

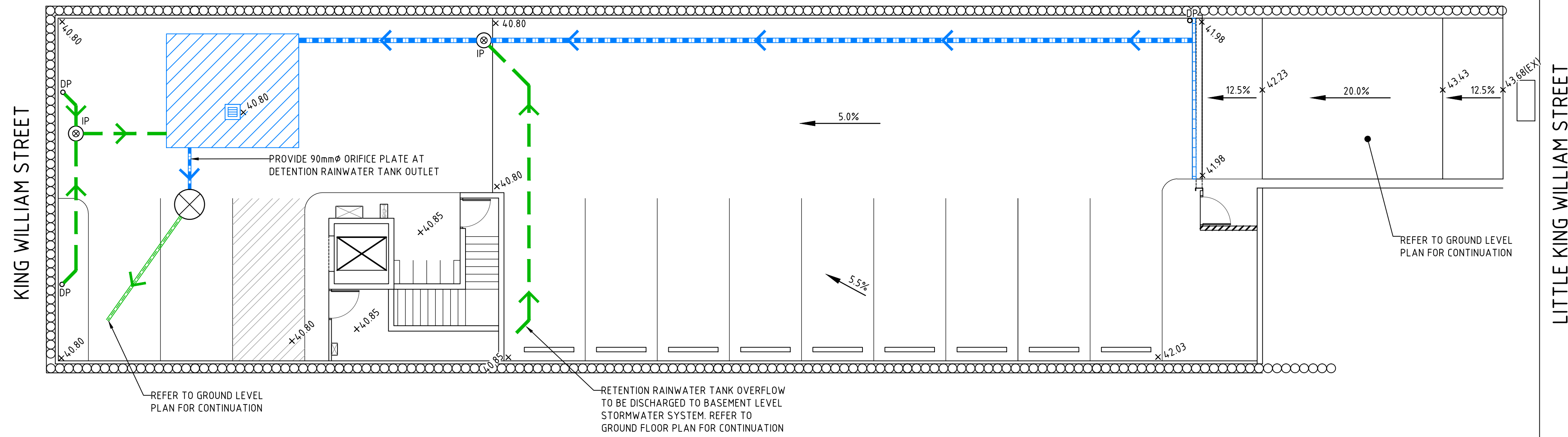
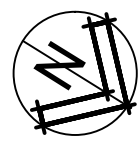


PRELIMINARY CIVIL STORMWATER CALCULATIONS

97 King William Street, Kent Town

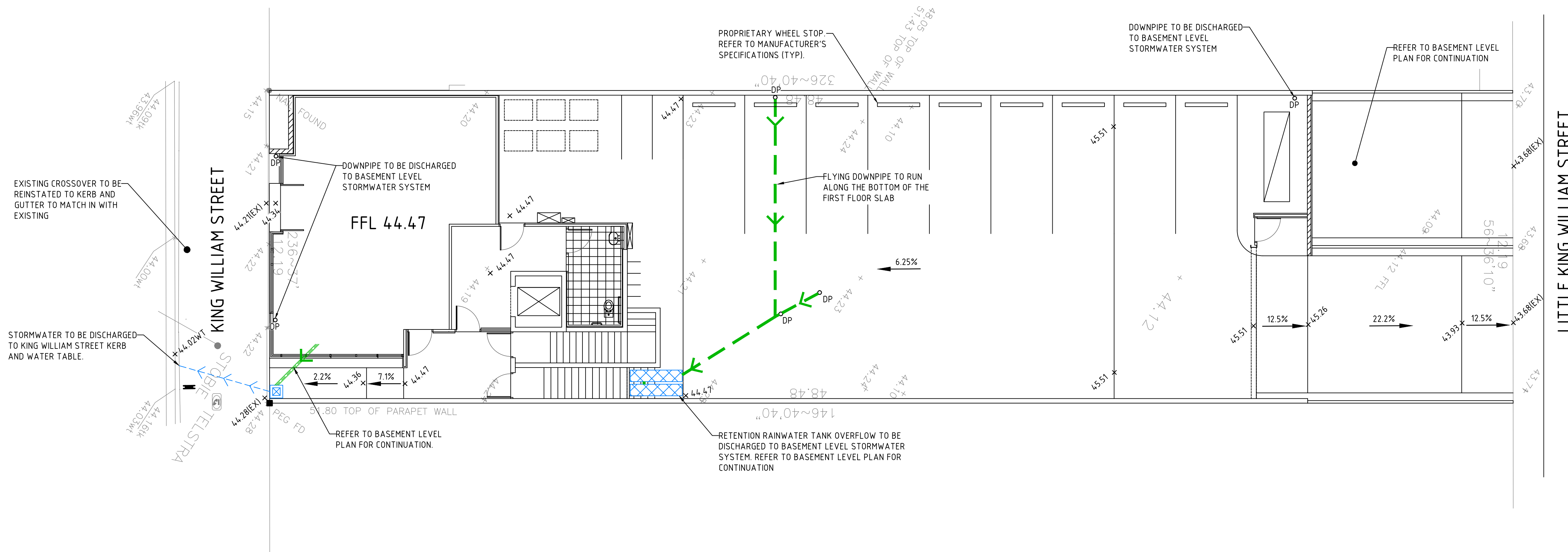
ISSUE C

DATE: 30/07/2018



BASEMENT LEVEL PLAN

SCALE 1:100



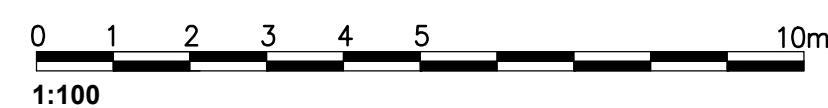
GROUND LEVEL PLAN

SCALE 1:100

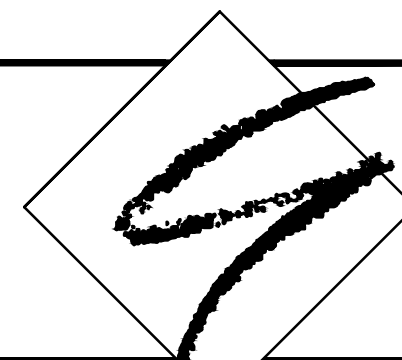
LEGEND

- XXXXX + DESIGN SPOT LEVEL
- X-XXX(X)X + EXISTING SPOT LEVEL
- 300mm SQUARE GRATED INLET PIT
- 300mm SQUARE JUNCTION BOX
- 150mm UPVC STORMWATER PIPE, MINIMUM GRADE OF 0.5%
- 150mm UPVC SEALED STORMWATER PIPE
- 12.5% PROPOSED SURFACE GRADE AND DRAINAGE DIRECTION
- 100mm WIDE ACO KLASSIK DRAIN OR APPROVED EQUIVALENT
- 40mm HDPE RISING MAIN AS PER PUMP MANUFACTURER SPECIFICATIONS
- DUAL STORMWATER PUMP STATION WITH CLASS B LID OR APPROVED EQUIVALENT. MAXIMUM DISCHARGE RATE OF 12L/s
- 150mm UPVC DOWNPIPE (REFER TO ARCHITECTURAL DRAWINGS FOR DETAILS)
- INSPECTION POINT
- GRAVITY FED CHECKER PLATE DRAIN OUTLET IN ACCORDANCE WITH COUNCIL'S GUIDELINES
- 7,000L BELOW GROUND DRAINWELL MODULAR DETENTION RAINWATER TANK OR APPROVED EQUIVALENT
- 2x 1,050L ABOVE GROUND SUPER SLIMLINE POLY WATER RAINWATER TANK OR APPROVED EQUIVALENT

Issue	Date	Amendment
A1	04.05.18	ISSUED FOR PLANNING APPROVAL
A2	22.05.18	ISSUED FOR PLANNING APPROVAL
A3	26.07.18	ISSUED FOR PLANNING APPROVAL



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97 KING WILLIAM STREET, KENT TOWN, SA
FOR RASHDEQ

Drawn	SY	Scales 1:100 on A1
Design	HP	Drawing Number
Approved	NM	
Date	04.05.2018	2018-7161 C1



