivities is stacked into the design.

Scale check: the QUDM section 5.6.1 initial-loss estimate for this change of surfaces is 15.8 m³; the routed requirement of 21.0 m³ (208 m³/ha) sits within the QUDM 30 to 400 m³/ha range and is of the same order as the 14 to 24 m³ tanks in the four comparable Council precedents reviewed (799 to 1,012 m² multiple dwellings in Cleveland), allowing for this site's larger change in imperviousness. The tank provides 22.7 m³ to the normal maximum; the 1.7 m³ above the 1% AEP design level is the freeboard to the normal maximum and the bypass crest, not additional design storage.

4.5 Collection network, pits and pipes

Pipes are sized by the QUDM Rational Method at a five-minute inlet time for the 10% AEP minor event and checked for the 1% AEP event, with full-bore Manning capacities (n 0.013). The east main collects GP1, RW2, PD2 and GP3; the west main collects RW1, PD1 and GP2; GP4 enters the treatment inlet pit directly. Both mains are 300 mm at 0.5% and arrive at inlet pit 3 (floor RL 29.43, 600 mm sediment sump); the west main runs along the rear of the building in a 0.45 m service strip, directly above the outlet pipe. Pipe alignments, pits and inverts are drawn on SW-Q04 (Appendix E; a reduced copy follows the schedule) and the long section is SW-Q05. The pit water level in operation equals the bioretention pond level; the hydraulic grade line at every ground grate remains below the grate with at least 0.12 m to spare in the 10% AEP event and 0.06 m in the 1% AEP event (Appendix D Table 23). Roof and podium downpipes, gutters and emergency overflow spouts are building hydraulics to AS/NZS 3500.3; spouts discharge to the side corridors.

Table 9: Pipe schedule (QUDM Rational Method, 5 minute inlet time; capacities full-bore Manning n 0.013)

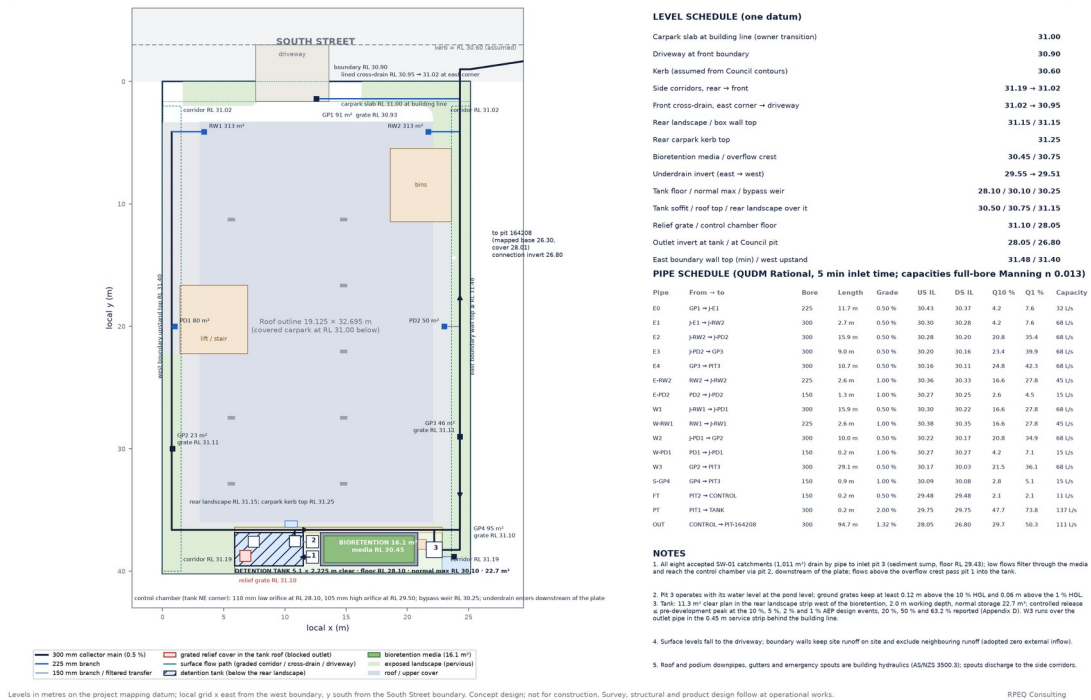
Pipe	From	To	Bore (mm)	Length (m)	Grade (%)	Upstream IL	Downstream IL	Q 10% (L/s)	Q 1% (L/s)	Capacity (L/s)
E0	GP1	J-E1	225	11.7	0.50	30.43	30.37	4.2	7.6	32
E1	J-E1	J-RW2	300	2.7	0.50	30.30	30.28	4.2	7.6	68
E2	J-RW2	J-PD2	300	15.9	0.50	30.28	30.20	20.8	35.4	68
E3	J-PD2	GP3	300	9.0	0.50	30.20	30.16	23.4	39.9	68
E4	GP3	PIT3	300	10.7	0.50	30.16	30.11	24.8	42.3	68
E-RW2	RW2	J-RW2	225	2.6	1.00	30.36	30.33	16.6	27.8	45
E-PD2	PD2	J-PD2	150	1.3	1.00	30.27	30.25	2.6	4.5	15
W1	J-RW1	J-PD1	300	15.9	0.50	30.30	30.22	16.6	27.8	68
W-RW1	RW1	J-RW1	225	2.6	1.00	30.38	30.35	16.6	27.8	45
W2	J-PD1	GP2	300	10.0	0.50	30.22	30.17	20.8	34.9	68
W-PD1	PD1	J-PD1	150	0.2	1.00	30.27	30.27	4.2	7.1	15
W3	GP2	PIT3	300	29.1	0.50	30.17	30.03	21.5	36.1	68
S-GP4	GP4	PIT3	150	0.9	1.00	30.09	30.08	2.8	5.1	15
FT	PIT2	CONTROL	150	0.2	0.50	29.48	29.48	2.1	2.1	11
PT	PIT1	TANK	300	0.2	2.00	29.75	29.75	47.7	73.8	137
OUT	CONTROL	PIT-164208	300	94.7	1.32	28.05	26.80	29.7	50.3	111

SW-Q04 Proposed drainage, treatment and detention — normal operation

Collection to inlet pit 3, bioretention, overflow to detention, controlled discharge to South Street pit 164208

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**Figure 4: Pipe, pit, treatment and tank locations (SW-Q04 reduced; full size in Appendix E)****4.6 Surface levels and allotment drainage**

One datum and one level schedule apply to the slab, driveway, corridors, pits, tank, treatment and receiver (Table below and SW-Q04). The carpark slab is RL 31.00 at the building line; the driveway falls to RL 30.90 at the boundary and the crossover to the kerb (about RL 30.60). The side corridors fall at 0.45% from RL 31.19 at the rear to RL 31.02 at the front, into a lined cross-drain along the frontage that discharges onto the driveway at RL 30.95. The rear landscape is RL 31.15, kerbed from the carpark at RL 31.25. Grates GP2, GP3 and GP4 sit 30 to 50 mm below the surrounding corridor and landscape; GP1 sits in a sag beside the driveway at RL 30.93.

Allotment drainage applies as roof and property drainage under AO6.1 and PSP2 section 1.2.1. The application is a material change of use on the single existing lot; no inter-allotment drainage is created. The internal drainage, tank and treatment system remain private common property of the body corporate (PSP2 section 1.3.1.1(5)(g)).

Table 10: Level schedule (one datum)

Element	RL (m)
Carpark slab at the building line (owner-stated entrance transition)	31.00
Driveway at the front boundary / kerb (assumed from Council contours)	30.90 / 30.60
Side corridors, rear to front	31.19 to 31.02
Front cross-drain, east corner to driveway edge	31.02 to 30.95
Rear landscape / bioretention wall top / rear carpark kerb top	31.15 / 31.15 / 31.25
Bioretention media / extended detention crest	30.45 / 30.75
Underdrain invert, east to west / box floor	29.55 to 29.51 / 29.46
Inlet pit 3 floor / overflow pit 1 crest / filtered pit 2 floor	29.43 / 30.75 / 29.33
Tank floor / normal maximum / bypass weir / soffit / roof top	28.10 / 30.10 / 30.25 / 30.50 / 30.75
Rear landscape over the tank / relief grate / control chamber floor	31.15 / 31.10 / 28.05
Outlet invert at the control chamber / at Council pit 164208 (mapped base 26.30)	28.05 / 26.80
East boundary wall top (minimum) / west boundary upstand top	31.48 / 31.40

5. MAJOR STORM AND BLOCKAGE ASSESSMENT

5.1 Surface levels and earthworks

Earthworks are required. Cut about 0.2 m on the west side, fill 0.35 to 0.7 m on the east side and rear-east with retaining walls on the east and rear boundaries; treatment box excavation to RL 29.21 (about 1.9 m below the rear landscape); tank excavation to RL 27.80 (about 3.4 m below the rear landscape) within 0.625 m of the rear boundary and 5.90 m of the west boundary, so the tank and box are built in a shored excavation with the neighbouring ground supported. Earthworks are required. The carpark slab is about 0.19 m above the mapped ground at the front centre; the rear landscape is 0.62 m above the mapped ground at the rear-east corner and -0.16 m below it at the rear-west corner. Retaining walls on the east and rear boundaries carry the fill; their tops are set by the corridor water levels in section 5.2. Adjoining ground is mapping-resolution evidence; boundary and neighbouring spot levels are to be surveyed before detailed design.

5.2 Flows above the internal capacity, up to the 1% AEP event

Flows that exceed the piped system, or that cannot enter it, run on graded surfaces to South Street. The two side corridors (1.5 m wide, turf, n 0.035) fall to the front; a lined cross-drain (1.0 m, n 0.015) along the frontage carries them to the driveway; the driveway and crossover deliver them to the kerb. The route lies wholly outside the building line, so the roller shutter and the covered carpark are not on it. The whole-site 1% AEP uncontrolled peak of 73.8 L/s (every-pattern maximum 90.5 L/s) is applied to each section in turn, which is conservative for the corridors because the two sides share it.

Table 11: Surface route hydraulics for the whole-site 1% AEP flow (Manning normal depth)

Section	Flow (L/s)	Width (m)	n	Slope (%)	Depth (mm)	Velocity (m/s)	Depth $\times$ velocity (m ² /s)
East corridor, whole-site 1 % flow	74	1.5	0.035	0.45	118	0.42	0.05
West corridor, west catchments 1 %	36	1.5	0.035	0.45	75	0.32	0.02
Front lined cross-drain, whole-site 1 %	74	1.0	0.015	0.64	82	0.90	0.07
Driveway to crossover, whole-site 1 %	74	6.0	0.016	6.17	14	0.89	0.01
Crossover to kerb, whole-site 1 %	74	6.0	0.016	10.00	12	1.03	0.01
East corridor, every-pattern maximum 1 % inflow	90.5	1.5	0.035	0.45	134	0.45	0.06

All sections are within the QUDM Table 7.3.6 major-storm limits (0.4 m depth, 0.6 m²/s). The east boundary wall top of at least RL 31.48 contains the every-pattern maximum in the east corridor with 150 mm freeboard; the west upstand to RL 31.40 contains the west corridor. At the 90.5 L/s individual-pattern stress flow, the cross-drain, driveway and crossover remain within those limits; their depths are 93, 16 and 14 mm respectively. The rear-box rim and tank relief-grate water levels are RL 31.168 and 31.197, below the carpark kerb RL 31.25.

5.3 Blockage and the route through the driveway crossover

Seven blockage scenarios were tested. In each, the piped or controlled element is assumed lost and the water is followed to South Street. The pre-development flow control (the tank plate) sits at the end of the treatment train, so a blockage upstream of it diverts water onto the surface route before it reaches the tank, and a blockage at the tank is relieved by the bypass weir and, if the outlet pipe itself is lost, by the grated relief cover in the tank roof, which spills onto the rear landscape.

Table 12: Blockage scenarios and the resulting route to South Street

Case	Element blocked	Response and check
S1	GP1 front grate 50 % (QUDM 7.5.2)	Front landscape sag ponds to the cross-drain lip and spills across the driveway to South Street; the front landscape is graded to the driveway edge
S2	Treatment inlet pit 3 and overflow pit 1 both blocked (rear box cannot discharge)	Box fills to the wall top RL 31.15 and overtops its full perimeter onto the rear landscape; the rear landscape (RL 31.15, kerbed from the carpark at RL 31.25) drains to both side corridors, then the front cross-drain and driveway
S3	Orifice plate blocked	Tank rises to the bypass weir crest RL 30.25 in the control chamber and bypasses the plate into the same 300 mm outlet pipe (capacity 111 L/s)
S4	Outlet pipe to Council pit blocked	Tank surcharges to the 900 x 900 grated relief cover in its roof at RL 31.10 and spills onto the rear landscape, which drains to the side corridors as in S2 (85 mm of head over the grate, 65 mm below the carpark kerb); the tank, covers, joints and vents are designed for a water level of RL 31.25
S5	Roof or podium downpipes blocked	Roof and podium emergency overflow spouts (building hydraulics, AS/NZS 3500.3) discharge onto the side corridors, which run to the driveway independently of the piped system
S6	Roller shutter closed	The surface route lies wholly outside the building line ($y < 1.62$); the covered carpark receives no direct rainfall and no design flow enters it, so the shutter position does not affect the route. Rear water is held below the carpark kerb (RL 31.25).
S7	South Street backwater	Free surface discharge assumes the road water level at the crossover stays below the boundary RL 30.90. If the road floods higher the site ponds against the frontage up to that level; the carpark (RL 31.00) remains above until the road reaches RL 31.00. Council road flood levels are requested for verification at operational works.

In the governing case S2 the bioretention box fills to its wall top RL 31.15 and overtops its 21.4 m perimeter with 16 mm of head; the rear landscape drains to both corridors and the water stays 84 mm below the rear carpark kerb. Drawings SW-Q04 and SW-Q06 show the normal and blocked conditions on the same architectural base; SW-Q05 is the continuous profile and SW-Q07 the containment sections.

5.4 Protection of adjoining properties (PO3)

Peak discharge from the site does not exceed the pre-development peak at the minor and major design events or at any event between them (10% to 1% AEP; section 4.4); at the more frequent events below the minor design storm the controlled peak exceeds the pre-development peak by at most 5.6 L/s, and that excess leaves the site by pipe to Council pit 164208, not across any boundary. Site runoff leaves only at the lawful point of discharge: by pipe to Council pit 164208 in normal operation and by the driveway crossover to the South Street kerb in the surcharge and blockage cases. Boundary walls on the east and rear and the upstand on the west prevent overland flow onto the neighbours and exclude neighbouring runoff. Velocities on the surface route are below 1.12 m/s and the discharge to the kerb is spread across the 6 m crossover, so no increase in velocity reaches adjoining land. Runoff volume increases with the added impervious area and is released to the piped Council system at controlled rates; the bioretention pond and media return part of the frequent-event volume to the atmosphere. No worsening or nuisance to adjacent, upstream or downstream land is created within the assessed events, subject to the operational-works verification in section 8.1.

6. STORMWATER QUALITY MANAGEMENT

6.1 Receiving environment and design objectives

Council assesses Item 3 against PO9 and PO10 of the Healthy waters code. PO9 requires that development protects and does not adversely impact the environmental values or water quality of receiving waterways; PO10 requires that the entry to and transport of contaminants in stormwater or waste water is avoided and has no acceptable outcome, so it is answered on performance. AO9.1 applies through the six-or-more-dwellings limb (15 dwellings) although the site is below 2,500 m². The design objectives are the minimum reductions in mean annual load from unmitigated development in the following table.