Strip drain capacity calculation

Reference 2018-7161

Project 97 King William St, Kent Town

Checked by SG

Designer H.P

Date 2018.04.10

Index 1

Refer '2018.04.10 - Post development Plan'

Refer 'BOM-IFD'

$$S_{min} \quad I = 182 \text{ mm/hr} \quad , \quad I = 26.4 \text{ mm/hr}$$

$$\begin{aligned} C'_{10} &= 0.1 + 0.0133 \left(\frac{1 \text{ hr}}{10 \text{ yr}} I - 25 \right) \\ &= 0.1 + 0.0133 (26.4 - 25) \\ &= 0.11862 \end{aligned}$$

Eq 14.12 ARR

Assume $S = 1.0$ (Impervious carpark surface)

$$\begin{aligned} C_{10} &= 0.95 + C'_{10}(1-S) \\ &= (0.9 \times 1.0) + 0.11862(1-1) \\ &= 0.9 \end{aligned}$$

Eq 14.11 ARR

$$\begin{aligned} C_{100} &= F_{100} C_{10} \quad \text{where } F_{100} = 1.2 \text{ (Table 14.6 ARR)} \\ &= 1.2 \times 0.9 \\ &= 1.08 \quad (\text{Let } C_{100} = 1.0) \end{aligned}$$

Eq 14.13 ARR

$$\therefore C_{100} = 1.0$$

Assume $t_c = 5 \text{ mins}$, $A = 45.19 \text{ m}^2$

$$\begin{aligned} Q_{100} &= 0.278 C_{100} \times \frac{5 \text{ mins}}{100} I \times A \\ &= 0.278 \times 1.0 \times 182 \times (45.19 \times 10^{-6}) \\ &= 2.29 \text{ L/s.} \end{aligned}$$

Eq 5.1 ARR

\therefore Peak rainfall down basement ramp is 2.29 L/s.

Strip drain capacity calculation:

Given downward ramp has a grade of 1:5 over 6 metres. \therefore Total upstream flow depth is $(1 \div 5) \times 6 = 1.2 \text{ m}$

Refer 'Strip drain flow capacity Calculation'
(Calculation page 3)

Reference 2018-7161Project 97 King William St, Kent TownChecked by GP.Designer H.P.Date 2018.04.0Index 2

Assuming a 100mm wide strip drain across the length of the ramp is used, the overall capacity is sufficient to capture the 100yr storm event.

Given the required upstream flow depth for a flow capacity of 2.34/s is 0.0065m. Given the upstream flow depth due to the access ramp is $\approx 1.7\text{m}$, \therefore sufficient capacity



Strip drain flow capacity Calculations

3

Using QUDM -

(i) Under weir flow conditions (Figure 7.05.4):

$$Q_g = BF \times 1.66 L h^{3/2} \quad (7.04)$$

where Q_g = flow into field inlet (m^3/s)
 BF = blockage factor = 0.5
 1.66 = weir coefficient
 L = weir length (m) (see note below)
 h = depth of water upstream of inlet (relative to weir crest) where flow velocity is low (i.e. velocity head is insignificant) otherwise use the height of energy level above the weir crest(m)

BF 0.5 (Clear opening)
 L 5.3 5.3m by 0.1m strip drain
 h 0.0065 m

Qg	0.002305282 m3/s	2.305282 L/s
----	------------------	--------------

Utilised weir perimeter length of 5.3m.

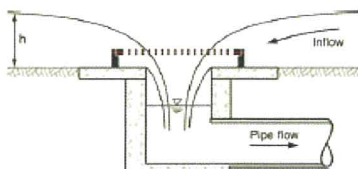
Equation 7.05 is based upon a pressure change coefficient of $K_g = 2.75$.

$$Q_g = BF \times 0.60 A_g (2gh)^{1/2} \quad (7.05)$$

where Q_g = flow into field inlet (m^3/s)
 BF = blockage factor = 0.5
 A_g = clear opening area of grate (m^2)
 h = depth of approaching water relative to the orifice (m)
 g = acceleration due to gravity ($9.79 m/s^2$)
 0.60 = constant = $(1/K_g)^{1/2} = (1/2.75)^{1/2}$
 K_g = pressure change coefficient for the grate

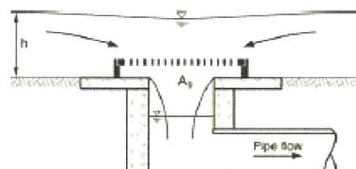
The pressure change coefficient (K_g) can vary significantly for unusual grate designs. The coefficient used in Equation 7.05 is based on a typical open mesh grate. It is noted that the pressure change coefficient for the old cast iron "City Grate" has been adopted as 2.23. Designers of unusual hydraulic structures should seek expert advice or review reference documents on orifice flow.

If the field inlet is fully drowned (i.e. no air gap exists below the grate and thus the hydraulic pressure below the grate is not atmospheric) then an estimate must be made of the head loss through the structures as per a normal Hydraulic Grade Line (HGL) analysis. Such calculations require considerable experience and hydraulic judgement. Guidance on head losses through screens is provided in Sections 7.16.14(c) and 12.04.6.



Field inlet operating under weir flow

Figure 7.05.4



Field inlet operating under free orifice flow

Figure 7.05.5

BF 0.5 (Clear opening)
 A_g 0.53 5.3m by 0.1m strip drain
 h 0.0065 m

Qg	0.056723125 m3/s	56.72312 L/s
----	------------------	--------------

Utilised weir opening area of $5.3 \times 0.1 = 0.53 m^2$



Box gutter capacity calculation

Reference 2018-7161

Project 97 King William St, Kent Town

Checked by CJG

Designer H.P

Date 04/05/2018

Index 4

Gutters to be designed for 100 yr ARI.

Roof considered to be 'two adjacent sloping roofs'

$$A_c = A_{h1} + A_{h2} + 0.5(A_{v2} - A_{v1})$$

Eq 3.4.3.3
AS 3500.3

Refer '2018.05.04 - Roof Catchment Plan'

$$A_{h1} = 247.82 \text{ m}^2$$

$$A_{h2} = 247.82 \text{ m}^2$$

$$A_{v1} = (0.02 \times 20.33) \times 12.19 \text{ m} \\ = 4.96 \text{ m}^2$$

$$A_{v2} = (0.02 \times 20.33) \times 12.19 \\ = 4.96 \text{ m}^2$$

$$A_c = 247.82 + 247.82 + 0.5(4.96 - 4.96) \\ = 495.64 \text{ m}^2$$

Assume a time of concentration, $t_c = 5 \text{ mins}$

Using BOM-IFD, $\frac{S_{min}}{100} I = 182 \text{ mm/hr}$

Given there are 3 downpipes proposed for the gutter system, \therefore Catchment area per downpipe is $A_c \div 3 = 495.64 \div 3 = 165.21 \text{ m}^2$

Using Figure I1, AS 3500.3, design $Q \approx 9.0 \text{ L/s}$

Proposed box gutter width = 600 mm

Proposed box gutter slope = 1:200

\therefore Min depth of box gutter, $h_a = 108 \text{ mm}$

Using Figure I4, AS 3500.3, assuming 150 mm ϕ downpipes

\therefore Depth of sump, $h_s = 70 \text{ mm}$

However as $h_s < 150 \text{ mm}$, \therefore Adopt $h_s = 150 \text{ mm}$

Clause 3.7.4 (e)
AS 3500.3

Using Figure I6, AS 3500.3, given $Q \approx 9.0 \text{ L/s}$ and $W_{bg} = 600 \text{ mm}$

$$\therefore L_{oc} = 28 \text{ mm}$$

Using Figure I8, AS 3500.3, given $Q \approx 9.0 \text{ L/s}$ and $W_{bg} = 600 \text{ mm}$

$$\therefore h_f = 84 \text{ mm}$$

Reference 2018-7161Project 97 King William St, Kent TownChecked by CJGDesigner H.PDate 04/05/2018Index 5

Check:

$$h_a > h_f + l_{oc}$$

$$108\text{mm} > 84 + 28$$

$$108\text{mm} > 112\text{mm}$$

$$\therefore \text{Adopt min } h_a = 112\text{mm}.$$

Check:

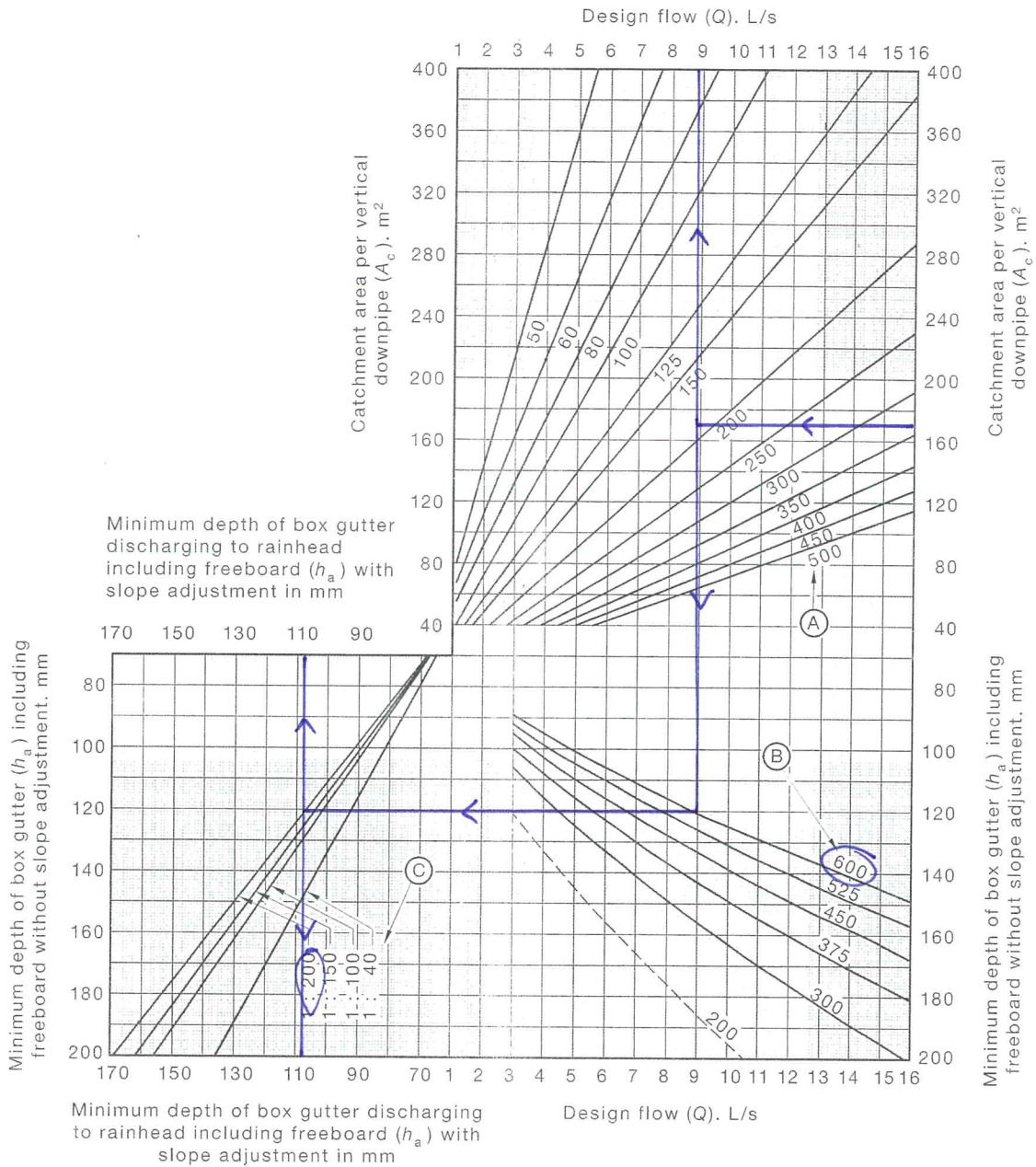
$$\text{Given } l_{oc} < 60\text{mm}$$

\therefore Depth of sump, h_s datum is measured from downstream sole of overflow channel $(60 - l_{oc})$ below sole of box gutter.

$$\therefore h_s = (60 - l_{oc}) + h_s \text{ of } 150\text{mm}$$

$$= (60 - 28) + 150$$

$$= 182\text{mm}$$



LEGEND:

- (A) = Design rainfall intensity ($^{100}I_5$) OR ($^{50}I_{10}$) in mm/h (typical)
- (B) = Width of box gutter (W_{bg}) in mm (typical)
- (C) = Gradient of box gutter (typical)

NOTE: Box gutters 200 mm wide may be used for domestic construction only. See Clause 3.7.1.