Table 13: Stormwater quality reduction targets (AO9.1)

Pollutant	Minimum reduction from unmitigated development	Achieved by the proposed treatment train
Total suspended solids	80%	Deemed to comply through the default treatment area of Planning Scheme Policy 2 section 1.3.1.1(4): finished media 16.1 m ² against 15.17 m ² required (1.5% of 1,011 m ²), all site runoff conveyed to the media with the bypass above RL 30.75 not credited; no MUSIC model, so no modelled percentage is claimed (section 6.3)
Total phosphorus	60%	Deemed to comply through the default treatment area of Planning Scheme Policy 2 section 1.3.1.1(4): finished media 16.1 m ² against 15.17 m ² required (1.5% of 1,011 m ²), all site runoff conveyed to the media with the bypass above RL 30.75 not credited; no MUSIC model, so no modelled percentage is claimed (section 6.3)
Total nitrogen	45%	Deemed to comply through the default treatment area of Planning Scheme Policy 2 section 1.3.1.1(4): finished media 16.1 m ² against 15.17 m ² required (1.5% of 1,011 m ²), all site runoff conveyed to the media with the bypass above RL 30.75 not credited; no MUSIC model, so no modelled percentage is claimed (section 6.3)
Gross pollutants greater than 5 mm	90%	Captured at source before the media: 5 mm litter baskets in ground inlets GP1 to GP4, gutter leaf guards and downpipe leaf diverters on every roof and podium collector, and the 600 mm sediment sump in pit 3; every catchment passes one of these (section 6.2)

The site discharges to the Council piped network in South Street within the mapped Cleveland catchment, entering at gully pit 164208 (shown on the aerial below with its 450 mm outlet pipe); the network's traced pipes end at pit 164220 and the ultimate outfall is not recorded in the asset data, with Moreton Bay the regional receiving water. The environmental values and water quality objectives scheduled for that receiving basin under the Environmental Protection (Water and Wetland Biodiversity) Policy 2019 apply; Council's AO9.1 load reductions are the adopted design objectives for this site. The Information Request frames Item 3 against the Healthy waters code alone and identifies no further State trigger for this application.



All eight catchments drain to inlet pit 3 at the rear. Gross pollutants are controlled at source: 5 mm litter baskets in the four ground inlets GP1 to GP4, leaf guards on gutters and leaf diverters on the roof and podium downpipes, and a 600 mm sediment sump in pit 3 ahead of the distribution apron; the 90% gross pollutant objective is therefore met at the inlets rather than by an in-line unit at the submerged pit. Low flows spread from the apron across the 16.1 m² media, filter through it and drain by the underdrain to the filtered-water pit 2 at the west end and on to the control chamber downstream of the orifice plate, so the treated flow is never backed up by the tank. Flows above the extended detention crest RL 30.75 pass the overflow pit 1 and a 300 mm transfer into the tank beside it. Bypassed water is not credited as treated.

Planning Scheme Policy 2 section 1.3.1.1(4) provides a default bioretention treatment area of 1.5% of the contributing catchment area in lieu of modelling. For the whole site that is 15.17 m²; the adopted finished media of 16.1 m² exceeds it. Council's Information Request asked for a MUSIC model; Council's written acceptance of the default route in place of that model is requested with this response. If Council still requires a model it will be prepared with the same catchments, dimensions and routing and the electronic file supplied.

The finished net filter media is 16.1 m² (nominal 7.4 × 2.175 m). Media follows the archived DS-078 basis: saturated hydraulic conductivity target 200 mm/h with a project review range of 100 to 300 mm/h, pH 5.5 to 7.5, electrical conductivity below 1.2 dS/m, total nitrogen below 800 mg/kg, total phosphorus below 40 mg/kg and organic content 3 to 10% under the specified tests; grading, bridging, layer, liner and testing requirements are in the supporting quality specification. Layers are 600 mm filter media, 100 mm transition and 200 mm drainage with a 100 mm slotted underdrain (Type 2 sealed profile, DS-072). Planting is 128 plants at 8 per m², *Carex appressa* and *Ficinia nodosa* in staggered groups.

The rear landscape strip between the building and the rear boundary holds, from west to east, the detention tank, the overflow and filtered-water pits, the bioretention box and the inlet pit (SW-Q08). Location and structure:

- strip from x 5.90 to 22.85 m; a 0.45 m service strip behind the building line carries the west main over the outlet pipe;
- bioretention: sealed reinforced-concrete box 7.95 × 2.725 m external, finished media 7.4 × 2.175 m (16.1 m²), Type 2 sealed profile (DS-072);
- pit 3 (inlet and sediment sump) and the distribution apron at the east end; pit 2 (filtered water) and pit 1 (overflow) in a column at the west end between the box and the tank;
- maintenance from the carpark rear aisle across the RL 31.25 kerb along the full north side and from the tank roof at the west end (50% of the perimeter, above the Water by Design 40% guide).

Levels:

- media RL 30.45, fixed by the gravity arrival of the front ground inlet: with the extended detention crest at RL 30.75 the 1% AEP hydraulic grade line at GP1 is RL 30.87, 64 mm below its grate;
- wall top RL 31.15 level with the rear landscape; the set-down of 0.70 m from the surround exceeds the Water by Design 2014 Table 8 allotment guide of 0.5 m for that reason, and the wall is guarded;
- underdrain falling at 0.5% from RL 29.55 at the east end to 29.51 at pit 2; box floor RL 29.46.

Connections:

- filtered transfer, 150 mm, from pit 2 to the control chamber at RL 29.48, downstream of the orifice plate, so the underdrain is never drowned by the tank; the required filtration capacity of 2.01 L/s at the upper conductivity is within the underdrain capacity of 3.65 L/s;
- overflow transfer, 300 mm, from pit 1 into the tank east wall at RL 29.75; the entry is below the tank normal maximum and runs full with 0.76 m of head from the pit 1 water level, capacity 172 L/s against the 73.8 L/s whole-site 1% AEP flow.

Offsets from the rear boundary, dimensioned on SW-Q08:

- filter bed 0.900 m; outer wall face 0.625 m; footing and excavation allowance 0.125 m; facility allowance 0.100 m;
- tank west face 5.90 m from the west boundary; tank and box walls 0.45 m behind the building line.

These are concept dimensions on the architectural grid, not surveyed setbacks. The tank and box are built in one shored excavation to RL 27.80; the boundary survey and the structural design confirm that the footings and excavation sit wholly within the site.

6.6 Drainage and landscape coordination

The tank, the bioretention and the rear carpark kerb sit within the rear common open space shown on the architectural plans. The tank roof is 0.40 m below the landscape surface, so the ground over it is turf or garden bed, not tree planting; planting positions, tree root zones and the communal use of that space are to be coordinated with the Item 6 landscape plan on this layout. Nothing is placed below the carpark slab: the collectors below the slab are shallow pipes, and the tank and box lie wholly outside the building footprint, 0.45 m behind the building line (Appendix D Table 24). The building's rear footing, the shored excavation beside it and the boundary retaining walls are structural design inputs.

7. CONSTRUCTION AND MAINTENANCE

7.1 Construction phase management

Construction controls must prevent sediment and other contaminants entering the stormwater system and protect treatment media from damage and clogging. Temporary drainage is coordinated with the earthworks sequence and a safe overflow path is maintained during construction.

The principal contractor is responsible for an approved site-specific erosion and sediment control plan before disturbance, implemented through the construction environmental plan. The superintendent checks implementation and the civil designer releases treatment hold points.

Temporary clean and dirty water separation, staged disturbance, a stabilised entry, stockpile protection, appropriate sediment controls and a safe bypass to the authorised drainage route are defined on the construction drawings. A silt fence is not used as an in-stream sediment basin and the biofilter does not treat construction sediment.

Controls are inspected before forecast rain, daily while earthworks are active and after runoff-producing rain, with rainfall, condition, sediment capacity, repairs and photographs recorded. Failed controls are restored before the contributing disturbance continues. Completed exposed surfaces are stabilised promptly and sediment tracking to South Street is prevented.

Fuel, chemicals and erodible materials are stored covered or bunded with spill kits and controlled refuelling. Concrete washout and paint or chemical waste are collected in contained facilities; nothing is discharged to the ground drainage or the filter. Dewatering water requires assessed quality and an approved disposal method. Washwater, sewage and trade waste connections to stormwater are prevented.

The box, liner, underdrain and tank are constructed with temporary protection while high-sediment flows are kept outside the treatment media. Filter media is not placed until the contributing construction sediment risk is mitigated and the superintendent or designer releases the area. The 600 mm media is placed in two lightly and evenly compacted lifts with level layer tops. Construction vehicles and stockpiles are excluded from the media.

Treatment hold points:

1. pre-start review of approved drawings, the erosion and sediment control plan, set-out, datum, services and responsibilities;
2. completed excavation, liner, hydraulic structures, tank, control plate and underdrain before covering, with photographs and surveyed inverts;
3. approved batch tests and dockets and a stabilised contributing catchment before filter placement;
4. installed layer depths, levels and conductivity before planting;
5. planting establishment, a functional drainage, overflow and relief test, and maintenance handover.

Hydraulic structures, the control plate and underdrains are surveyed to the approved tolerance, generally ± 25 mm under DS-078; actual ponding and freeboard are verified rather than consumed by tolerance.

Before handover the contractor provides as-constructed plans and sections with the level datum, product and batch certificates, liner and pipe inspection records, media depth, grading, chemistry and conductivity results, the plant schedule and establishment photographs, drainage, overflow and relief commissioning evidence, rectification close-out, access instructions and the asset-specific maintenance manual.

7.2 Operation and maintenance

The body corporate retains the private drainage, detention and treatment assets as common property, appoints a competent maintenance contractor, funds routine and responsive maintenance and keeps the maintenance register. No transfer to Council is proposed. Personnel and equipment access to the filter, inlet pit, control chamber, tank covers and cleanouts is from the carpark rear

aisle across the kerb and from the rear landscape over the tank; no routine entry into a pit or the tank is assumed.

Inspections are every two months in the wet season and every four months in the dry season, more often during establishment or high litter loading, after significant runoff, reported overflow, spills or damage, and at least once a year during or immediately after rainfall above 50 mm in a day.

Drawdown is checked about 12 hours after rain ceases. No inspection frequency overrides an immediate blockage or safety response.

Inlet baskets and the sediment sump: inspect each visit and after significant runoff; empty before the manufacturer's capacity threshold or, absent product instructions, at 50% of the available storage; record the quantity removed and the disposal route.

Control chamber and plate: inspect each visit; clear the screen and both orifices (110 mm and 105 mm); confirm the bypass weir and the relief grate in the tank roof are clear; any change to the plate, weir or relief requires engineering review because it changes the discharge control.

Tank: inspect through the access covers each visit for sediment, standing water after 24 hours and cover condition; remove sediment before it reaches 100 mm; keep the covers sealed and trafficable.

Filter surface and vegetation: investigate water remaining more than 12 hours after rain, crusting, erosion or settlement; maintain at least 95% plant cover; replace dead plants and remove weeds; no fertiliser, pesticide, mowing or topsoil import over the media.

Underdrain cleanouts, overflow and bypass: keep caps secure and openings and routes clear; investigate restricted discharge and flush by an approved method, disposing of the flushing waste rather than returning it across the filter.

Records identify the asset, date and time, inspector, weather, photographs, measured condition, action, waste disposal, the responsible person and the close-out date; performance and budget are reviewed annually. Building management keeps the bin store clean and covered, collects litter promptly, keeps washwater out of surface drains and directs bin-wash wastewater only to an approved sewer arrangement. Residents and cleaners receive the stormwater asset and spill-reporting instructions at occupation and on management changes.

8. CONCLUSIONS AND CERTIFICATION

8.1 Stormwater quantity (Item 2; PO3, PO4, PO6)

The amended concept design collects all roof, podium and ground runoff from the eight defined catchments in pits and pipes sized for the 10% AEP event and checked for the 1% AEP event, treats it in a rear bioretention system and detains it in a 22.7 m³ tank in the rear landscape strip beside it, whose two-stage control keeps the peak discharge at the site boundary at or below the pre-development peak for every design event from the 10% AEP minor storm to the 1% AEP major storm. Surface levels are set on one datum from the RL 31.00 carpark entrance so every inlet collects as designed and the whole site drains to the driveway. Flows above the piped capacity, and every tested blockage, follow graded corridors, a lined cross-drain and the driveway crossover to South Street with depths and velocities within the QUDM major-storm limits, outside the building line. Earthworks and boundary walls are required and are shown. PO4 is met by the minor and major system design; PO6 by the collection of roof and surface water to the lawful point of discharge with no flow into buildings; PO3 by the maintained pre-development peaks, the contained surface route and the controlled release to the Council network.

The concept relies on Council asset mapping and adopted assumptions in place of data that does not exist in the project record: the water level in pit 164208 during design storms, the road flood level at the crossover, the survey datum and boundary levels, and the structural and product design of the tank, box and plate. Each is identified in this plan with the sensitivity tested. Their verification is proposed as a condition to be satisfied at operational works, and Council's agreement to that staging is requested. A verified condition outside the tested envelope would require the affected element to be redesigned.

8.2 Stormwater quality (Item 3; PO9, PO10)

Runoff from the whole site is directed to a 16.1 m² bioretention system that exceeds the 1.5% default treatment area of Planning Scheme Policy 2 section 1.3.1.1(4), with gross pollutants controlled at every inlet, a sediment sump ahead of the media, a defined high-flow bypass, and the connected hydraulics, levels, planting, media specification, construction and maintenance provisions set out in sections 6 and 7 and on SW-Q08. The entry and transport of contaminants is avoided by the source controls and the management measures. Council's written acceptance of the default treatment area route in place of the requested model is sought; the design does not otherwise depend on that acceptance.

8.3 Certification

This report, its calculations and drawings were prepared and reviewed by Craig Harris, RPEQ Civil Engineer, registration 13194, RPEQ Consulting. Revision 5 retains the stormwater assessment and incorporates the additional water-supply hydraulic assessment in section 9 and Appendix F. Revision 6 is its public issue, with no change to engineering content. This revision is certified as a concept design for the development application and Council RFI response, subject to the stated technical qualifications and detailed-design requirements. It is not a construction issue.

Table 14: Certification

Role	Name	Registration	Revision	Date	Signature
Prepared, reviewed and certified for issue by	Craig Harris, RPEQ Civil Engineer	RPEQ 13194	Revision 6	15 October 2026	

9. WATER SERVICING

This supplementary section responds to Council water Information Request Item 4 within the revised SWMP. It retains the assumed-service assessment and adds the independent assessment of the subsequently supplied current and ultimate network models. Appendix F contains the calculations, comparison, sensitivity and adverse findings; Appendix G contains the civil water concept WS-01. The existing stormwater assessment in sections 1–8 and Appendices A–E remains retained. [R1; M1–M7]

Adopt the independent EPANET current/ultimate predictions as the conditional network basis for concept assessment, with the retained assumed-service method as a transparent comparison. Under the shared service assumptions, provide domestic pressure boosting, a fire pump and a brigade booster assembly. The governing tested ultimate differences are 66.3 kPa at 3 L/s domestic and 729.2 kPa at 30 L/s fire (with 2 L/s simultaneous domestic) under varying maximum-day background. These are calculated pressure differences, not selected equipment duties. Neither method establishes reliable indefinite ultimate supply or justifies deleting the storage contingency. Confirm datum, network operating basis, actual losses, permitted pumping and assured refill/storage during detailed design, before construction. [C01, C02; M2, M3, M6]

The assumed-service calculation gives domestic/fire differences of 86.2/764.9 kPa. The current model gives 36.0/693.9 kPa and the ultimate model gives 66.3/729.2 kPa, with varying maximum-day background for fire. The complete source-linked comparison remains in Appendix F.

Sources: Appendix F, C01/C02 and M2 design comparison. Model pressure assumes concept connection RL30.90 and common datum; fire event is 30 L/s plus 2 L/s domestic. Differences are not pump selections. The two-thirds peak-hour fire comparison, full HGL/time records, sensitivity and duration findings remain in Appendix F.

Daily fire events complete; ultimate extended source storage depletes, including the paired baseline. No-storage adequacy and indefinite sustainable supply are not established. Resolve assured refill, storage/reliability and permissible direct pumping before issue of the detailed design for construction. The architect and hydraulic/fire designer must coordinate actual meter, pump, brigade booster and any tank footprint with access, landscaping and other services.

The completed stormwater detention, treatment, catchment, drainage and blockage design is retained. The rear stormwater tank is not fire-water storage. This amendment changes no stormwater geometry, outlet, level or drainage drawing; any future water-storage layout conflict requires an explicit coordinated design revision.

9.1 Meter, boosting and concept arrangement

Appendix G shows separate domestic and fire takeoffs from the mapped frontage main, asset 55284 in DMA204, nominal DN250 uPVC. The meter and fire equipment reservations are in the west frontage landscape area beside the pedestrian entry; their footprints and clearances are provisional. The map/model connection is not a surveyed or Council-approved tapping. [R2–R4; M1]

Provide a provisional DN50 domestic master meter with isolation, strainer/unions and Council-approved containment backflow protection, followed by the domestic boosting arrangement and dwelling distribution. Final meter size, flow range, pressure loss and the approved submeter/common-area arrangement require hydraulic design. Provide a separate fire connection, provisional DN150 service, fire pump and street-accessible brigade booster assembly. Confirm device sequence, bypass, direct pumping permission and any tank/refill arrangement before detailed design. [R8–R9; C01–C03; Appendix G]

The brigade booster assembly provides fire-brigade assistance; it does not establish normal automatic supply adequacy. The fire/hydraulic designer and building certifier confirm the applicable code edition, simultaneous hydrant/sprinkler duty, pressure and duration. The retained screening duty is not a confirmed building-system duty. Coordinate actual equipment, noise, ventilation, power, access, fire separation, other services and any storage footprint with the architect and landscape designer. [R9; Appendix F1, F7]

9.2 Infrastructure Works Code response

The following concept responses address the water matter in Council Item 4. The held City Plan version is the assessment reference. Detailed design must confirm applicable requirements and demonstrate compliance with the stated supply and storage conditions. Reliable supply and storage remain explicit conditions rather than an unqualified capacity claim. [R1, R6; Appendix F]

Water servicing response to Infrastructure Works Code PO5–PO9

Outcome	Concept response and remaining verification
PO5 — network connection and capacity	Separate takeoffs, common demand/pressure budgets and conditional current/ultimate predictions. Confirm connection, reliable supply/refill and storage before final capacity acceptance.
PO6 — whole-of-life cost	Retain boosting where the shared pressure budgets require it; compare accepted direct, pumped and tank-assisted options using verified supply and equipment duty.
PO7 — access and maintenance	Street-accessible frontage concept; verify meter/brigade access, working clearances, removal routes, bypass and isolation with actual equipment.
PO8 — fitness for design life	Select verified material, corrosion/backflow protection, pressure/shutoff rating and maintainable equipment; specify testing and maintenance in detailed design.
PO9 — reliable potable/non-potable supply	Explicit demand and hydraulic assessment retained. Sustained supply and storage are unresolved; confirm source reliability, refill and final service design during detailed design, before construction.

APPENDIX A SUPPORTING EVIDENCE AND DRAWING REGISTER

Table 15: Supporting evidence and drawing register

Reference	Document or drawing	Status in this revision
SW-01	Proposed drainage catchments on the submitted Issue A ground plan	Adopted catchment allocation (Appendix C)
SW-Q04	Proposed drainage, treatment and detention, normal operation, with level and pipe schedules	This revision (Appendix E)
SW-Q05	Stormwater long section, GP1 to Council pit 164208	This revision (Appendix E)
SW-Q06	Blockage case, surface route to South Street through the driveway crossover	This revision (Appendix E)
SW-Q07	Site cross sections: corridor containment, frontage, detention tank, bioretention	This revision (Appendix E)
SW-Q08	Rear strip: bioretention and detention tank, plan, long section and cross section with rear-boundary offsets	This revision (Appendix E)
Calculation figures	Ensemble peaks by AEP (section 4.4); storage by duration and stage-storage and stage-discharge (Appendix D)	This revision
Aerial figures	Western neighbour 4 Clayton Court (section 3.2); site and direction to the lawful point of discharge (section 3.4); receiving pit 164208 (section 6.1)	Council aerial, contour and asset imagery supplied by the author; annotations added
Council Information Request	MCU26/0056, 9 June 2026, Items 2 and 3	Request answered
Pre-lodgement minutes	PLM25/0045, 8 December 2025, page 5 (South Street discharge)	Adopted
Architectural set	Interworks Issue A, 30 October 2025: drawings 2.1, 2.2, 2.6, 5.1	Governing layout
Redland City Plan 2018 v14	Healthy waters code 9.3.1.3; Tables 9.3.1.3.1 and 9.3.1.3.2	Assessment benchmark
Planning Scheme Policy 2	Infrastructure works, sections 1.2 and 1.3	Acceptable-outcome standard
QUDM 2016	Sections 4.5, 4.6, 5.5, 5.6, 5.9, 7.3, 7.5	Design manual
ARR 2019	Book 9 v4.2 section 6.4; Book 2 chapter 5; ARR Data Hub site data (patterns, losses, pre-burst)	Design procedure
Bureau of Meteorology	2016 IFD design rainfall, site grid 27.5375° S 153.2625° E (Appendix B)	Rainfall input
Water by Design 2014	Bioretention Technical Design Guidelines v1.1; DS-070, DS-072, DS-078	Treatment design guidance and standard details
Council asset mapping	Stormwater pits, pipes and 0.25 m contours (checked 21 September 2026)	Receiving asset and terrain evidence
Calculation records	Hydrology and detention, levels and network, and drawing source calculations; retained by RPEQ Consulting	Available to Council on request
Water assessment	Section 9 and Appendices F–G; water supply hydraulic assessment and concept WS-01	Final — Issued for RFI Response; assumed-supply calculations, network hydraulic assessment and conditional supply findings retained.

APPENDIX B RAINFALL SOURCE DATA

Bureau of Meteorology 2016 IFD design rainfall intensities for the site coordinate latitude -27.548597, longitude 153.266223 (nearest grid 27.5375° S, 153.2625° E), downloaded 21 September 2026 and verified against a fresh download. Values are rainfall intensity in mm/h; column headings are annual exceedance probability.

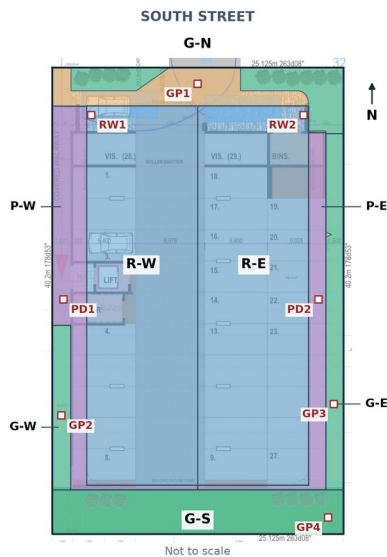
Table 16: Rainfall intensities by duration and annual exceedance probability (mm/h)

Duration	Minutes	63.2%	50%	20%	10%	5%	2%	1%
1 min	1	162	183	250	295	338	396	439
2 min	2	135	153	211	252	294	351	395
3 min	3	127	143	198	236	273	324	364
4 min	4	121	137	188	223	258	304	340
5 min	5	116	131	179	212	244	287	320
10 min	10	94.9	107	146	172	197	229	253
15 min	15	80.5	91.1	124	146	166	193	214
20 min	20	70.1	79.3	108	127	145	169	187
25 min	25	62.3	70.5	96.1	113	130	152	168
30 min	30	56.1	63.6	86.8	102	118	138	153
45 min	45	43.9	49.8	68.3	81	93.4	110	123
1 hour	60	36.5	41.4	57	67.8	78.6	93.1	104
1.5 hour	90	27.8	31.6	43.8	52.4	61.1	72.9	82.2
2 hour	120	22.9	26	36.3	43.6	50.9	61.1	69.1
3 hour	180	17.4	19.8	27.8	33.5	39.4	47.5	54
4.5 hour	270	13.3	15.2	21.4	25.9	30.6	37	42.2
6 hour	360	11	12.6	17.9	21.7	25.6	31.1	35.5
9 hour	540	8.53	9.78	13.9	16.9	20	24.4	27.9
12 hour	720	7.12	8.19	11.7	14.3	16.9	20.6	23.6
18 hour	1080	5.53	6.38	9.18	11.2	13.3	16.3	18.7
24 hour	1440	4.61	5.34	7.72	9.45	11.2	13.7	15.8
30 hour	1800	4	4.63	6.73	8.25	9.83	12	13.8
36 hour	2160	3.55	4.12	6	7.37	8.78	10.8	12.4
48 hour	2880	2.92	3.39	4.97	6.12	7.31	9.01	10.4
72 hour	4320	2.18	2.53	3.72	4.6	5.52	6.84	7.91
96 hour	5760	1.74	2.02	2.98	3.69	4.43	5.5	6.37
120 hour	7200	1.45	1.68	2.46	3.05	3.68	4.56	5.3
144 hour	8640	1.24	1.43	2.09	2.59	3.12	3.86	4.49
168 hour	10080	1.08	1.24	1.8	2.23	2.69	3.31	3.86

APPENDIX C PROPOSED DRAINAGE CATCHMENTS

Drainage catchments

SW-01 | CONCEPT
12 September 2026



CATCHMENT ALLOCATION

Catchment / outlet	Impervious	Pervious	Total m ²	% of site
R-W → RW1 (Roof west)	312.65	0.00	312.65	30.9
R-E → RW2 (Roof east)	312.65	0.00	312.65	30.9
P-W → PD1 (Upper surface west)	80.07	0.00	80.07	7.9
P-E → PD2 (Upper surface east)	50.04	0.00	50.04	5.0
G-N → GP1 (Ground front)	48.06	42.78	90.84	9.0
G-W → GP2 (Ground west)	0.00	23.02	23.02	2.3
G-E → GP3 (Ground east)	0.11	45.95	46.07	4.6
G-S → GP4 (Ground rear)	0.00	95.48	95.48	9.4
TOTAL	803.57	207.22	1,011	100.0

Areas in m². Site total shown as the adopted 1,011 m² (components sum to 1,010.79 before rounding). Percentages are of the site area.

■ Roof
 ■ Exposed upper surface
 ■ Exposed ground landscape
 ■ Exposed ground hardstand

COLLECTION BASIS

RW1 / RW2: proposed west / east roofwater collectors.
 PD1 / PD2: proposed west / east podium drainage outlets.
 GP1-GP4: proposed front, west, east and rear ground inlets.
 Collector symbols are concept locations; drainage divides and surface falls are design intentions for level development.

The architectural landscape schedule remains 264 m².
 Rainfall-exposed ground landscape is 207.22 m².
 Upper surfaces receive no infiltration or storage credit.
 The 0.77 m² cadastral balance is included as impervious area in G-N; it is not a mapped additional parcel.

Source: Interworks Architects, drawings 2.1, 2.2 and 2.6, Issue A, 30 October 2025; submitted MCU26/0056 plans.

Roof footprint: 19.125 × 32.695 m. Upper and ground plans align at the north and west boundaries; their 0.20 m depth difference is retained.

Site-only catchment; zero external inflow adopted. Covered ground is excluded from direct rainfall. Ground landscape is assumed pervious.

Proposed discharge to South Street through the treatment/control system; final connections, treatment capture, levels and hydraulic design remain to be developed.

Figure 6: SW-01 Proposed drainage catchments on the submitted Issue A ground plan, with the catchment allocation

SW-01 shows the eight disjoint catchments and their collection points on the submitted ground-floor plan, with the allocation schedule below the layout. Roof 625.3 m², exposed upper surface 130.1 m², exposed ground landscape 207.2 m² and ground hardstand with the cadastral allowance 48.2 m² total 1,011 m².

APPENDIX D CALCULATION PACKAGE

D1 Inputs and method

Table 17: Hydrological and hydraulic inputs

Input	Pre-development	Post-development	Source
Site area	1,011 m ²	1,011 m ²	DCDB, rounded
Impervious area	239.0 m ²	803.6 m ²	Task A.4 baseline; SW-01 catchment schedule
Pervious area	771.8 m ²	207.2 m ²	Task A.4 baseline; SW-01 catchment schedule
Inlet time	15 min	5 min	QUDM Table 4.6.2
Design criterion	peak at the site boundary	controlled peak at or below pre-development at 10%, 5%, 2% and 1% AEP; 20%, 50% and 63.2% AEP reported	Healthy waters code Table 9.3.1.3.2 and AO4.1; PSP2 1.2.1.4(1); QUDM 5.5.2(iii)
Impervious losses	IL 1.0 mm, CL 0.0 mm/h	same	ARR Book 9 section 6.4.2
Pervious losses	storm IL 26.0 mm less median pre-burst, CL 1.7 mm/h	same	ARR Data Hub (site); ARR Book 9 section 6.4.2
Pre-burst at 60 min by AEP (mm)	63.2% 2.8, 50% 2.8, 20% 6.0, 10% 8.4, 5% 10.6, 2% 9.6, 1% 9.4	same	ARR Data Hub Preburst50
Temporal patterns	East Coast (North) point, 10 per duration and band; 24 durations	same	ARR Data Hub
Routing step; discharge coefficient	not applicable	one sixth of the pattern increment, 10 to 60 s; Cd 0.60	QUDM equation 5.3
Outlet pipe	not applicable	300 mm, 95 m, n 0.013, K 1.5	Concept route SW-Q04
Underdrain bypass added to the controlled peak	not applicable	2.0 L/s	Filtration capacity at Ksat 300 mm/h; enters downstream of the plate
Tailwater (design); tank floor / normal maximum / bypass crest	not applicable	RL 26.30; RL 28.10 / 30.10 / 30.25	Mapped pipe invert; this design
Climate sensitivity	+2.4 °C, duration-dependent factors (1.399 at 60 min, 1.203 at 24 h)	same	ARR Book 1 chapter 6 (held derivation)

D2 Ensemble results by AEP and duration

Table 18: Ensemble mean peaks by duration for the 10% and 1% AEP events (L/s) and routed storage (m³)

Duration (min)	10% pre	10% post	10% controlled	10% storage	1% pre	1% post	1% controlled	1% storage
10	16.7	47.7	27.4	12.1	32.7	73.8	46.6	19.6
15	25.4	46.2	29.1	13.7	45.3	71.9	50.3	21.0
20	29.4	43.7	29.7	14.2	49.7	64.1	49.3	20.7
25	30.3	43.0	29.3	13.6	51.5	64.4	50.2	21.0
30	30.6	42.7	28.9	13.5	46.7	63.4	48.6	20.4
45	28.9	38.6	27.7	12.5	45.3	58.8	46.2	19.7
60	26.9	36.7	26.2	11.1	40.7	51.4	40.6	18.1
90	24.5	32.3	24.1	9.4	37.6	44.4	36.7	16.7
120	22.3	32.0	22.9	8.5	32.2	40.1	30.1	14.2
180	20.4	20.7	20.8	7.1	31.3	31.6	28.0	12.3
270	18.1	18.4	18.7	6.0	27.2	27.5	25.0	10.2
360	17.7	18.0	18.2	5.6	28.1	28.4	27.6	10.8
540	12.2	12.5	14.4	3.5	21.3	21.6	23.2	8.7
720	11.7	12.0	13.8	3.3	25.7	26.0	26.9	9.8

Duration (min)	10% pre	10% post	10% controlled	10% storage	1% pre	1% post	1% controlled	1% storage
1080	7.7	8.0	10.0	1.9	12.2	12.5	14.5	3.6
1440	8.9	9.1	11.1	2.3	11.4	11.7	13.7	3.2
1800	6.7	7.0	9.0	1.6	7.9	8.1	10.1	1.9
2160	5.3	5.6	7.6	1.3	8.6	8.9	10.9	2.2
2880	7.0	7.2	9.2	1.7	10.3	10.6	12.6	2.9
4320	4.8	5.1	7.1	1.2	5.9	6.2	8.2	1.4
5760	4.7	4.9	6.9	1.2	6.1	6.3	8.3	1.5
7200	5.5	5.7	7.7	1.3	6.4	6.7	8.7	1.6
8640	5.6	5.8	7.8	1.3	6.6	6.9	8.9	1.6
10080	3.7	4.0	6.0	1.0	5.7	6.0	8.0	1.4