FIGURE 11 DESIGN GRAPH FOR A FREELY DISCHARGING BOX GUTTER

21/5/18

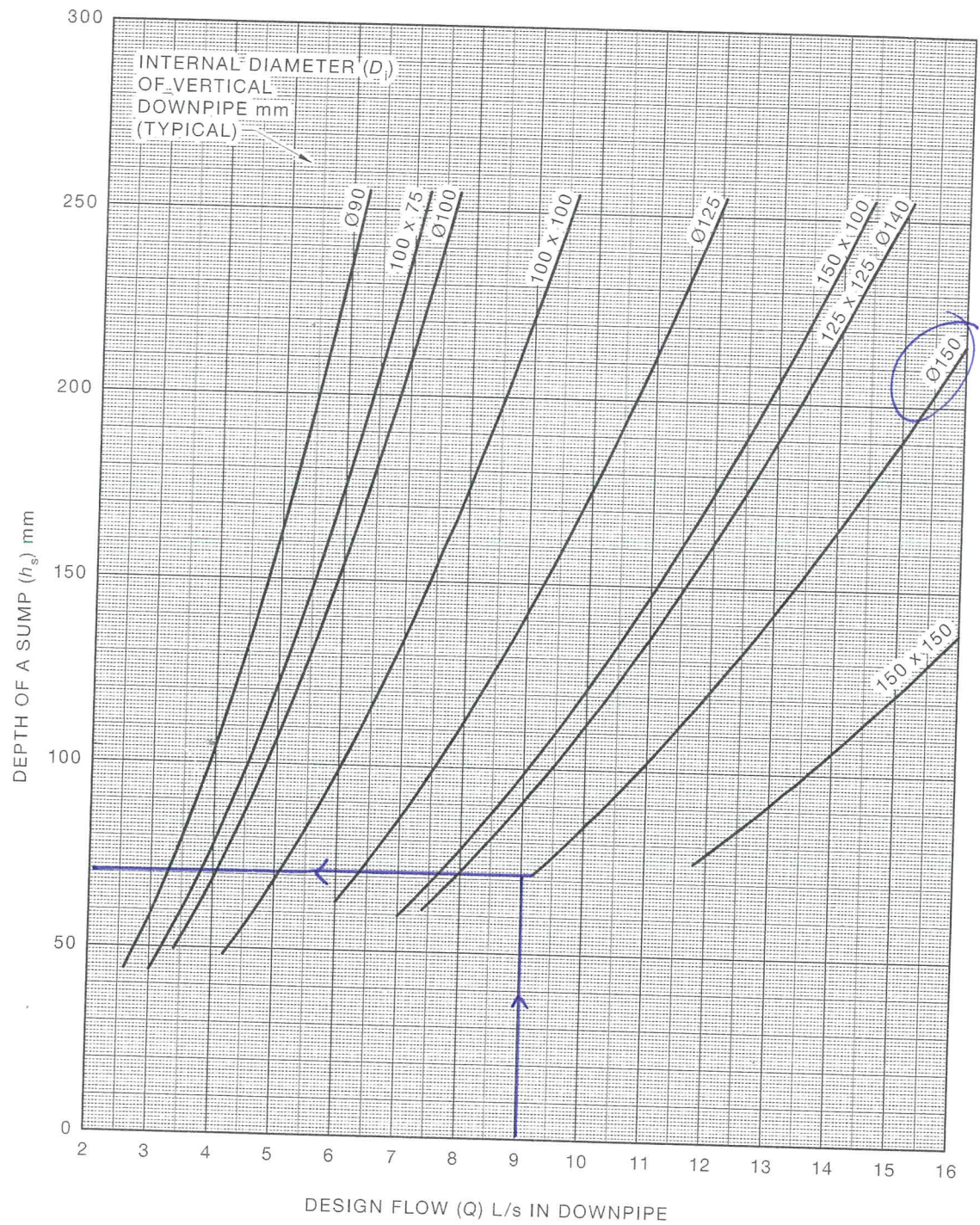
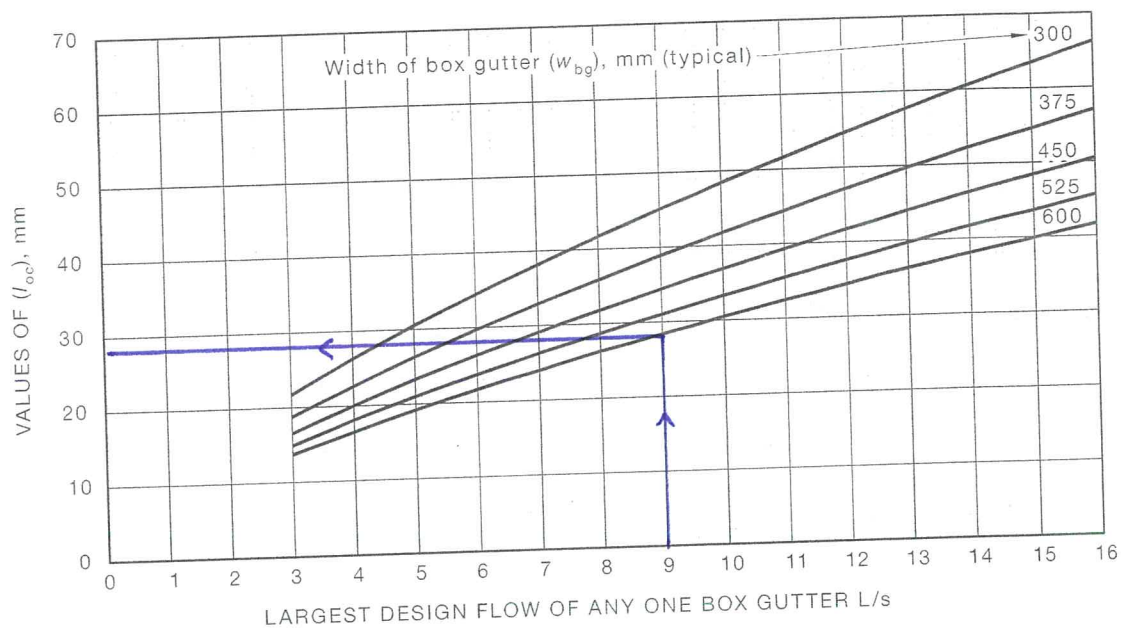
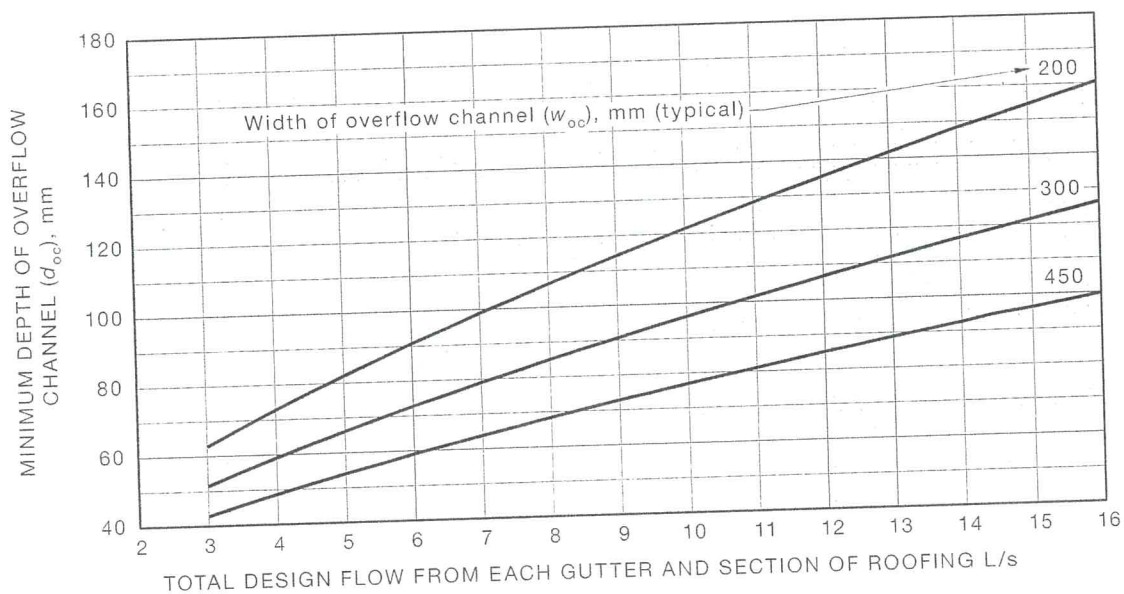