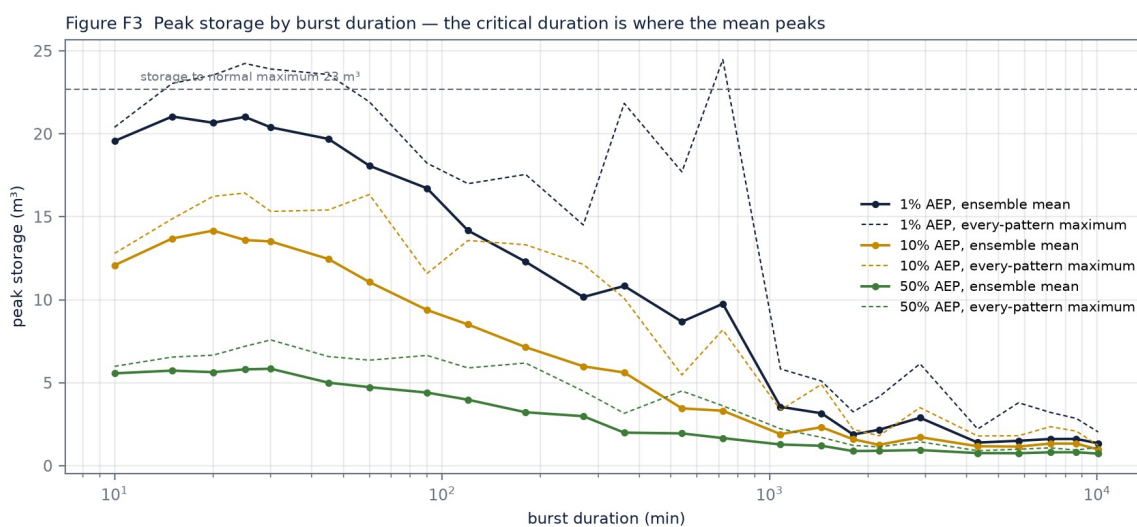


Figure 7: Peak storage by burst duration

D3 Detention control and sensitivities

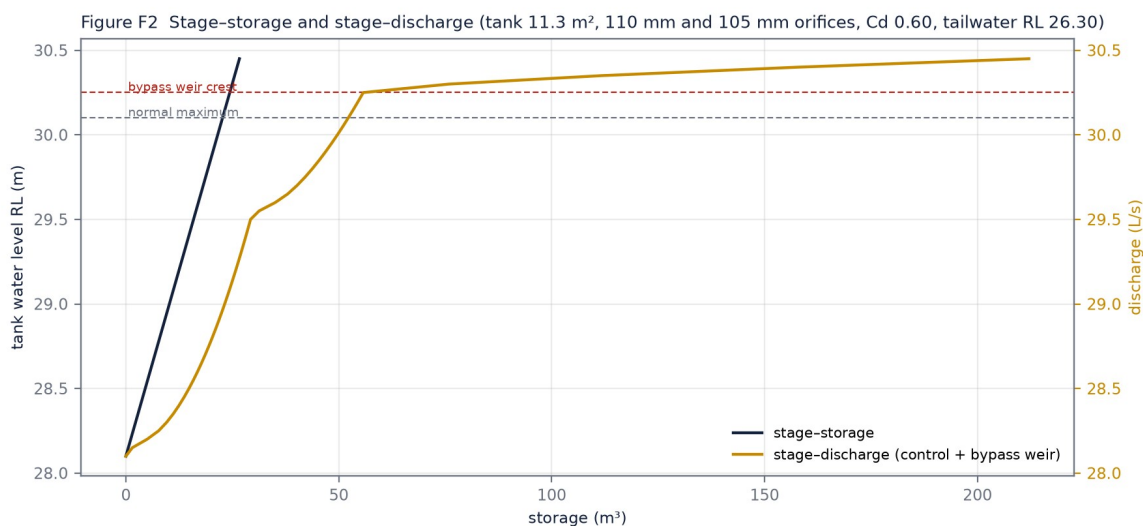


Figure 8: Stage-storage and stage-discharge of the adopted tank and control

Table 19: Stage, storage and discharge rating

RL (m)	Depth (m)	Storage (m ³)	Control discharge (L/s)	Bypass weir (L/s)
28.10	0.00	0.0	0.0	0.0

RL (m)	Depth (m)	Storage (m³)	Control discharge (L/s)	Bypass weir (L/s)
28.20	0.10	1.1	5.0	0.0
28.30	0.20	2.3	9.6	0.0
28.40	0.30	3.4	12.5	0.0
28.50	0.40	4.5	14.8	0.0
28.60	0.50	5.7	16.8	0.0
28.70	0.60	6.8	18.6	0.0
28.80	0.70	7.9	20.3	0.0
28.90	0.80	9.1	21.8	0.0
29.00	0.90	10.2	23.2	0.0
29.10	1.00	11.3	24.5	0.0
29.20	1.10	12.5	25.8	0.0
29.30	1.20	13.6	27.0	0.0
29.40	1.30	14.8	28.2	0.0
29.50	1.40	15.9	29.3	0.0
29.60	1.50	17.0	35.1	0.0
29.70	1.60	18.2	40.2	0.0
29.80	1.70	19.3	43.8	0.0
29.90	1.80	20.4	46.9	0.0
30.00	1.90	21.6	49.7	0.0
30.10	2.00	22.7	52.2	0.0
30.20	2.10	23.8	54.6	0.0
30.30	2.20	25.0	56.9	19.0
30.40	2.30	26.1	59.0	98.8

Table 20: Sensitivity results on the adopted geometry (durations to 6 hours)

Sensitivity	AEP	Pre (L/s)	Controlled (L/s)	Result	Storage (m³)	Design level RL	Any-pattern max RL
Tailwater RL 28.01	10%	30.6	29.7	Pass	14.2	29.35	29.55
Tailwater RL 28.01	5%	37.5	36.8	Pass	17.1	29.60	29.77
Tailwater RL 28.01	2%	46.3	45.0	Pass	19.4	29.81	30.04
Tailwater RL 28.01	1%	51.5	49.6	Pass	21.2	29.97	30.26
Tailwater RL 28.4	10%	30.6	31.5	Fail	16.6	29.57	29.72
Tailwater RL 28.4	5%	37.5	39.1	Fail	18.6	29.74	29.94
Tailwater RL 28.4	2%	46.3	45.2	Pass	20.9	29.94	30.21
Tailwater RL 28.4	1%	51.5	49.5	Pass	22.7	30.10	30.26
Tailwater RL 28.7	10%	30.6	34.2	Fail	18.2	29.70	29.88
Tailwater RL 28.7	5%	37.5	39.6	Fail	20.0	29.86	30.09
Tailwater RL 28.7	2%	46.3	45.3	Pass	22.2	30.06	30.26
Tailwater RL 28.7	1%	51.5	49.1	Pass	23.8	30.19	30.26
Cd 0.55	10%	30.6	29.2	Pass	15.2	29.44	29.62
Cd 0.55	5%	37.5	37.3	Pass	17.9	29.68	29.85
Cd 0.55	2%	46.3	44.4	Pass	20.2	29.88	30.13
Cd 0.55	1%	51.5	48.8	Pass	22.1	30.05	30.26
Cd 0.65	10%	30.6	30.7	Fail	13.2	29.26	29.46
Cd 0.65	5%	37.5	36.7	Pass	16.1	29.52	29.69
Cd 0.65	2%	46.3	46.2	Pass	18.4	29.72	29.93
Cd 0.65	1%	51.5	51.8	Fail	20.1	29.88	30.13
Low orifice 50% blocked	10%	30.6	30.4	Pass	19.7	29.84	30.08
Low orifice 50% blocked	5%	37.5	35.5	Pass	22.1	30.05	30.26
Low orifice 50% blocked	2%	46.3	40.8	Pass	24.3	30.25	30.27
Low orifice 50% blocked	1%	51.5	42.6	Pass	24.6	30.26	30.27

Sensitivity	AEP	Pre (L/s)	Controlled (L/s)	Result	Storage (m³)	Design level RL	Any-pattern max RL
QUDM Table 5.6.1 losses	10%	24.7	27.7	Fail	12.4	29.20	29.48
QUDM Table 5.6.1 losses	5%	30.8	32.9	Fail	15.6	29.47	29.70
QUDM Table 5.6.1 losses	2%	37.9	43.2	Fail	18.5	29.73	29.95
QUDM Table 5.6.1 losses	1%	44.5	48.4	Fail	20.3	29.89	30.16
+2.4 °C, both conditions	10%	47.0	48.1	Fail	20.2	29.88	30.11
+2.4 °C, both conditions	5%	56.7	54.4	Pass	22.7	30.10	30.26
+2.4 °C, both conditions	2%	66.1	59.8	Pass	24.5	30.26	30.27
+2.4 °C, both conditions	1%	73.3	60.3	Pass	24.5	30.26	30.27
+2.4 °C, developed only	10%	30.6	48.1	Fail	20.2	29.88	30.11
+2.4 °C, developed only	5%	37.5	54.4	Fail	22.7	30.10	30.26
+2.4 °C, developed only	2%	46.3	59.8	Fail	24.5	30.26	30.27
+2.4 °C, developed only	1%	51.5	60.3	Fail	24.5	30.26	30.27

Table 21: Adopted tank geometry and assessment basis

Item	Adopted geometry and assessment basis
Clear plan	5.10 × 2.225 m; 11.35 m², at the east end of the rear strip beside the pit column. The smallest length at the strip width that meets the criteria: lengths 4.5 to 6.5 m searched in 0.1 m steps; 4.5, 4.6, 4.7, 4.8, 4.9, 5.0 m fail.
Working depth and storage	2.0 m to normal maximum RL 30.10; 22.7 m³ normal storage; 24.4 m³ to the bypass crest RL 30.25.
Control	110 mm low orifice and 105 mm high orifice; high-orifice lower edge RL 29.50; the plate with the largest minimum margin (0.59 L/s at 5% AEP) among the 2 passing plates at that length.
Assessment	Ensemble design peaks satisfy the criteria at 10%, 5%, 2% and 1% AEP; 20%, 50% and 63.2% AEP reported. Sensitivities and individual-pattern bypass operation are reported separately.

D4 Verification

Table 22: Verification checks

Check	Result
Every temporal pattern sums to 100% within 0.05 (asserted at load)	Pass
Rainfall volume, 1% AEP 60 min burst	104.0 mm on 1,011 m² = 105.1 m³; model depth 104.0 mm
Post-development runoff volume, 1% AEP 60 min	model ensemble mean 100.58 m³ against the loss model applied to the total depth 100.53 m³
Orifice flow, QUDM equation 5.3 at 1.0 m head to centre	hand 25.244 L/s; strip integration 25.244 L/s
Timestep halving, 1% AEP critical duration	15 min: 21.044 m³ at 10 s, 21.067 m³ at 5 s, difference 0.0232 m³
Mass balance, all routed cases	maximum absolute residual 5.41e+00 m³
QUDM Rational cross-check of pre-development peaks	10%: ARR 30.6 vs Rational 29.4 L/s; 5%: ARR 37.5 vs Rational 35.1 L/s; 2%: ARR 46.3 vs Rational 44.8 L/s; 1%: ARR 51.5 vs Rational 51.8 L/s
QUDM 5.6.1 initial-loss volume estimate	564.57 m² × 28 mm = 15.8 m³ (preliminary check against the routed 21.0 m³)
Precedent scale	208 m³/ha against 14 to 24 m³ tanks in four Council precedents on 799 to 1,012 m² sites and the QUDM 30 to 400 m³/ha range

D5 Surface route and blockage cases

Surface hydraulics and the blockage cases are tabulated in sections 5.2 and 5.3 and drawn on SW-Q06 and SW-Q07; the rear box rim overtopping in case S2 is computed as a broad-crested weir over the full box perimeter.

D6 Hydraulic grade line at the ground inlets

Table 23: Hydraulic grade line at the ground inlets with the piped system operating

Inlet	Grate RL	HGL 10% RL	Freeboard 10% (m)	HGL 1% RL	Freeboard 1% (m)
GP1	30.93	30.81	0.12	30.87	0.06
GP3	31.11	30.79	0.32	30.81	0.30
GP2	31.11	30.80	0.32	30.83	0.29
GP4	31.10	30.78	0.32	30.78	0.32

The HGL is the pond level plus the overflow pit 1 weir head (83 mm at 10% AEP, 111 mm at 1% AEP over 1.2 m of weir with 50% blockage) plus pipe friction and $0.5 v^2/2g$ per structure along the arrival path.

D7 Tank placement and clearances

Table 24: Tank placement in the rear landscape strip

Item	Value
Internal plan (local x, y)	x 6.15 to 11.25 m, y 37.10 to 39.325 m; 11.35 m ² clear
Walls / external plan	250 mm; 5.60 × 2.725 m
Floor / normal maximum / bypass crest / soffit / roof top	RL 28.10 / 30.10 / 30.25 / 30.50 / 30.75; cover below the landscape 0.40 m
Clearances	west boundary 5.90 m; rear boundary 0.625 m to the outer wall face; building line 0.45 m; pits 1 and 2 0.25 m; bioretention box 1.40 m
Control chamber	0.9 × 0.9 m clear inside the tank at its north-east corner, floor RL 28.05; plate in the tank-side wall; outlet north into the service strip
Access covers	0.9 × 0.9 m clear at (7.5, 37.60), sealed; 0.9 × 0.9 m clear at (10.8, 37.55), sealed (control chamber)
Relief grate	0.9 × 0.9 m at (6.75, 38.75), RL 31.10 in a shallow dish 50 mm below the landscape; passes the whole-site 1% flow with 85 mm head at 50% blockage, 65 mm below the carpark kerb
Excavation	base slab RL 27.80, 3.35 m below the landscape, shored; groundwater to be established by the geotechnical investigation
Structural design water level (outlet blocked)	RL 31.25 (the rear carpark kerb); the tank, covers, joints and vents are designed for that head, for groundwater uplift when empty and for landscape and maintenance loads

APPENDIX E DRAWINGS

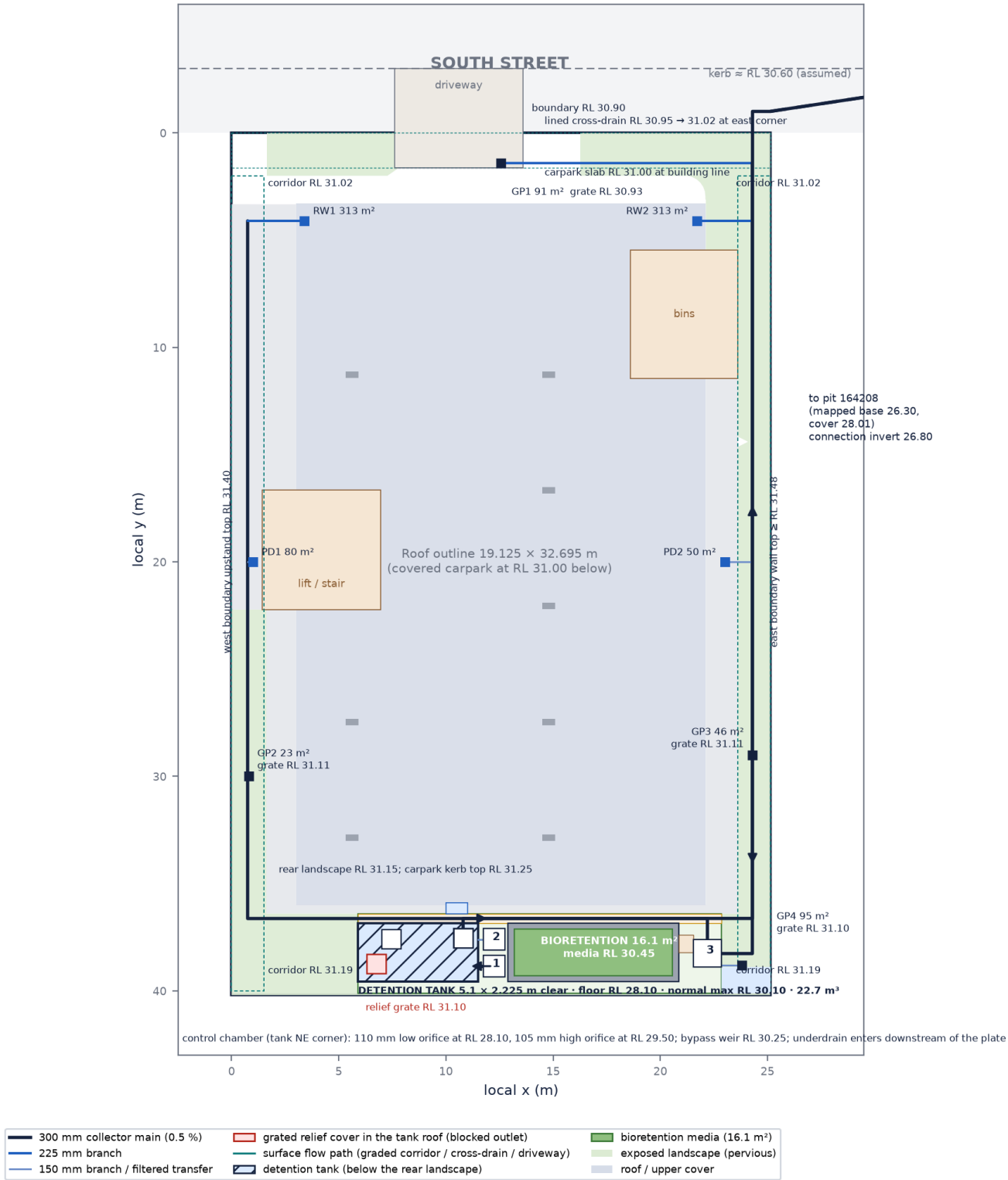
Figure 9: SW-Q04 Proposed drainage, treatment and detention, normal operation