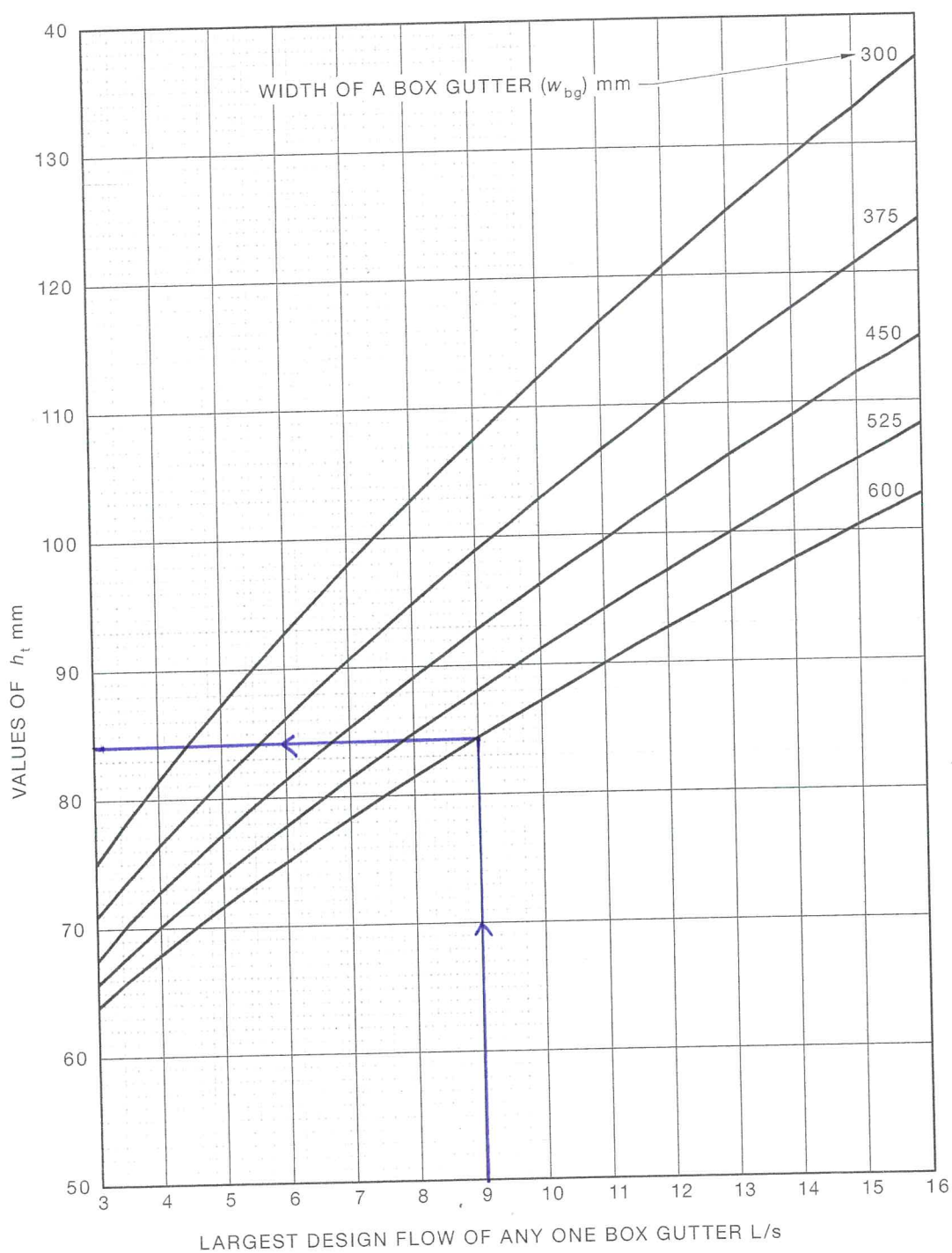
FIGURE 14 DESIGN GRAPH FOR SUMP

CJG
21/5/18

(a) Determination of values for l_{oc} (b) Determination of values for d_{oc}

NOTE: Graph (a) applies to both sump/side overflow device, and sump/high-capacity overflow device.

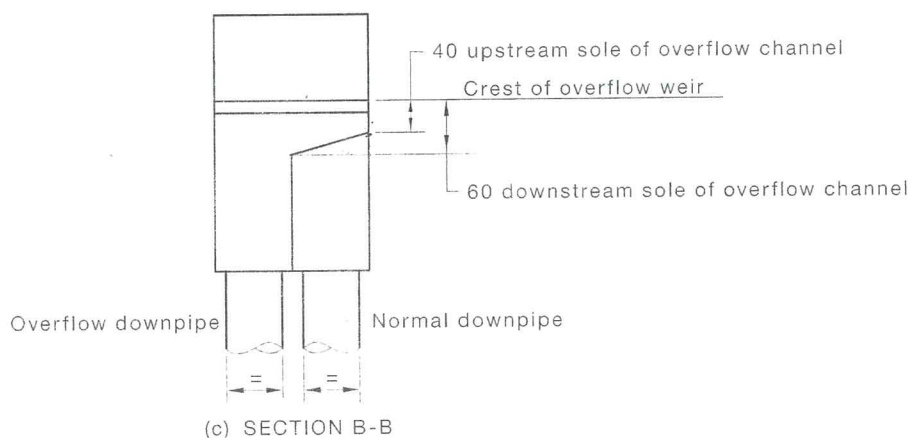
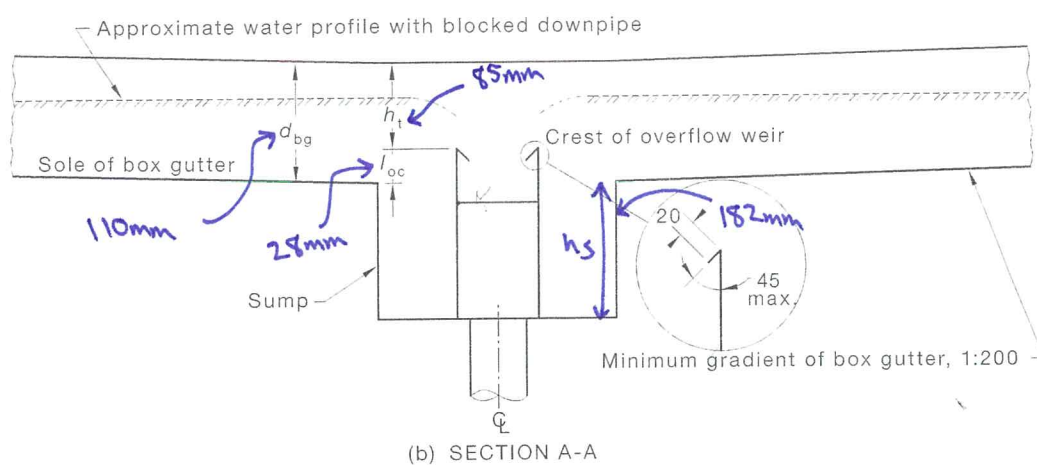
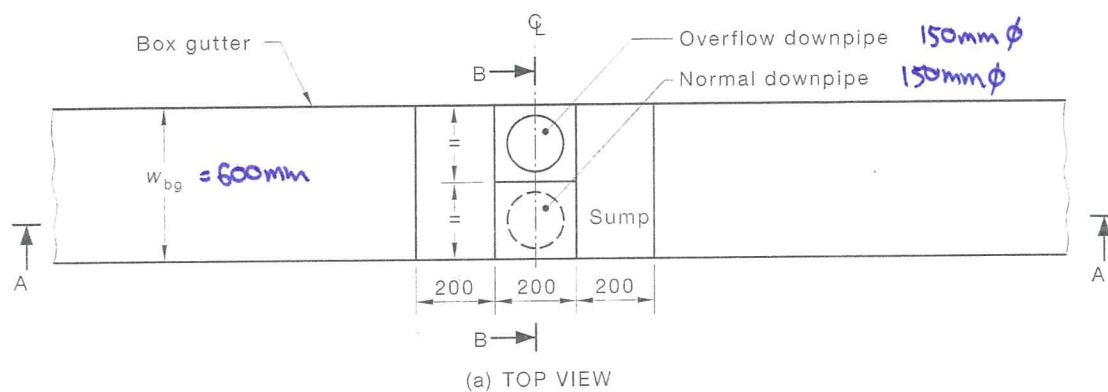
FIGURE 16 DESIGN GRAPH FOR SUMP/SIDE OVERFLOW DEVICE



NOTE: Additional values may be calculated using the equation:

$$Q = A_c \times {}^{100}I_5 / 3600$$

FIGURE 18 DESIGN GRAPH FOR SUMP/HIGH-CAPACITY OVERFLOW DEVICE



NOTES:

- The depth of the sump (h_s) shall be measured—
 - if $l_{oc} > 60$, from the sole of the box gutter at the sump; or
 - if $l_{oc} < 60$, the downstream sole of the overflow channel (i.e. $60 - l_{oc}$ below the sole of the box gutter at the sump).
- The sump shall be fully sealed to the box gutter.
- See Clause 3.7.5 for criteria for overflow devices.
- The normal outlet may be moved longitudinally to enable better inspection and maintenance access [see Clause 3.7.4 (f)].

DIMENSIONS IN MILLIMETRES

FIGURE 17 SUMP/HIGH-CAPACITY OVERFLOW DEVICE

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2/5/18



Project: 97 King William St, Kent Town
Designer: HP Date: 22/05/2018

Reference: 2018-7161
Checked by: CJG
Index: 11

STORMWATER DETENTION CALCULATIONS		REF./COMMENT
<p><u>Council Requirements</u></p> <ul style="list-style-type: none">- Post-development 5yr to be restricted to pre-development 5 yr.- Post-development 100yr to be restricted to pre-development 5yr.- Stormwater to be discharged to King William St kerb & water table.- 2,000L rainwater tank to be used for retention.		



Project: 97 King William St, Kent Town
 Designer: HP Date: 22/05/2018

Reference: 2018-7161
 Checked by: CJG
 Index: 12

STORMWATER DETENTION CALCULATIONS

REF./COMMENT

Council Requirements

	Pre	Post
ARI (years)	5	5
tc (min)	5	5

Site BOM IFDs

I(10/1) (mm/h)	23.4
Pre-dev I(5/5) (mm/h)	83.6
Post-dev I(5/5) (mm/h)	83.6

BOM IFD
 BOM IFD
 BOM IFD

Council Specified Pre-Development Runoff Coefficient

No	n/a
----	-----

Pre-Development Flow

Site Surfaces	Area (m2)	f
Roof	259.56	1
Concrete/Paved/Bitumen	331.37	0.9
Landscaped	0	0.1

Pre development
 Plan

Total Area = 590.93 m2
 favg = 0.944

C(10/1) = 0.100
 C10 = 0.855

C5 = 0.812

ARR Table 14.6	
ARI (years)	Frequency Factor, Fy
1	0.8
2	0.85
5	0.95
10	1
20	1.05
50	1.15
100	1.2

ARR Eq. 14.12
 ARR Eq. 14.11
 ARR Eq. 14.13

Pre Development Flow, Qpre = 11.16 L/s

Qpre = 11.16

5.3.2 Rational Method

(a) The Formula

As used in design, the formula of the Rational Method is:

$$Q_Y = 0.278 C_Y \cdot I_{t_c, Y} \cdot A \quad (5.1)$$

where Q_Y = peak flow rate (m^3/s) of average recurrence interval (ARI) of Y years

C_Y = runoff coefficient (dimensionless) for ARI of Y years

A = area of catchment (km^2)

$I_{t_c, Y}$ = average rainfall intensity (mm/h) for design duration of t_c hours and ARI of Y years.

The value of 0.278 (or 1/3.6) is merely a conversion factor to balance the units used. If area is in hectares instead of km^2 , the conversion factor is 0.00278 (or 1/360).

$$C_{10}^1 = 0.1 + (0.7 - 0.1) \times (I_1 - 25) / (70 - 25) \\ = 0.1 + 0.0133 \times (I_1 - 25) \quad (14.12)$$

$$C_{10} = 0.9 \times f + C_{10}^1 \times (1 - f) \quad (14.11)$$

$$C_Y = F_Y \cdot C_{10} \quad (14.13)$$



Project: 97 King William St, Kent Town
Designer: HP Date: 22/05/2018

Reference: 2018-7161
Checked by: CJG
Index: 13

STORMWATER DETENTION CALCULATIONS

REF./COMMENT

Post-Development Flow

Unrestricted Flow: Runoff considered to be undetained

Site Surfaces	Area (m2)	f
Roof	0	1
Concrete/Paved/Bitumen	0	0.9
Landscaped	0	0.1

Total Area = 0 m2
favg = 0.000

C10 = 0.100
C5 = 0.095

Unrestricted Post Development Flow, Qun-post = 0.00 L/s

Allowable Flow, Qall = 11.16 L/s

Restricted Flow: Runoff considered to be detained

Site Surface	Area (m2)	f
Roof	495.64	1
Concrete/Paved/Bitumen	95.29	0.9
Landscaped	0	0.1

Total Area = 590.93 m2
favg = 0.984

C10 = 0.887
C5 = 0.843

Refer to attached detention calculations

Post-Development
Catchment Plan

ARR Eq. 14.11
ARR Eq. 14.13

Qun-post = 0.00

Qall = 11.16

Roof Catchment
Plan

ARR Eq. 14.11
ARR Eq. 14.13
C5 = 0.843



Project: 97 King William St, Kent Town

Designer: HP

Date: 22/05/2018

Reference: 2018-7161

Checked by: CJG

Index: 14

STORMWATER DETENTION CALCULATIONS

Detention Calculations

ARI = 5 years
Area = 590.93 m²
tc = 5 min
C5 = 0.843

Detention Volume Required = 122 L

Duration	mm/h	In flow L/s	Out flow allowed	Qin - Qout	Detention Required
5	83.60	11.56	11.16	0.41	122
6	77.70	10.75	11.16	-0.41	0
7	73.83	10.21	11.16	-0.94	0
8	69.95	9.68	11.16	-1.48	0
9	66.08	9.14	11.16	-2.02	0
10	62.20	8.60	11.16	-2.55	0
11	60.35	8.35	11.16	-2.81	0
12	58.50	8.09	11.16	-3.06	0
13	56.65	7.84	11.16	-3.32	0
14	54.80	7.58	11.16	-3.58	0
15	52.95	7.32	11.16	-3.83	0
16	51.10	7.07	11.16	-4.09	0
17	49.25	6.81	11.16	-4.34	0
18	47.40	6.56	11.16	-4.60	0
19	45.55	6.30	11.16	-4.86	0
20	43.70	6.05	11.16	-5.11	0
21	42.79	5.92	11.16	-5.24	0
22	41.88	5.79	11.16	-5.36	0
23	40.97	5.67	11.16	-5.49	0
24	40.06	5.54	11.16	-5.62	0
25	39.15	5.42	11.16	-5.74	0
26	38.24	5.29	11.16	-5.87	0
27	37.33	5.16	11.16	-5.99	0
28	36.42	5.04	11.16	-6.12	0
29	35.51	4.91	11.16	-6.24	0
30	34.60	4.79	11.16	-6.37	0
31	34.20	4.73	11.16	-6.43	0
32	33.80	4.68	11.16	-6.48	0
33	33.40	4.62	11.16	-6.54	0
34	33.00	4.57	11.16	-6.59	0
35	32.60	4.51	11.16	-6.65	0
36	32.20	4.45	11.16	-6.70	0
37	31.80	4.40	11.16	-6.76	0
38	31.40	4.34	11.16	-6.81	0
39	31.00	4.29	11.16	-6.87	0
40	30.60	4.23	11.16	-6.92	0
41	30.20	4.18	11.16	-6.98	0
42	29.80	4.12	11.16	-7.03	0

43	29.40	4.07	11.16	-7.09	0
44	29.00	4.01	11.16	-7.15	0
45	28.60	3.96	11.16	-7.20	0
46	28.20	3.90	11.16	-7.26	0
47	27.80	3.85	11.16	-7.31	0
48	27.40	3.79	11.16	-7.37	0
49	27.00	3.74	11.16	-7.42	0
50	26.60	3.68	11.16	-7.48	0
51	26.20	3.62	11.16	-7.53	0
52	25.80	3.57	11.16	-7.59	0
53	25.40	3.51	11.16	-7.64	0
54	25.00	3.46	11.16	-7.70	0
55	24.60	3.40	11.16	-7.75	0
56	24.20	3.35	11.16	-7.81	0
57	23.80	3.29	11.16	-7.86	0
58	23.40	3.24	11.16	-7.92	0
59	23.00	3.18	11.16	-7.98	0
60	22.60	3.13	11.16	-8.03	0



Project: 97 King William St, Kent Town
 Designer: HP Date: 22/05/2018

Reference: 2018-7161
 Checked by: CJG
 Index: 15

STORMWATER DETENTION CALCULATIONS

REF./COMMENT

Council Requirements

	Pre	Post
ARI (years)	5	100
tc (min)	5	5

Site BOM IFDs

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Pre-dev I(5/5) (mm/h)	83.6
Post-dev I(100/5) (mm/h)	182

BOM IFD
 BOM IFD
 BOM IFD

Council Specified Pre-Development Runoff Coefficient

No	n/a
----	-----

Pre-Development Flow

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Pre development
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 ARR Eq. 14.11
 ARR Eq. 14.13

Pre Development Flow, Qpre = 11.16 L/s

Qpre = 11.16

5.3.2 Rational Method

(a) The Formula

As used in design, the formula of the Rational Method is:

$$Q_Y = 0.278 C_Y \cdot I_{t_c, Y} \cdot A \quad (5.1)$$

where Q_Y = peak flow rate (m^3/s) of average recurrence interval (ARI) of Y years

C_Y = runoff coefficient (dimensionless) for ARI of Y years

A = area of catchment (km^2)

$I_{t_c, Y}$ = average rainfall intensity (mm/h) for design duration of t_c hours and ARI of Y years.