SW-Q04 Proposed drainage, treatment and detention — normal operation

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Collection to inlet pit 3, bioretention, overflow to detention, controlled discharge to South Street pit 164208

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LEVEL SCHEDULE (one datum)

Carpark slab at building line (owner transition)	31.00
Driveway at front boundary	30.90
Kerb (assumed from Council contours)	30.60
Side corridors, rear → front	31.19 → 31.02
Front cross-drain, east corner → driveway	31.02 → 30.95
Rear landscape / box wall top	31.15 / 31.15
Rear carpark kerb top	31.25
Bioretention media / overflow crest	30.45 / 30.75
Underdrain invert (east → west)	29.55 → 29.51
Tank floor / normal max / bypass weir	28.10 / 30.10 / 30.25
Tank soffit / roof top / rear landscape over it	30.50 / 30.75 / 31.15
Relief grate / control chamber floor	31.10 / 28.05
Outlet invert at tank / at Council pit	28.05 / 26.80
East boundary wall top (min) / west upstand	31.48 / 31.40

PIPE SCHEDULE (QUDM Rational, 5 min inlet time; capacities full-bore Manning n 0.013)

Pipe	From → to	Bore	Length	Grade	US IL	DS IL	Q10 %	Q1 %	Capacity
E0	GP1 → J-E1	225	11.7 m	0.50 %	30.43	30.37	4.2	7.6	32 L/s
E1	J-E1 → J-RW2	300	2.7 m	0.50 %	30.30	30.28	4.2	7.6	68 L/s
E2	J-RW2 → J-PD2	300	15.9 m	0.50 %	30.28	30.20	20.8	35.4	68 L/s
E3	J-PD2 → GP3	300	9.0 m	0.50 %	30.20	30.16	23.4	39.9	68 L/s
E4	GP3 → PIT3	300	10.7 m	0.50 %	30.16	30.11	24.8	42.3	68 L/s
E-RW2	RW2 → J-RW2	225	2.6 m	1.00 %	30.36	30.33	16.6	27.8	45 L/s
E-PD2	PD2 → J-PD2	150	1.3 m	1.00 %	30.27	30.25	2.6	4.5	15 L/s
W1	J-RW1 → J-PD1	300	15.9 m	0.50 %	30.30	30.22	16.6	27.8	68 L/s
W-RW1	RW1 → J-RW1	225	2.6 m	1.00 %	30.38	30.35	16.6	27.8	45 L/s
W2	J-PD1 → GP2	300	10.0 m	0.50 %	30.22	30.17	20.8	34.9	68 L/s
W-PD1	PD1 → J-PD1	150	0.2 m	1.00 %	30.27	30.27	4.2	7.1	15 L/s
W3	GP2 → PIT3	300	29.1 m	0.50 %	30.17	30.03	21.5	36.1	68 L/s
S-GP4	GP4 → PIT3	150	0.9 m	1.00 %	30.09	30.08	2.8	5.1	15 L/s
FT	PIT2 → CONTROL	150	0.2 m	0.50 %	29.48	29.48	2.1	2.1	11 L/s
PT	PIT1 → TANK	300	0.2 m	2.00 %	29.75	29.75	47.7	73.8	137 L/s
OUT	CONTROL → PIT-164208	300	94.7 m	1.32 %	28.05	26.80	29.7	50.3	111 L/s

NOTES

- All eight accepted SW-01 catchments (1,011 m²) drain by pipe to inlet pit 3 (sediment sump, floor RL 29.43); low flows filter through the media and reach the control chamber via pit 2, downstream of the plate; flows above the overflow crest pass pit 1 into the tank.
- Pit 3 operates with its water level at the pond level; ground grates keep at least 0.12 m above the 10 % HGL and 0.06 m above the 1 % HGL.
- Tank: 11.3 m² clear plan in the rear landscape strip west of the bioretention, 2.0 m working depth, normal storage 22.7 m³; controlled release ≤ pre-development peak at the 10 %, 5 %, 2 % and 1 % AEP design events, 20 %, 50 % and 63.2 % reported (Appendix D). W3 runs over the outlet pipe in the 0.45 m service strip behind the building line.
- Surface levels fall to the driveway; boundary walls keep site runoff on site and exclude neighbouring runoff (adopted zero external inflow).
- Roof and podium downpipes, gutters and emergency spouts are building hydraulics (AS/NZS 3500.3); spouts discharge to the side corridors.

Levels in metres on the project mapping datum; local grid x east from the west boundary, y south from the South Street boundary. Concept design; not for construction. Survey, structural and product design follow at operational works.

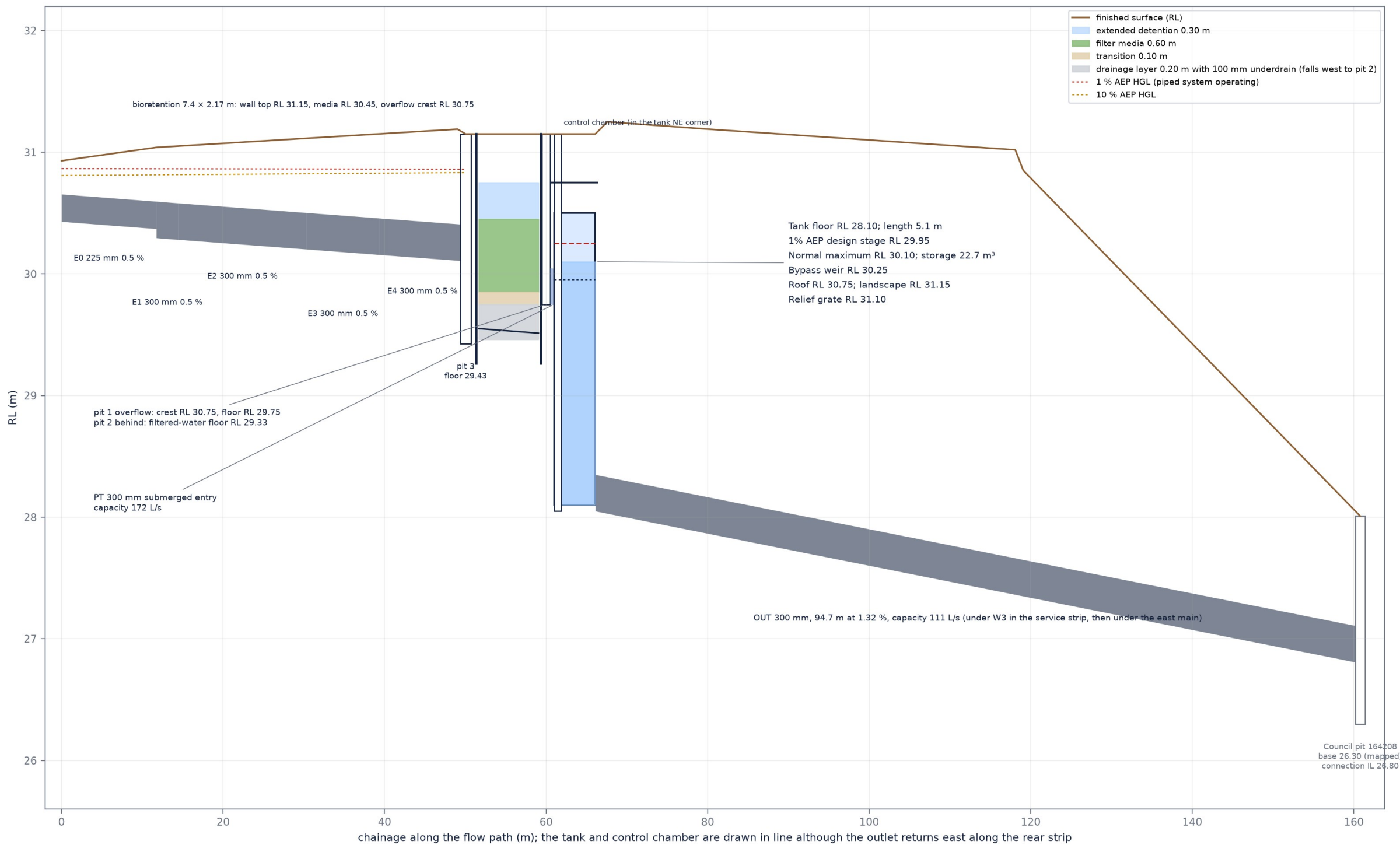
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Figure 10: SW-Q05 Stormwater long section, GP1 to Council pit 164208

SW-Q05 Stormwater long section — front inlet GP1 to Council pit 164208

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Along the flow path: east main, inlet pit 3, bioretention (east to west), overflow pit 1, detention tank in the rear strip, control chamber and outlet (horizontal exaggeration)MCU26/0056 · Stormwater Management Plan Rev 4 · Concept for RFI response



Levels in metres on the project mapping datum; local grid x east from the west boundary, y south from the South Street boundary. Concept design; not for construction. Survey, structural and product design follow at operational works.

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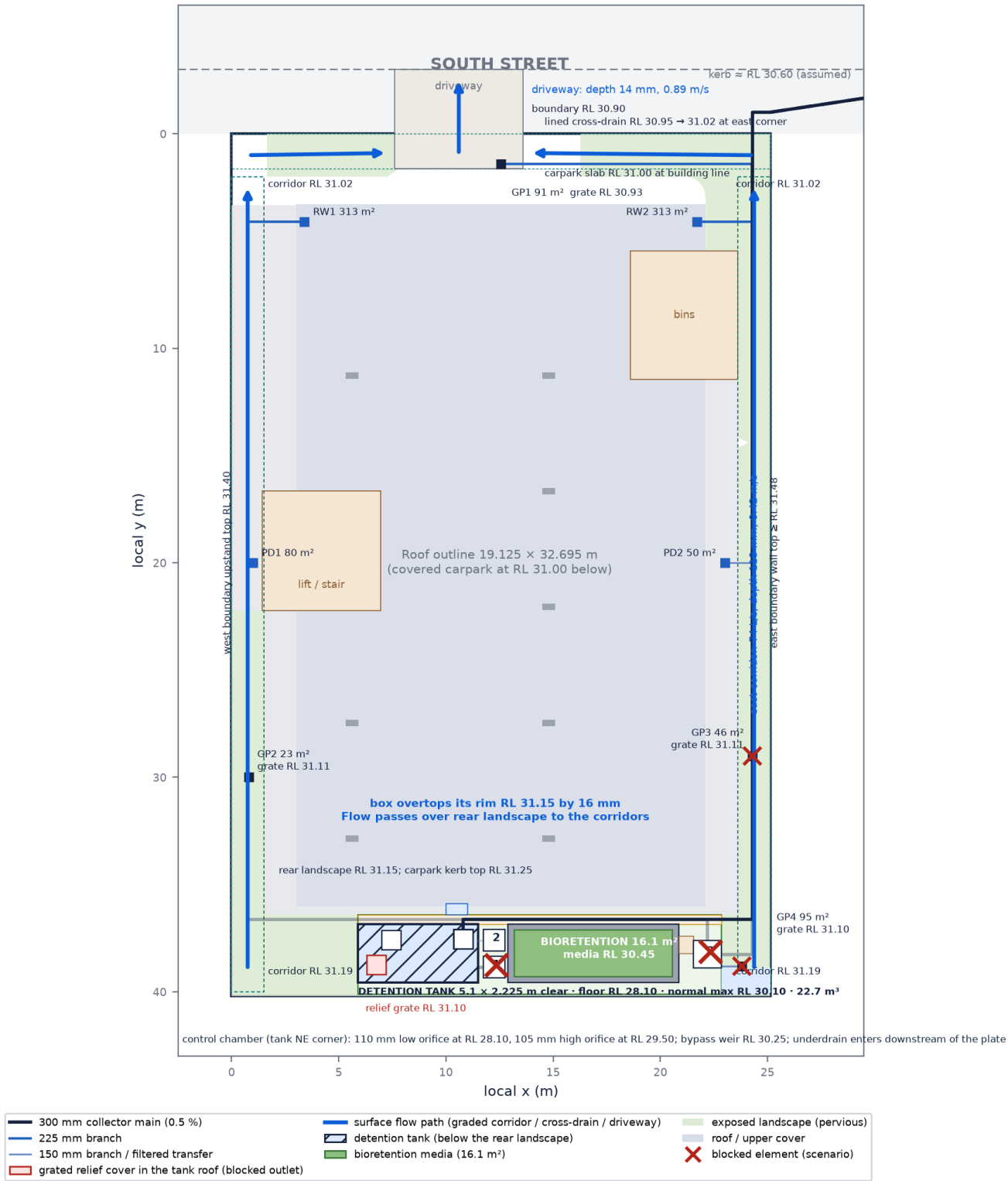
Figure 11: SW-Q06 Blockage case, surface route to South Street through the driveway crossover

SW-Q06 Blockage case — surface route to South Street through the driveway crossover

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Scenario S2: inlet pit 3 and overflow pit 1 blocked; the piped system is bypassed and the whole-site 1 % AEP flow runs on the surface

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BLOCKAGE SCENARIOS AND RESPONSES

S1 GP1 front grate 50 % (QUDM 7.5.2)

Front landscape sag ponds to the cross-drain lip and spills across the driveway to South Street; the front landscape is graded to the driveway edge

S2 Treatment inlet pit 3 and overflow pit 1 both blocked (rear box cannot discharge)

Box fills to the wall top RL 31.15 and overtops its full perimeter onto the rear landscape; the rear landscape (RL 31.15, kerbed from the carpark at RL 31.25) drains to both side corridors, then the front cross-drain and driveway

S3 Orifice plate blocked

Tank rises to the bypass weir crest RL 30.25 in the control chamber and bypasses the plate into the same 300 mm outlet pipe (capacity 111 L/s)

S4 Outlet pipe to Council pit blocked

Tank surcharges to the 900 x 900 grated relief cover in its roof at RL 31.10 and spills onto the rear landscape, which drains to the side corridors as in S2 (85 mm of head over the grate, 65 mm below the carpark kerb); the tank, covers, joints and vents are designed for a water level of RL 31.25

S5 Roof or podium downpipes blocked

Roof and podium emergency overflow spouts (building hydraulics, AS/NZS 3500.3) discharge onto the side corridors, which run to the driveway independently of the piped system

S6 Roller shutter closed

The surface route lies wholly outside the building line (y < 1.62); the covered carpark receives no direct rainfall and no design flow enters it, so the shutter position does not affect the route. Rear water is held below the carpark kerb (RL 31.25).

S7 South Street backwater

Free surface discharge assumes the road water level at the crossover stays below the boundary RL 30.90. If the road floods higher the site ponds against the frontage up to that level; the carpark (RL 31.00) remains above until the road reaches RL 31.00. Council road flood levels are requested for verification at operational works.

SURFACE HYDRAULICS (Manning normal depth; QUDM Table 7.3.6 limits 0.4 m, 0.6 m²/s)

Section	Flow	Width	n	Slope	Depth	Velocity	d-v
East corridor, whole-site 1 % flow	74 L/s	1.5 m	0.035	0.45 %	118 mm	0.42 m/s	0.05
West corridor, west catchments 1 %	36 L/s	1.5 m	0.035	0.45 %	75 mm	0.32 m/s	0.02
Front lined cross-drain, whole-site	74 L/s	1.0 m	0.015	0.64 %	82 mm	0.90 m/s	0.07
Driveway to crossover, whole-site 1	74 L/s	6.0 m	0.016	6.17 %	14 mm	0.89 m/s	0.01
Crossover to kerb, whole-site 1 %	74 L/s	6.0 m	0.016	10.00 %	12 mm	1.03 m/s	0.01
East corridor, every-pattern maximum	90.5 L/s	1.5 m	0.035	0.45 %	134 mm	0.45 m/s	0.06

All sections within the QUDM major-storm limits: True. Containment: east boundary wall top ≥ RL 31.48; rear kerb RL 31.25 keeps water out of the carpark (84 mm margin); route lies outside the building line so the roller shutter is not on the route.

Levels in metres on the project mapping datum; local grid x east from the west boundary, y south from the South Street boundary. Concept design; not for construction. Survey, structural and product design follow at operational works.