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$$C_{10}^1 = 0.1 + (0.7 - 0.1) \times (10I_1 - 25) / (70 - 25) \\ = 0.1 + 0.0133 \times (10I_1 - 25) \quad (14.12)$$

$$C_{10} = 0.9 \times f + C_{10}^1 \times (1 - f) \quad (14.11)$$

$$C_Y = F_Y \cdot C_{10} \quad (14.13)$$



Project: 97 King William St, Kent Town
 Designer: HP Date: 22/05/2018

Reference: 2018-7161
 Checked by: CJG
 Index: 16

STORMWATER DETENTION CALCULATIONS

REF./COMMENT

Post-Development Flow

Unrestricted Flow: Runoff considered to be undetained

Site Surfaces	Area (m2)	f
Roof	0	1
Concrete/Paved/Bitumen	0	0.9
Landscaped	0	0.1

Total Area = 0 m2
 favg = 0.000

C10 = 0.100
 C100 = 0.120

Unrestricted Post Development Flow, Qun-post = 0.00 L/s

Allowable Flow, Qall = 11.16 L/s

Restricted Flow: Runoff considered to be detained

Site Surface	Area (m2)	f
Roof	495.64	1
Concrete/Paved/Bitumen	95.29	0.9
Landscaped	0	0.1

Total Area = 590.93 m2
 favg = 0.984

C10 = 0.887
 C100 = 1.000

Refer to attached detention calculations

Post-Development
 Catchment Plan

ARR Eq. 14.11
 ARR Eq. 14.13

Qun-post = 0.00

Qall = 11.16

Roof Catchment
 Plan

ARR Eq. 14.11
 ARR Eq. 14.13
 C100 = 1.000



Project: 97 King William St, Kent Town

Designer: HP

Date: 22/05/2018

Reference: 2018-7161

Checked by: CJG

Index: 17

STORMWATER DETENTION CALCULATIONS

Detention Calculations

ARI = 100 years

Area = 590.93 m²

tc = 5 min

C100 = 1.000

Detention Volume Required = 6728 L

Duration	mm/h	In flow L/s	Out flow allowed	Qin - Qout	Detention Required
5	182.00	29.87	11.16	18.72	5615
6	169.00	27.74	11.16	16.58	5970
7	160.00	26.26	11.16	15.11	6345
8	151.00	24.79	11.16	13.63	6542
9	142.00	23.31	11.16	12.15	6562
10	133.00	21.83	11.16	10.67	6405
11	128.84	21.15	11.16	9.99	6595
12	124.68	20.47	11.16	9.31	6702
13	120.52	19.78	11.16	8.63	6728
14	116.36	19.10	11.16	7.94	6672
15	112.20	18.42	11.16	7.26	6534
16	108.04	17.73	11.16	6.58	6314
17	103.88	17.05	11.16	5.89	6013
18	99.72	16.37	11.16	5.21	5629
19	95.56	15.69	11.16	4.53	5163
20	91.40	15.00	11.16	3.85	4615
21	89.42	14.68	11.16	3.52	4437
22	87.44	14.35	11.16	3.20	4219
23	85.46	14.03	11.16	2.87	3962
24	83.48	13.70	11.16	2.55	3666
25	81.50	13.38	11.16	2.22	3331
26	79.52	13.05	11.16	1.90	2958
27	77.54	12.73	11.16	1.57	2545
28	75.56	12.40	11.16	1.25	2093
29	73.58	12.08	11.16	0.92	1602
30	71.60	11.75	11.16	0.60	1073
31	70.74	11.61	11.16	0.45	846
32	69.88	11.47	11.16	0.31	602
33	69.02	11.33	11.16	0.17	341
34	68.16	11.19	11.16	0.03	64
35	67.30	11.05	11.16	-0.11	0
36	66.44	10.91	11.16	-0.25	0
37	65.58	10.76	11.16	-0.39	0
38	64.72	10.62	11.16	-0.53	0
39	63.86	10.48	11.16	-0.67	0
40	63.00	10.34	11.16	-0.82	0
41	62.14	10.20	11.16	-0.96	0
42	61.28	10.06	11.16	-1.10	0

43	60.42	9.92	11.16	-1.24	0
44	59.56	9.78	11.16	-1.38	0
45	58.70	9.64	11.16	-1.52	0
46	57.84	9.49	11.16	-1.66	0
47	56.98	9.35	11.16	-1.80	0
48	56.12	9.21	11.16	-1.95	0
49	55.26	9.07	11.16	-2.09	0
50	54.40	8.93	11.16	-2.23	0
51	53.54	8.79	11.16	-2.37	0
52	52.68	8.65	11.16	-2.51	0
53	51.82	8.51	11.16	-2.65	0
54	50.96	8.36	11.16	-2.79	0
55	50.10	8.22	11.16	-2.93	0
56	49.24	8.08	11.16	-3.07	0
57	48.38	7.94	11.16	-3.22	0
58	47.52	7.80	11.16	-3.36	0
59	46.66	7.66	11.16	-3.50	0
60	45.80	7.52	11.16	-3.64	0

- Total detention volume required is 6,728L.

- Total retention volume required is 2,000L.

- Adopt two 1,000L above ground tanks located on ground floor for retention.

- Adopt a 6,728L underground tank located on basement floor for detention.



Project: 97 King William St, Kent Town
Designer: HP Date: 16/05/2018

Reference: 2018-7161
Checked by: CJG
Index: 18

CALCULATIONS		REF./COMMENT																	
Orifice Diameter Calculations for 5yr		Equation 5.7																	
Using DRAINS Equation 5.7																			
OSD storages are usually controlled by circular orifices with the discharge equation being:																			
$Q = C_o \cdot \frac{\pi}{4} d^2 \cdot (2gh)^{0.5}$																			
where C_o is a contraction coefficient, taken as a constant of 0.6 in DRAINS, d is the orifice diameter (m), g is the acceleration due to gravity (m/s ²), and h is the height from the water surface to the centre of the orifice (m).																			
Tank Dimensions (Indicative Only) Refer 'Novaplas Drainwell modular system' Adopt 5x11x1 units (5x0.755, 11x0.4, 1x0.44) Width (m) Length (m) Height (m)																			
<table><tr><td>3.78</td><td>4.4</td><td>0.44</td></tr></table>			3.78	4.4	0.44														
3.78	4.4		0.44																
Orifice Calculation																			
<table><tr><td>Detention volume, V</td><td>7.26</td><td>m^3</td></tr><tr><td>Gravity, g</td><td>9.81</td><td>m/s^2</td></tr><tr><td>Concentration coefficient, Cc</td><td>0.6</td><td></td></tr><tr><td>Orifice diameter, d</td><td>0.0925</td><td>m</td></tr><tr><td>Height of water surface to orifice centre, h</td><td>0.390257937</td><td>m</td></tr><tr><td>Required discharge rate, Q</td><td>11.15706519</td><td>L/s</td></tr></table>			Detention volume, V	7.26	m^3	Gravity, g	9.81	m/s^2	Concentration coefficient, Cc	0.6		Orifice diameter, d	0.0925	m	Height of water surface to orifice centre, h	0.390257937	m	Required discharge rate, Q	11.15706519
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Height of water surface to orifice centre, h	0.390257937	m																	
Required discharge rate, Q	11.15706519	L/s																	
<table><tr><td>Orifice diameter</td><td>92.50</td><td>mm</td></tr></table>		Orifice diameter	92.50	mm															
Orifice diameter	92.50	mm																	
Required orifice plate diameter is 92.5mm to restrict underground rainwater tank outflow to pre-development 5 year flow rate of 11.16L/s. Adopt 90mm orifice plate for ease of construction.																			