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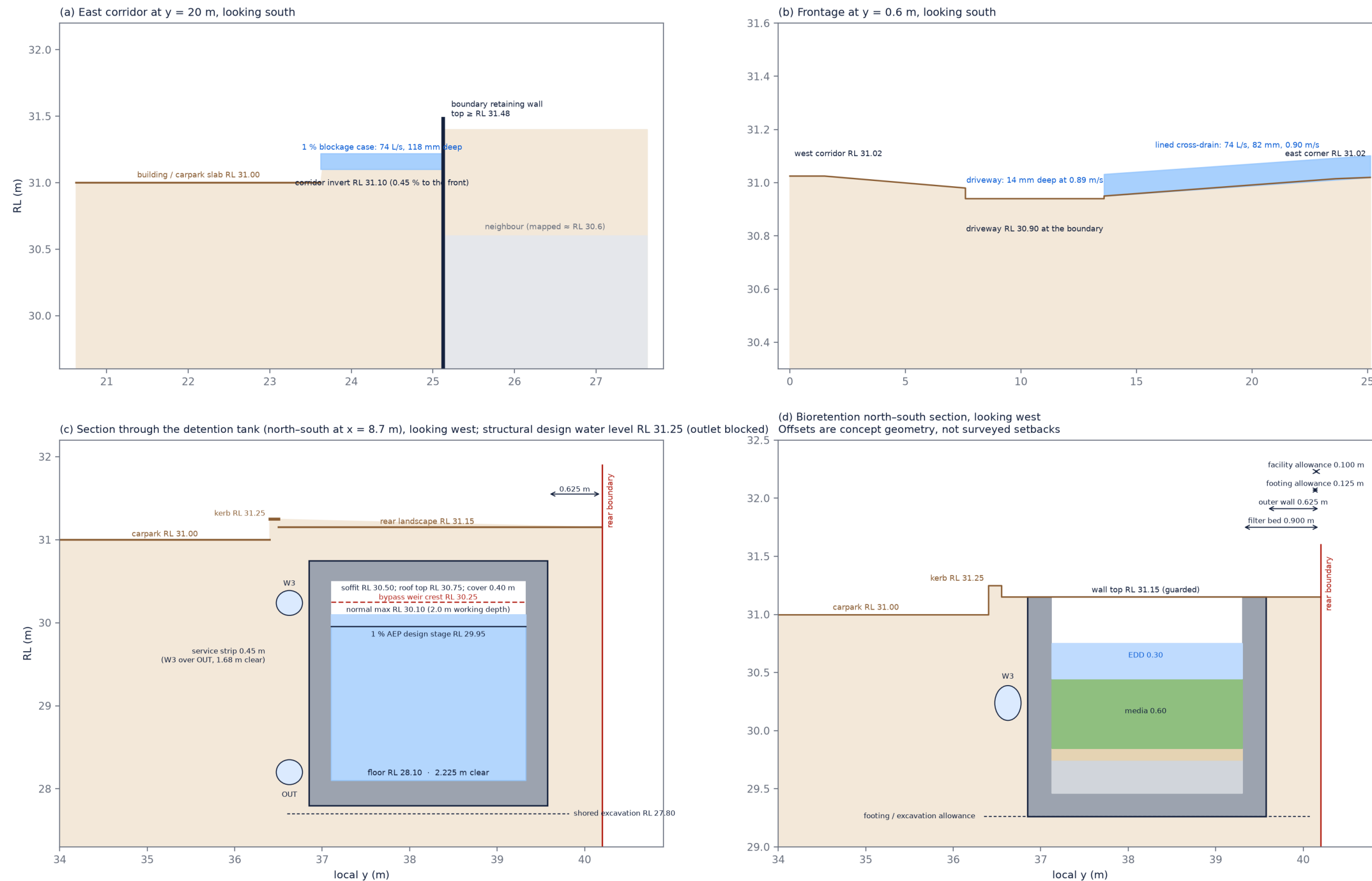
Figure 12: SW-Q07 Site cross sections: corridor containment, frontage, detention tank and bioretention

SW-Q07 Site cross sections — corridor containment, frontage, detention tank and bioretention

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East corridor (y = 20), frontage (y = 0.6), rear-strip tank and bioretention; 1 % AEP blockage and routed design water levels

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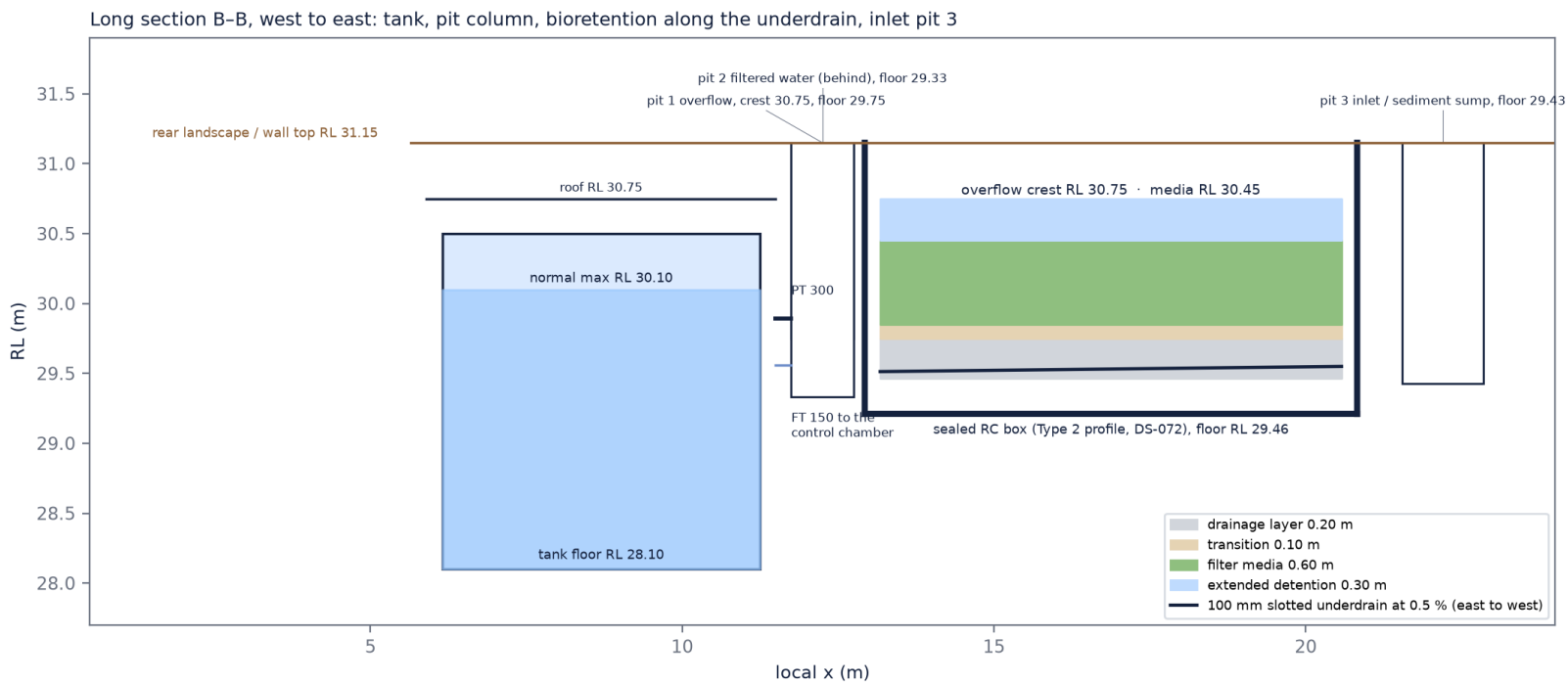
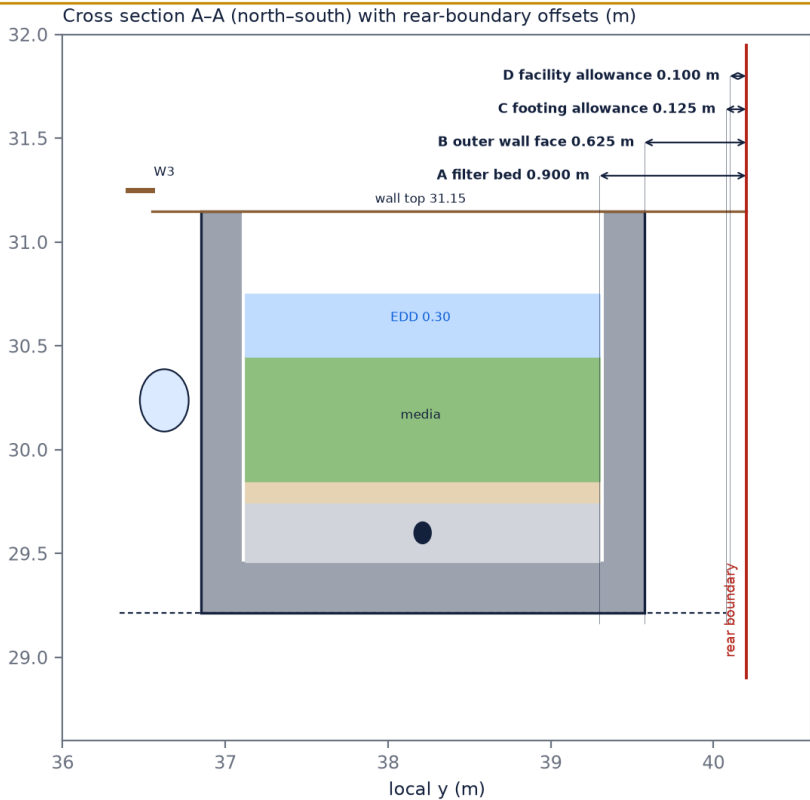
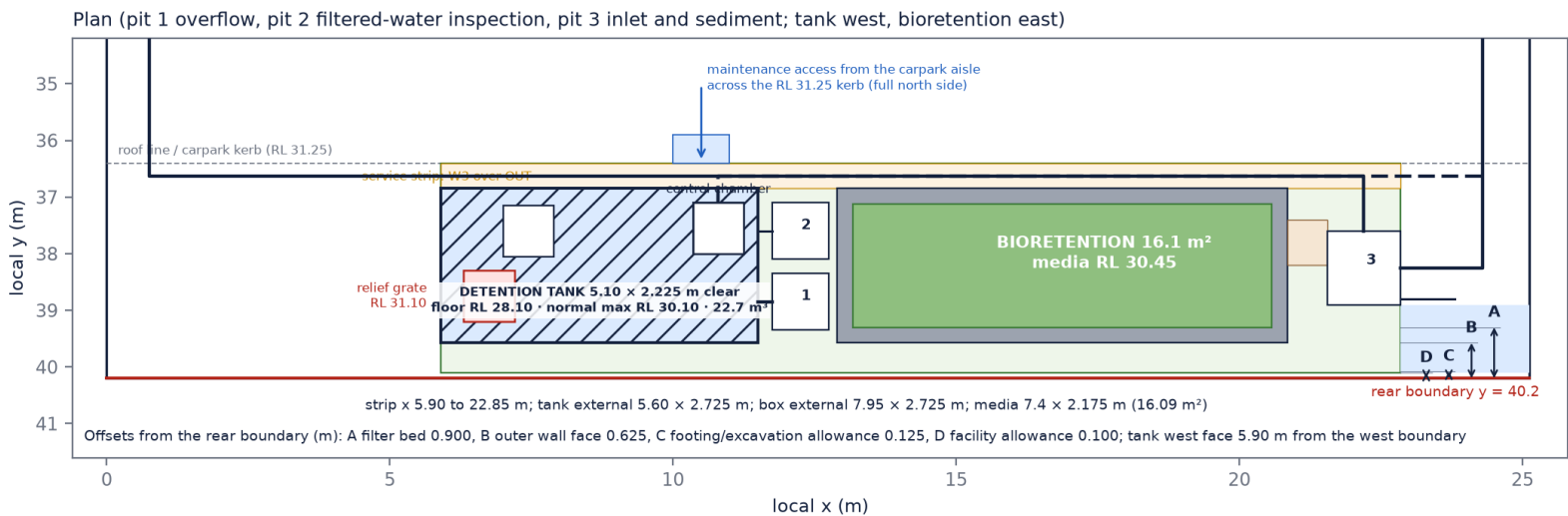
Figure 13: SW-Q08 Rear strip: bioretention and detention tank, plan, long section and cross section with rear-boundary offsets

SW-Q08 Rear strip — bioretention and detention tank

Plan, long section and cross section with rear-boundary offsets; media RL 30.45, overflow crest RL 30.75, wall top RL 31.15

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DESIGN NOTES

- Media level fixed by the gravity arrival of the front ground inlet GP1 (grate RL 30.93); pit 3 runs submerged at the pond level; 1 % HGL at GP1 RL 30.87.
- Set-down of 0.70 m from the surround exceeds the Water by Design 2014 Table 8 guide of 0.5 m for the reason in note 1; the box is a sealed RC structure with a guarded wall.
- Filtration capacity 2.01 L/s at Ksat 300 mm/h; 100 mm underdrain capacity 3.65 L/s; the filtered transfer enters the control chamber downstream of the plate at RL 29.48, so the underdrain is never drowned by the tank.
- Overflow pit 1 (600 x 600, crest RL 30.75, 50 % blocked) passes the 1 % site flow with 111 mm head and 0.29 m freeboard to the wall top; the 300 mm transfer enters the tank below its normal maximum with 0.76 m of driving head (172 L/s capacity).
- Gross pollutants: 5 mm baskets in GP1-GP4, leaf guards on gutters and downpipes, 600 mm sediment sump in pit 3 with the distribution apron; no in-line unit is placed at the submerged pit 3.
- Rear-boundary offsets are concept geometry on the architectural grid; the tank and box are built in a shored excavation to RL 27.80 and a boundary survey and the structural design confirm constructability wholly within the site.
- Planting 128 plants (8/m²) Carex appressa and Ficinia nodosa; maintenance from the carpark aisle across the kerb along the full north side and from the tank roof at the west end (50 % of the perimeter).

Levels in metres on the project mapping datum; local grid x east from the west boundary, y south from the South Street boundary. Concept design; not for construction. Survey, structural and product design follow at operational works.

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APPENDIX F: WATER SUPPLY HYDRAULIC ASSESSMENT

F1 Purpose, reliance and adopted conclusion

This appendix strengthens the existing SWMP with the additional water-supply network assessment. It retains assumed service at published/historical benchmarks and compares it with independent EPANET predictions from the supplied current and ultimate models. Predictions are conditional, not a utility-issued pressure report or field test. All supporting calculations and the concept drawing are contained in this SWMP. [R1; C01–C03; M1–M7]

Adopt the independent EPANET current/ultimate predictions as the conditional network basis for concept assessment, with the retained assumed-service method as a transparent comparison. Under the shared service assumptions, provide domestic pressure boosting, a fire pump and a brigade booster assembly. The governing tested ultimate differences are 66.3 kPa at 3 L/s domestic and 729.2 kPa at 30 L/s fire (with 2 L/s simultaneous domestic) under varying maximum-day background. These are calculated pressure differences, not selected equipment duties. Neither method establishes reliable indefinite ultimate supply or justifies deleting the storage contingency. Confirm datum, network operating basis, actual losses, permitted pumping and assured refill/storage during detailed design, before construction. [C01, C02; M2, M3, M6]

Table 25: Common service criteria and independently checked arithmetic

Input / result	Domestic C01	Fire C02
Development flow	3 L/s diversified peak	30 L/s fire; model also includes 2 L/s domestic
Hydraulic bore / route	50 mm / 60 m	150 mm / 80 m
Hazen–Williams C; rise; device loss	130; 12 m; 5 m	130; 12 m; 5 m
Remote outlet target	100 kPa	700 kPa
Pipe friction / velocity	3.5887 m / 1.5279 m/s	1.6155 m / 1.6977 m/s
Required inlet pressure	301.9062 kPa	882.5553 kPa
Assumed available pressure	22 m = 215.7463 kPa	12 m = 117.6798 kPa
Calculated difference	86.1599 kPa	764.8755 kPa

Calculation: $hf = 10.67 L Q^{1.852} / (C^{1.852} D^{4.87})$, Q in m³/s; required pressure = outlet target + $9.80665 \times (12 + hf + 5)$. Fittings are in the separate device allowance, not counted again as equivalent length. The model fire service loss uses the 30 L/s fire branch; the common main withdraws 32 L/s. [C01–C03; M3]

F2 Criteria, connection and source scenarios

Netserv Plan Part A (September 2017), Table 12-1, printed page 29, supplies the retained 230 L/EP/day, 22 m minimum operating pressure, 55 m maximum pressure and 30 L/s for four hours for residential buildings over three storeys, with two-thirds peak-hour background. These are network planning benchmarks, not measured site performance. The 12 m fire residual is a historical assumed comparator from Council preliminary assessment MC011817, document A1702640 (2009), pages 1–2; it is not stated as a fire residual in Netserv Table 12-1 and is not a current guarantee at this site. [R7, R10; C02]

Frontage asset 55284 in DMA204 is nominal DN250 uPVC; the model hydraulic bore is 258.8 mm. The site connection node (SITE) splits the main between model junctions J8851 and J8847 into 119.7712308 m and 52.7067202 m, preserving total hydraulic length 172.4779510 m, bore and roughness. The projected map position is not an approved tapping or survey. Pressure = (model HGL – concept RL30.90) × 9.80665 kPa/m assumes a common vertical datum. Endpoint elevations are not interpolated as site pressure. The level comes from the retained driveway-boundary concept level; datum correspondence remains unverified. [M1; M3]

Table 26: Council network scenarios used in the independent assessment

Scenario	Network, demand and controls	Reliance
2026IDM_MD	Current (2026) network, demand and controls as supplied	Council base model with the scenario's network, demand and control changes applied as supplied. Current conditions.
ULTIDM_MD	Ultimate network, demand and controls as supplied	Council base model with the scenario's network, demand and control changes applied as supplied. Ultimate augmentations and planned demand; not proof that works exist.

The demands generated by the supplied model are used as supplied; demand allocation is not applied a second time. Reservoirs keep their fixed heads, tanks their supplied geometry and starting levels, and pumps their curves, settings and operating status. The model's pump and valve control logic is reproduced with its rate limits, bounds, deadband and closed-pump protection. Ultimate network changes are those in Council's ultimate scenario; this assessment adds no utility augmentation of its own. The scenario set-up is recorded in M1 and the model inputs and control logic in M7.

The source is Council network data: the 2021 LGIP water distribution model supplied by Council. Between the current and ultimate scenarios the supplied data differ in 212 pipe attributes, not 212 separate upgrades: status, bore, roughness, length and description changes. Examples include pipe 211855A length 148.0399 to 55.0204 m, pipe 239531 bore 104.4 to 96.5 mm, and pipe 1479A roughness coefficient 100 to 110. These are supplied current-to-ultimate assumptions, not proposed works or evidence of built infrastructure. [M1]

F3 Demand reconciliation and comparison

The average-day screen is 15 dwellings × 3 persons × 230 L/EP/day = 10.35 kL/day (0.1197917 L/s). The 3 L/s peak is a separate preliminary demand; it is not a daily volume or a peaking derivation from that average. [C01–C03]

Table 27: Development replacement and operating cases

Case	Demand and background applied
Source parcel	Parcel 8028026 at J8847: current 2.7 L/s base and ultimate 12.9 L/s base with source residential pattern. Subtract the source parcel allowance before adding the complete proposal at SITE. Preserve other loads and NRW.
No development / proposal	No-development retains the existing 2.7 L/s parcel allowance in each scenario-specific background. Proposal uses the new average-day loading with the supplied residential demand pattern normalised for the proposal only. This is replacement, not addition of another full parcel.
flow-3 / Q-H curve	Constant prescribed SITE flow, with varying maximum-day source patterns over 24 h. flow-3 is the 3 L/s peak-pressure screen; Q-H points are daily minima at each prescribed withdrawal, not a simultaneous static curve.
fire-event24	30 L/s fire plus 2 L/s domestic, elapsed hours 9–13; residential/commercial background held at two-thirds source pattern peak; NRW unscaled. After hour 13, 2 L/s domestic remains.
md-event24	Same fire window and site demand; varying maximum-day background retained. This sensitivity gives the lower tested fire pressure and governs the concept comparison.
Extended md/fire cases	72 h requested; fire at hours 33–37. Paired no-fire case retains 2 L/s site domestic. Ultimate depletion in both backgrounds prevents an indefinite-supply claim.

Table 28: Proposal versus existing site — paired local impact

Scenario	Case	SITE head change from reference
2026IDM_MD	proposal	-0.0500 to -0.0003 m
2026IDM_MD	flow-3	-0.7066 to -0.0901 m
ULTIDM_MD	proposal	-0.0537 to -0.0003 m
ULTIDM_MD	flow-3	-0.7592 to -0.1032 m

Source: M3, paired comparisons of the proposal and flow-3 cases against the existing-site reference in each scenario. These changes are paired event/sample comparisons, not the difference between two independent minima. Shared source/demand assumptions mean agreement between methods is not independent field confirmation. [M3, M5]

F4 Governing supply and pressure differences

Table 29: Retained model minima and corresponding service pressure shortfalls

Scenario / case	HGL (m)	Time (h:m:s)	Pressure* (kPa)	Required / difference (kPa)
Current / flow-3	58.0172	9:42:00	265.9	301.9 / 36.0
Current / fire-event24	60.0585	10:03:00	285.9	882.6 / 596.6
Current / md-event24	50.1393	9:59:30	188.7	882.6 / 693.9
Ultimate / flow-3	54.9204	9:54:39	235.6	301.9 / 66.3
Ultimate / fire-event24	58.1439	9:35:00	267.2	882.6 / 615.4
Ultimate / md-event24	46.5406	9:10:30	153.4	882.6 / 729.2

*Conditional on connection RL30.90 and common datum. Time is elapsed simulation time, not a dated field observation. Domestic minimum covers the whole day; fire minimum covers the prescribed fire window. Sources: M2 design comparison; M3 case minima and their times at the site connection. Tighter final runs reproduce these values to the reported precision [M6].

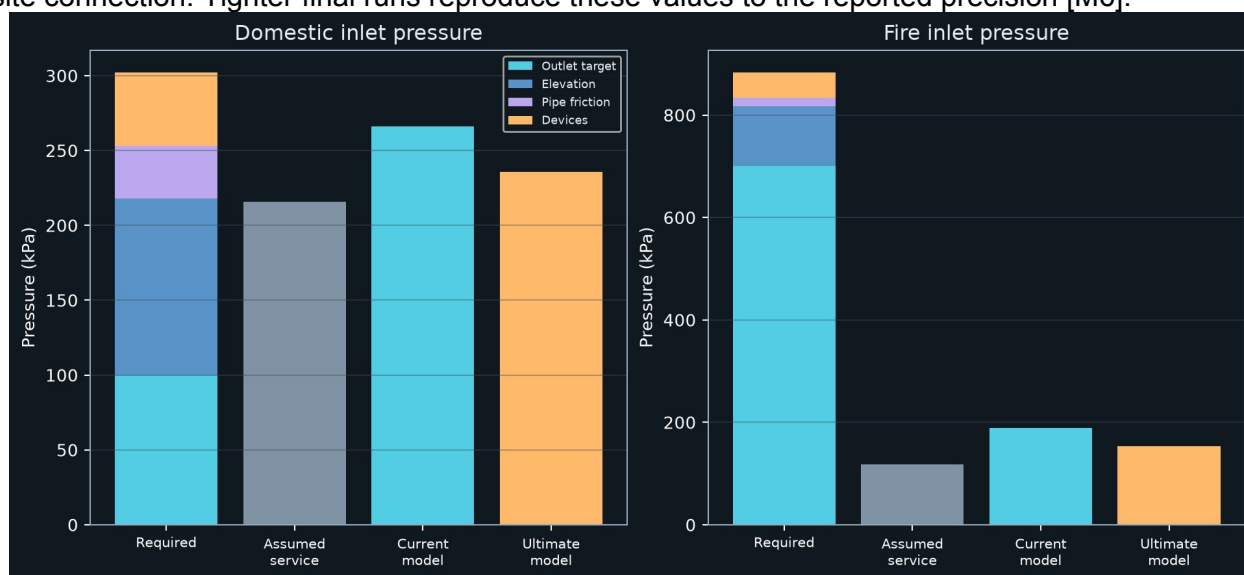


Figure 14: Required inlet budgets and assumed/modelled available pressure

Sources: C01 and C02; M2 design comparison. The fire model bars use the varying maximum-day event. Outlet target, elevation and internal loss inputs are shared across both methods. The model predicts more available pressure than the older assumed-service comparison; no averaging is applied.

F5 Pressure–flow behaviour and numerical acceptance

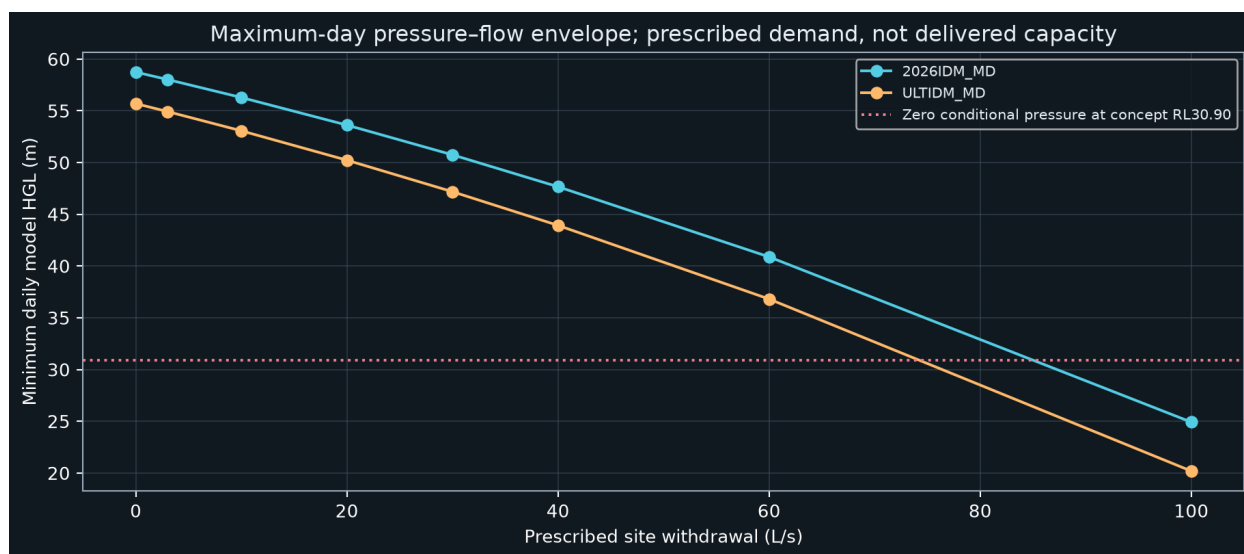


Figure 15: Daily minimum HGL envelope from retained prescribed-flow cases

Sources: M2 pressure–flow envelope and M3 prescribed-flow cases at 0, 3, 10, 20, 30, 40, 60 and 100 L/s. At high prescribed flows the model permits negative pressure; such points are infeasibility diagnostics, not physically delivered capacity. The zero-flow point has ongoing background demand and is not a field static-pressure test.

Final numerical review uses 60 s hydraulic steps and relative accuracy 0.00001 for governing peak/fire cases. Mass residual is within 0.001 L/s, with no unexplained storage residuals; control behaviour follows the supplied model's control rules. Local DMA204 head differences remain below 0.000005 m against the predeclared 0.1 m review trigger. Model inputs were confirmed unchanged between runs and the original daily peak and fire runs reproduce exactly. These are numerical implementation checks, not calibration, native MIKE identity or utility-wide operating-history validation. [M6]

Every +1 m actual connection elevation reduces the reported pressure by 9.80665 kPa without changing model HGL. No defensible numerical error bound is assigned to an unknown datum. Device loss is held at 5 m for comparison; final manufacturer curves must vary with flow. Changing the remote fire target from 700 to 350 kPa reduces the required inlet by 350 kPa, but only the fire designer may approve that criterion. [C03; M2]

Table 30: Retained fire criterion sensitivity — assumed residual held at 117.7 kPa

Fire flow / outlet	Required inlet	Calculated difference
10 L/s / 700 kPa	868.8 kPa	751.1 kPa
20 L/s / 700 kPa	874.2 kPa	756.5 kPa
30 L/s / 700 kPa	882.6 kPa	764.9 kPa
30 L/s / 350 kPa	532.6 kPa	414.9 kPa

Source: C03. This held-pressure sensitivity is not an interpolated network supply curve and is not an alternative approved duty.

F6 Fire duration, daily recovery and adverse findings

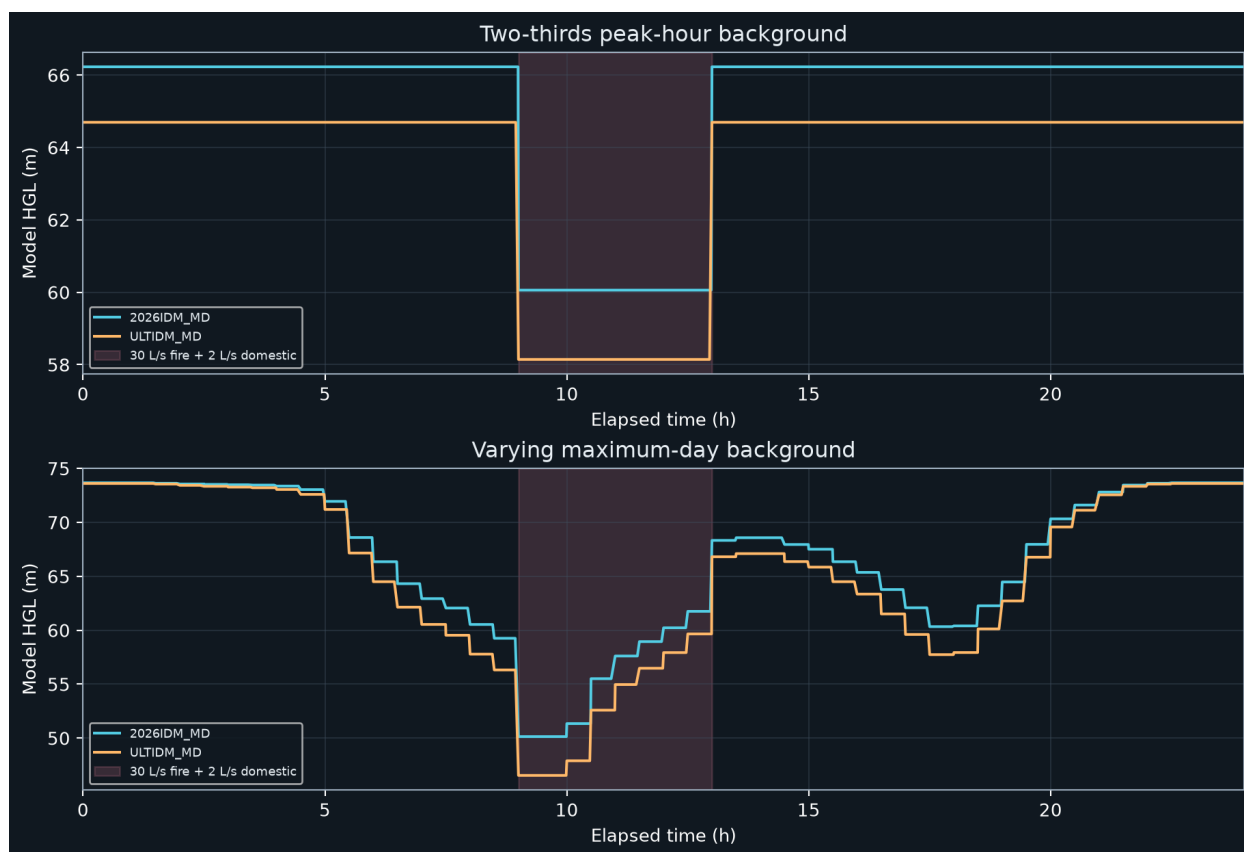


Figure 16: Daily fire event and post-event response in both backgrounds

Source: M4, scenarios 2026IDM_MD and ULTIDM_MD, fire-event24 and md-event24; hydraulic grade line at the site connection over time. All four daily cases complete the prescribed four-hour event and the subsequent eleven-hour observation. Completion alone does not prove physical flow availability, full storage recovery or an indefinitely repeatable supply. In the retained paired comparisons, all tank volumes do not return to within 1 m³ of the no-fire reference during the observation. [M3]