Intensity Frequency Duration Table

Location: 34.925S 138.625E Issued: 10/4/2018

Rainfall intensity in mm/h for various durations and Average Recurrence Interval

Average Recurrence Interval

Duration	1 YEAR	2 YEARS	5 YEARS	10 YEARS	20 YEARS	50 YEARS	100 YEARS
5Mins	45.9	61.4	83.6	99.7	121	154	182
6Mins	42.7	57.1	77.7	92.6	113	143	169
10Mins	34.5	46.0	62.2	73.9	89.7	113	133
20Mins	24.6	32.6	43.7	51.5	62.2	78.0	91.4
30Mins	19.6	26.0	34.6	40.7	49.0	61.2	71.6
1Hr	13.0	17.2	22.6	26.4	31.7	39.3	45.8
2Hrs	8.53	11.2	14.6	16.9	20.2	24.8	28.8
3Hrs	6.67	8.73	11.3	13.0	15.5	19.0	21.9
6Hrs	4.37	5.69	7.27	8.33	9.82	12.0	13.7
12Hrs	2.80	3.63	4.60	5.25	6.17	7.48	8.56
24Hrs	1.69	2.20	2.80	3.20	3.77	4.58	5.25
48Hrs	.957	1.25	1.61	1.85	2.19	2.68	3.09
72Hrs	.671	.882	1.14	1.31	1.56	1.91	2.21

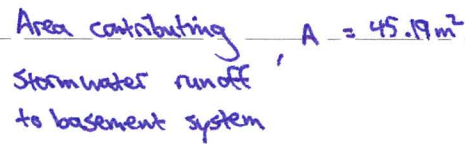
aw data: 17.77, 3.79, 0.91, 35.01, 6.8, 1.72, skew=0.56, F2=4.47, F50=14.98)

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CG
21/5/18

Basement

2018.04.10 - Post development Plan



BASEMENT PLAN

scale 1:100

NOTE: REFER TO ENGINEERS DETAILS
FOR ALL SERVICES AND SLAB
PENETRATIONS INCLUDING ELECTRICAL,
HYDRAULIC AND FIRE.

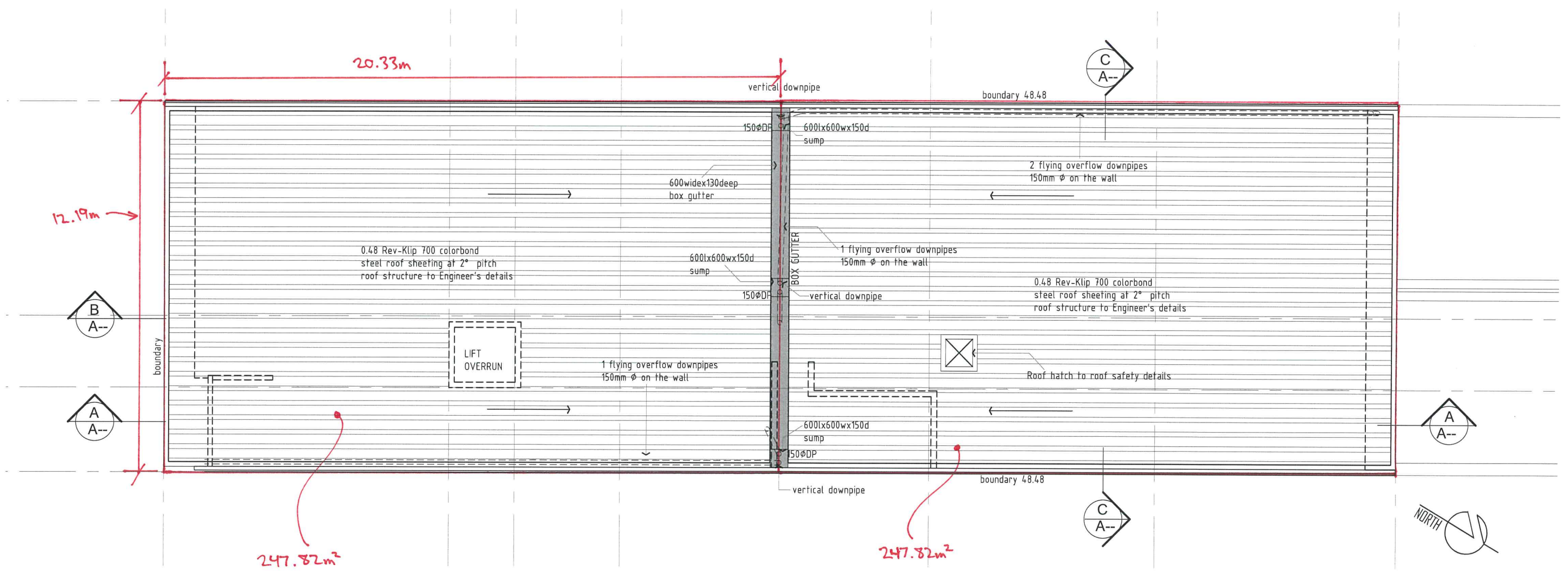


GROUND FLOOR PLAN

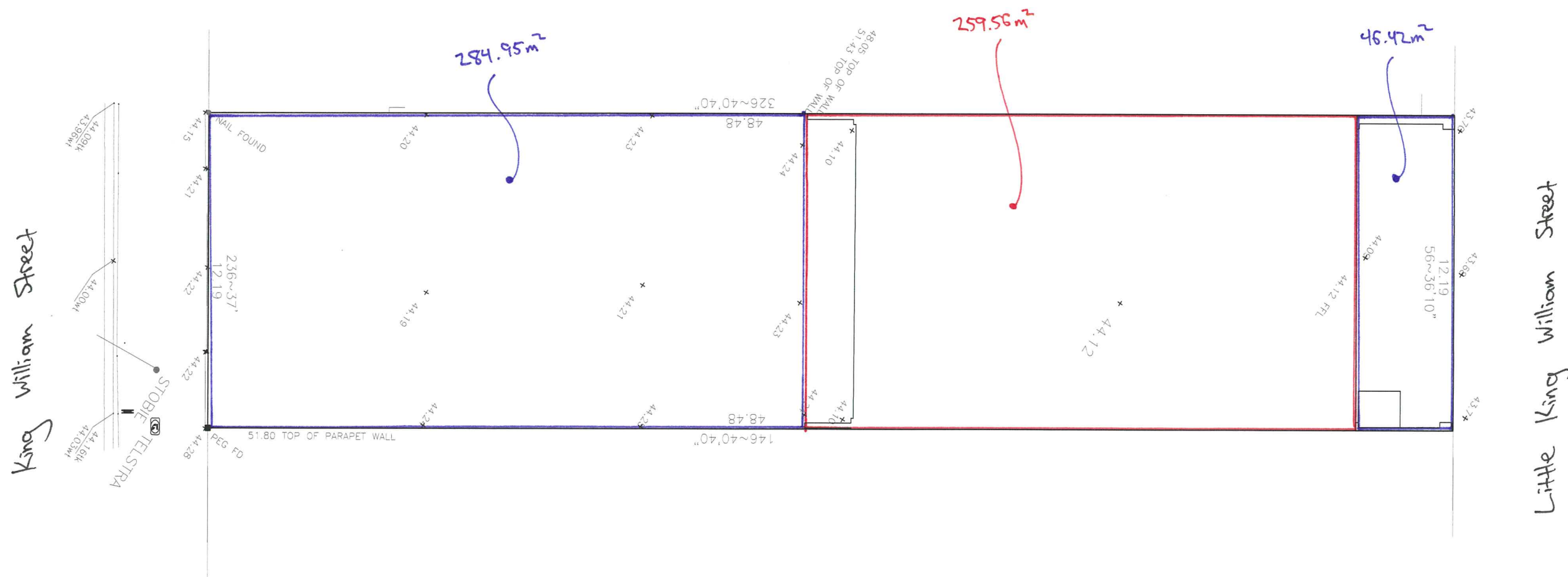
scale 1:100

NOTE: REFER TO ENGINEERS DETAILS
FOR ALL SERVICES AND SLAB
PENETRATIONS INCLUDING ELECTRICAL,
HYDRAULIC AND FIRE.

2018.05.04 - Roof Catchment Plan



2018.05.14 - Pre development Plan



Total site area, $A_{\text{site}} = 590.93 \text{ m}^2$

Total roof area, $A_{\text{roof}} = 259.56 \text{ m}^2$

Total impervious surface area, $A_{\text{surface}} = 331.37 \text{ m}^2$