Pressure-dependent sensitivity uses a global assumed 22 m required pressure and exponent 0.5, including fire demand. Curtailed fire withdrawal cannot be described as delivery of 30 L/s. Demand without a connected source, negative pressures at sources, operating warnings and a failed start with all tanks at minimum level are retained adverse findings. Fixed-head boundary draw is not verified source production capacity. [M2; M6]

F7 Extended storage and coordinated design

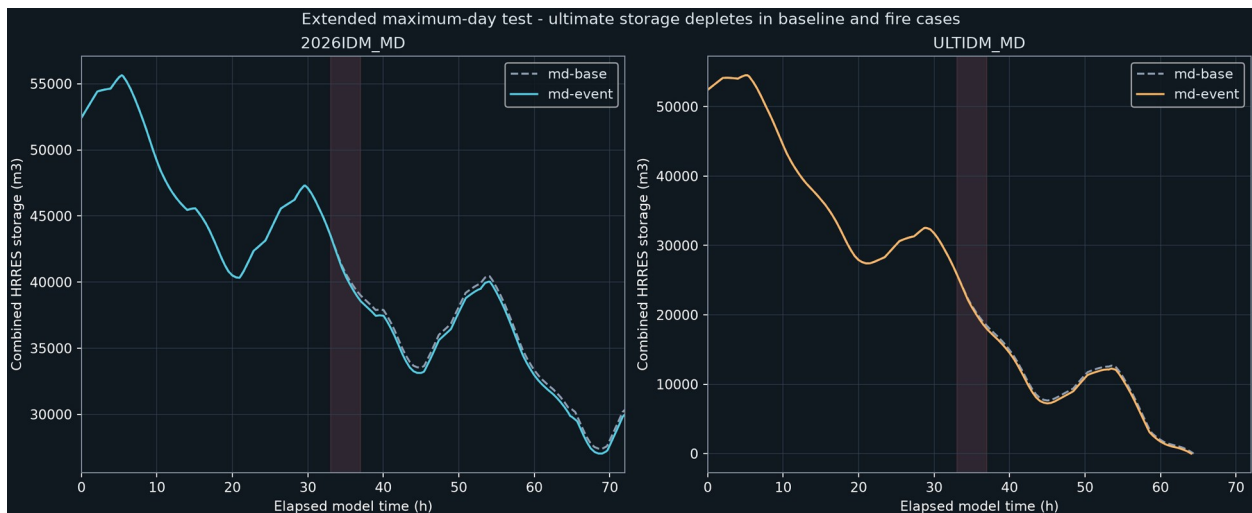


Figure 17: Extended maximum-day storage: paired baseline and fire event

Source: M4, cases md-base and md-event, combined volume of reservoirs HRRES1–HRRES3; requested 72 h, fire hours 33–37. The current cases complete. Ultimate source storage depletes and the simulations stop, including the no-fire baseline; do not assign all depletion to this development or present failed tail results as design pressure. Source settings do not establish sustainable ultimate supply. [M3; M6]

A pump adds head, not water volume. Resolve permissible direct pumping, source reliability and assured refill. If the verified supply cannot sustain the confirmed duty, provide an approved break/suction tank and refill arrangement or an accepted network solution. Size usable storage from the maximum cumulative demand-minus-assured-refill deficit over the required event/recovery, with required reserves, unusable volume and reliability allowance. No refill has been guaranteed here. At zero refill, $30 \text{ L/s} \times 4 \text{ h} = 432 \text{ kL}$ fire volume; $2 \text{ L/s} \times 4 \text{ h} = 28.8 \text{ kL}$ separate domestic withdrawal. These arithmetic bounds are not a tank selection. [C02; M3] Reserve the domestic boost, fire pump and brigade booster concept at the frontage; WS-01 shows symbolic reservations, not certified footprints. Architect, landscape and fire/hydraulic designer must resolve storage contingency, clearances, access, noise, power and maintenance before layout freeze. Existing rear stormwater detention/treatment is not fire-water storage; no stormwater volume or drawing is changed by this assessment.

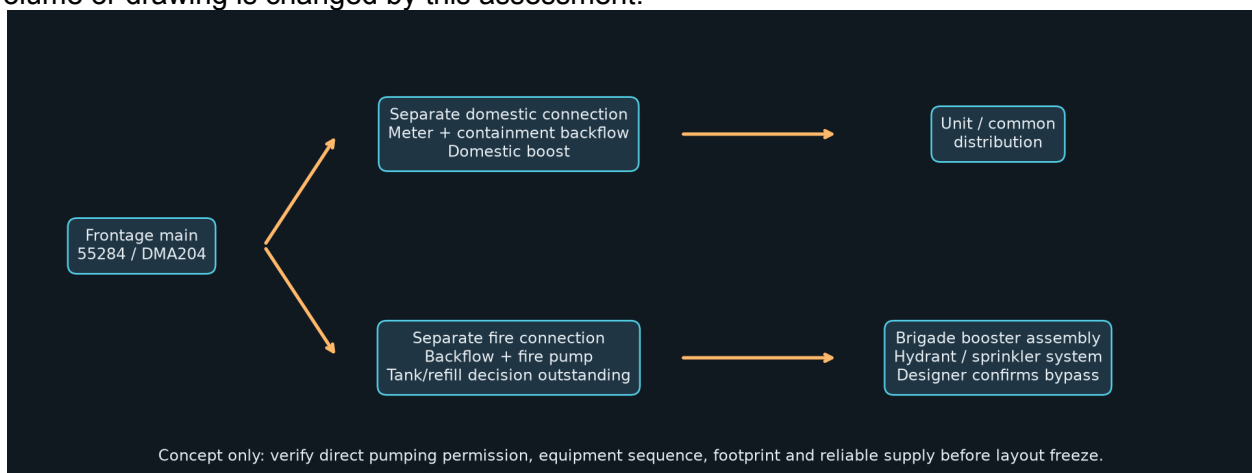


Figure 18: Coordinated separate-service concept and unresolved storage decision

F8 Calculation records and engineering handover

Table 31: Calculation records

Reference	Record	Content
C01–C03	Assumed-service calculation	Domestic (C01) and fire (C02) inlet-pressure budgets, fire-criterion sensitivity (C03) and the average-day demand screen.
M1	Scenario set-up and site connection	Council current and ultimate scenarios as supplied; site connection on the frontage main; pipe-attribute differences between the scenarios.
M2	Design comparison and pressure–flow envelope	Common inlet-pressure comparison, daily minimum HGL envelope, connection-level and pressure-dependent sensitivities and source-capacity checks.
M3	Scenario cases and comparisons	Domestic, fire and extended cases; minima and times at the site connection; paired proposal and fire comparisons.
M4	Hydraulic event records	Time series plotted in Figures 16 and 17: hydraulic grade line at the site and reservoir volumes.
M5	Demand reconciliation	Replacement of the existing parcel allowance by the proposal demand at the site connection.
M6	Final numerical and control review	Time-step and accuracy checks, mass balance, control behaviour and the retained adverse findings.
M7	Reproduction record	Solver version, scenario inputs, connection mapping and control logic needed to reproduce the results.

These records are retained by RPEQ Consulting with the calculation package and are available to Council on request. The published references R7 and R10 are listed in F9.

The network results were computed with the EPANET hydraulic solver. The supplied model's pump and valve control logic was applied through an external feedback controller, so a standalone EPANET input file does not reproduce the results on its own. This report transfers the accepted outputs; it does not re-run the solver or re-benchmark the model against Council's native modelling software. [M7]

This assessment was issued for the Council RFI response. At detailed design, Council confirms the connection, scenario currency, datum, permissible pumping and reliable supply/refill. The hydraulic/fire designer and certifier confirm demand, code edition, losses, final pump/storage design and pressure ratings. The architect and landscape designer coordinate actual equipment and access. These detailed-design requirements remain; this document is not authority for procurement or construction.

F9 Water-assessment references

R1 — Redland City Council Information Request MCU26/0056, 9 June 2026, PDF page 2: stormwater Items 2–3 and water Item 4.

R2 — Interworks Architects submitted plans, drawing 2.1 Issue A, 30 October 2025; development application MCU26/0056, Appendix A Proposed Plans, PDF page 2. Adopted base for WS-01.

R3 — Interworks Architects Issue C, 15 June 2026, drawing 2.1. Retained frontage comparison; does not silently replace the submitted base.

R4 — Council public Asset Mapping: Water Pressure Pipes, Hydrants, Valves and District Metered Areas; queried 22 September 2026. Frontage asset 55284 / DMA204; cadastral/model alignment is indicative, not a survey.

R5 — Council network data: Council-supplied current and ultimate water-network models; independent extraction, inputs and assessment recorded as M1–M7 in F8. Conditional project results, not Council-issued site pressures.

R6 — Redland City Plan Version 14, updated 8 April 2026, Infrastructure Works Code, printed page 393 / PDF page 397. Confirm planning applicability.

R7 — Redland Water Netserv Plan Part A, September 2017, Table 12-1, printed page 29. Retained historical network planning benchmarks, not current site supply.

R8 — Redland City Council: Water and sewerage connections; Water meters; Information for hydraulic designers. Retained guidance retrieved 21 September 2026; confirm final connection requirements with Council. <https://www.redland.qld.gov.au/Water-supply-and-sewerage/Water-and-sewerage-connections>

R9 — NCC 2022 Volume One, Part E1, clauses E1D2 and E1D6; AS 2419.1-2021 fire hydrant installations. Retained concept interpretation; fire/hydraulic designer and certifier confirm governing edition and system duty.

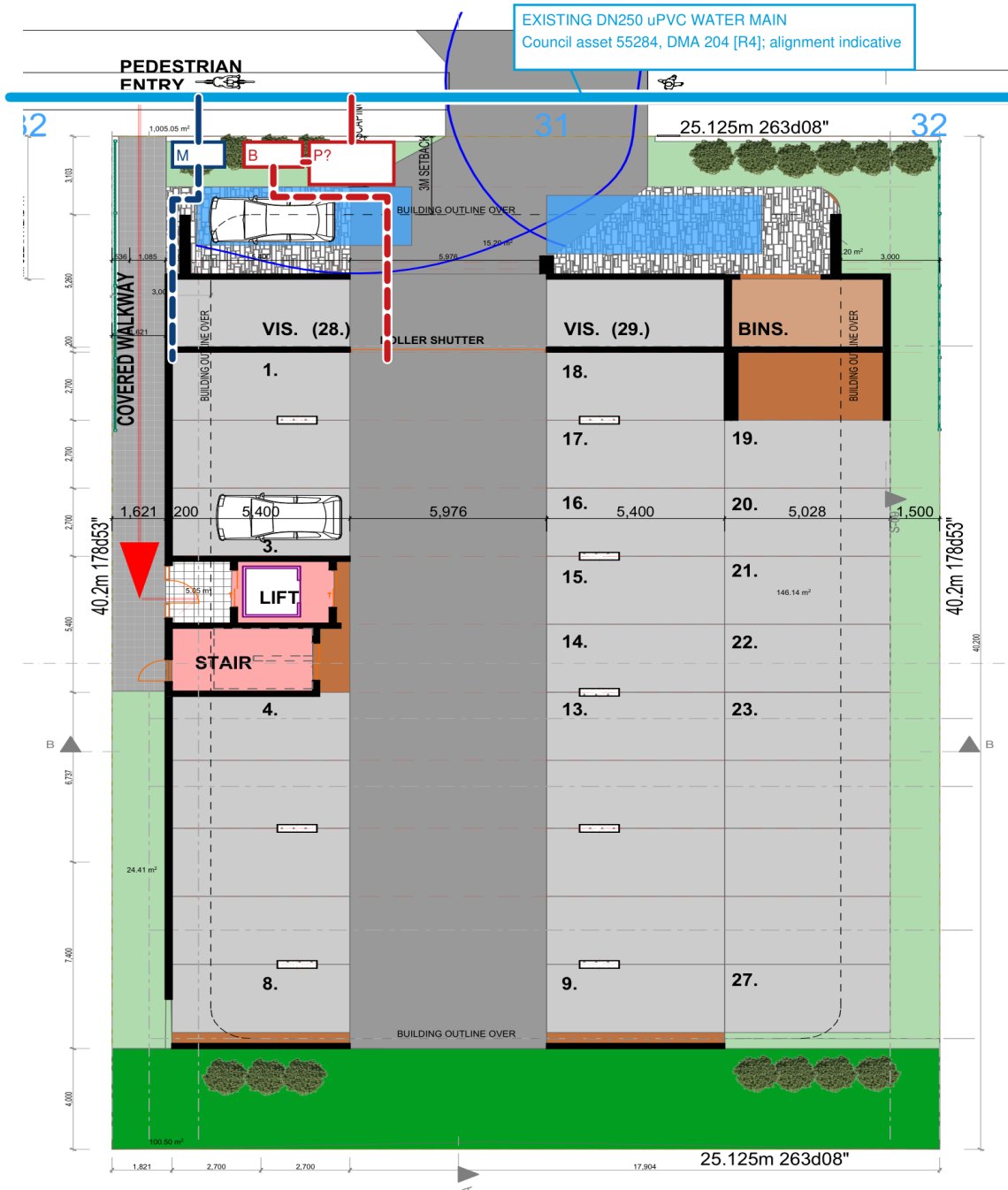
R10 — Historical other-site Council preliminary water assessment MC011817, document A1702640, pages 1–2. The 12 m residual is a comparator, not a Netserv Table 12-1 requirement or site guarantee; 2009 is the document's recorded creation date.

APPENDIX G CIVIL WATER CONCEPT — WS-01

RPEQ CONSULTING | CIVIL WATER CONCEPT | WS-01

134 South Street, Thornlands | MCU26/0056 | Rev 5 - Issued for RFI Response - 24 September 2026

SOUTH STREET (road reserve)



BLUE = DOMESTIC RED = FIRE DASHED = INDICATIVE ROUTE

Read with Stormwater Management Plan, Rev 5, sections 9 and Appendices F-G. Verify final sizes, levels, duties, storage and locations during detailed design. NOT FOR CONSTRUCTION.

CONCEPT DETAILS AND HOLD POINTS

1. M - Domestic DN50 master meter, containment backflow and separate boost arrangement are provisional. At 3 L/s the ultimate model gives 235.6 kPa against 301.9 kPa required: calculated difference 66.3 kPa. The older 22 m assumed-service screen gives 86.2 kPa. No pump selected.
2. B - Brigade booster assembly, provisional. Fire designer confirms code edition, hydrant/sprinkler duty, appliance access, separation/protection, bypass and brigade requirements. An assembly is not proof of automatic supply adequacy.
3. P? - Fire pump set. At 30 L/s fire plus 2 L/s domestic, ultimate varying maximum-day supply is 153.4 kPa: calculated fire boost difference 729.2 kPa. Two-thirds peak background gives 615.4 kPa. Not equipment selection; direct pumping requires approval.
4. Boxes reserve symbolic locations, not equipment footprints. Keep the pedestrian route and driveway clear. Coordinate pump enclosure, working space, noise, ventilation, power/fuel and any storage with architect and landscape design before fixing the layout.
5. Nominal domestic DN50 and fire DN150; separate takeoffs. Main asset 55284 is nominal DN250; model bore 258.8 mm. The model SITE split is map-derived, not a surveyed or approved tapping. Dashed routes require detailed design.
6. No-storage adequacy is NOT established. Daily four-hour fire simulations complete, but ultimate extended runs deplete HRRES storage; paired baseline also depletes. Resolve assured refill, usable storage or an accepted network solution before issue of the detailed design for construction.
7. Pressure values assume connection RL30.90 and common project/model datum, both unverified. Verify survey, main location/depth/material and tapping. HGL remains the primary model result. No construction setout or clearance certification.
8. Base: submitted architecture Issue A, drawing 2.1, 30 October 2025 [R2]; frontage comparison with Issue C, 15 June 2026 [R3]. Read SWMP Appendix F, shared calculation C01-C03 and model evidence M1-M7. Do not scale.

SERVICE PRINCIPLE - DIAGRAMMATIC

Main -> separate domestic takeoff -> isolation / backflow / meter (M) -> domestic distribution
Main -> separate fire takeoff -> isolation / backflow -> fire pump set (P?) with bypass -> brigade booster assembly (B) -> fire distribution

Device order and brigade connection bypass are to be confirmed by the fire designer and Council. This schematic is not an installation detail.