

NEW ISLINGTON PHASE 3

DRAINAGE STATEMENT – Revision E

1.0 INTRODUCTION

- 1.1 The current proposals comprise of a terrace block of 13 houses, 3 terrace blocks of 7 houses and 2 taller mansion house blocks adjacent to the canal.

The development includes a pattern of new private access roads and gardens and sits adjacent to the earlier Phase 1 and Phase 2 developments comprising terraced housing and a Mansion House block adjacent to the canal.

A post development site layout drawing is included in Appendix A.

2.0 DRAINAGE PROPOSALS

- 2.1 The strategic drainage proposals are to drain the site by utilising new separate foul and surface water systems.

The existing drainage layout following the completion of the Phase 2 works and the new proposed drainage layout are included in Appendix B.

2.2 FOUL WATER

- 2.2.1 As shown on the proposed drainage layout in Appendix B, and as agreed with United Utilities, foul water from the site will drain under gravity within the site before discharging to the existing combined adopted sewer in Coppersmith Road.

- 2.2.2 Connection for the entirety of the Phase 3, is made via a new connection to the combined adopted sewer in Coppersmith Road.

- 2.2.3 The proposed total maximum foul water discharge has been calculated in accordance with BS EN752 as 21.2l/s.

2.3 SURFACE WATER

- 2.3.1 In developing the design for surface water disposal from the site, based on sustainable drainage principles, various options were considered as summarised below.

Assessment is based on the generally accepted hierarchy for surface water disposal in accordance with the NPPF 2018 and National Standards for Sustainable Drainage Systems.

2.3.1.1 INFILTRATION TO THE GROUND

A Phase 2 Geo-Environmental Site Investigation has been carried out across the site by specialist geotechnical and environmental consultants, ref: LKC 16 1334. This is included in Appendix C.

Investigations comprised boreholes, trial pits and dynamic sampling across the site and show ground conditions to comprise a significant depth of reworked made ground

(up to 6-8m) associated with historical development and re-profiling at the time of the construction of the canals over stiff natural clays deposits.

As noted in the site investigation report, as part of the site investigation fieldwork, a series of soakaway tests were carried out in trial pits to ascertain the infiltration in the near surface formation and the suitability of using this formation as a soakaway for surface water. Soil infiltration rates were recorded as 'failed or very slow' with the report concluding that the ground is unlikely to be suitable for shallow soakaways or infiltration.

Therefore, as part of the proposals, surface water infiltration to the ground is limited to direct rainfall onto gardens areas only.

2.3.1.2 SWALES / DITCHES / BASINS

Due to the city centre location of the proposed development there is insufficient space available for the inclusion of swales, ditches or basins and therefore these methods have been discounted as not being practical.

2.3.1.3 PERMEABLE PAVING

As noted above site investigation has shown that the site is unlikely to be suitable for shallow soakaways or infiltration. The use of permeable paving directly draining to the ground is therefore discounted.

2.3.1.4 GREEN ROOFS

Use of green roof space has been reviewed by the Architect, however due to current detailing and preferred prefabricated modular construction techniques has been discounted as impractical.

2.3.1.5 DISCHARGE TO A WATERCOURSE

Options to discharge to potentially available watercourses has been considered in the design as described below.

2.3.2.5.1 'SHOOTERS BROOK'

It is understood that historically the Shooters Brook watercourse ran close to the site, initially in an open cut as shown on maps dating to 1794 before later being culverted following industrial development of the area around 1831.

This surface water sewer remains indicated on some of the available UU sheets though there are discrepancies in the layouts, particularly at the sheet boundaries. Detailed investigations were therefore undertaken at the time of the development of the new canal basin in 2002 with the aim of confirming the existing layout and presence / path of the culverted watercourse.

This work is summarised in the Martin Stockley Associates report included in Appendix D, 'Summary of Desk Study and Site Investigation on Shooters Brook', the findings of which were not consistent with the UU records allowing the conclusion that the Shooters Brook culvert was no longer in operation as a watercourse.

The potential to drain surface water to Shooters Brook watercourse has therefore been discounted.

2.3.2.5.2 CANAL BASIN

The existing recently completed canal basin sits adjacent to the site to the South West, though due to the level differences across the site this is at a higher level than the majority of the site.

The relative level of any outfall and potential problems associated with canal water levels present particular difficulties in the draining of the site to the canal without the use of deep drainage runs and pumping stations.

Furthermore, Manchester City Council, as owners of the canal basin have confirmed that surface water drainage to the canal basin is not permitted due to existing issues with the canal water level and potential overtopping in times of high flow.

The potential to drain surface water to the Canal Basin has therefore been discounted.

2.3.1.6 CONNECTION TO AN EXISTING SURFACE WATER SEWER

There is no existing surface water network in the vicinity of the site so this option is discounted.

2.3.1.7 CONNECTION TO AN EXISTING PUBLIC COMBINED SEWER

There is an existing 1500mm diameter public combined sewer crossing the site, colloquially referred to as the 'Shooters Brook Relief Sewer'. It is assumed that the Shooters Brook watercourse as described above was historically diverted into the relief sewer and this sewer also carried foul and surface water drainage from the site and wider area via connections from the former road network and low-rise council house development.

Drainage for the dwellings adjacent to the North East boundary of the site, constructed in 2010, consists of separate attenuated surface and foul water systems, which combine to connect to the existing combined sewer in Coppersmith Road.

It is proposed that the new foul and surface water network will similarly be connected to the existing combined sewers in Coppersmith Road as shown on the proposed layout drawing 284/05/60 in Appendix B.

Whilst the proposed site layout and the proportion of impermeable surfacing contributing to the surface water runoff is broadly similar to the arrangement prior to the clearance of the Cardroom Estate, the requirements of the Manchester City, Salford City and Trafford Councils Level 2 Hybrid SFRA will be recognised in the design.

In accordance with this document the development will provide a minimum reduction of surface water runoff of 50% compared with the arrangement directly prior to site clearance with the aim of reducing further where this is reasonably practical, towards the greenfield run off rate. This will be achieved through attenuation of surface water flows including on site storage and discharge at controlled rates to the network.

Initial calculations included in Appendix E for information are based on a return period of 1 in 100 years including a 40% allowance for climate change and suggest a minimum proposed total allowable flow of 18l/s can be achieved due to topographical site constraints and storage limitations. This represents a reduction of surface water flow of around 78% when compared to the arrangement directly prior to site clearance.

The required total attenuation storage volume is therefore 325m³. This volume will be

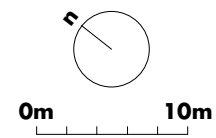
split between two attenuation tanks positioned at the upper and lower site levels, each with the necessary flow control to limit the total discharge to 18l/s.

- 2.3.2 Car parking across the development is generally limited to individual plots or adjacent to Mansion Houses and consequently the likelihood and risk of small spillages is considered to be low.

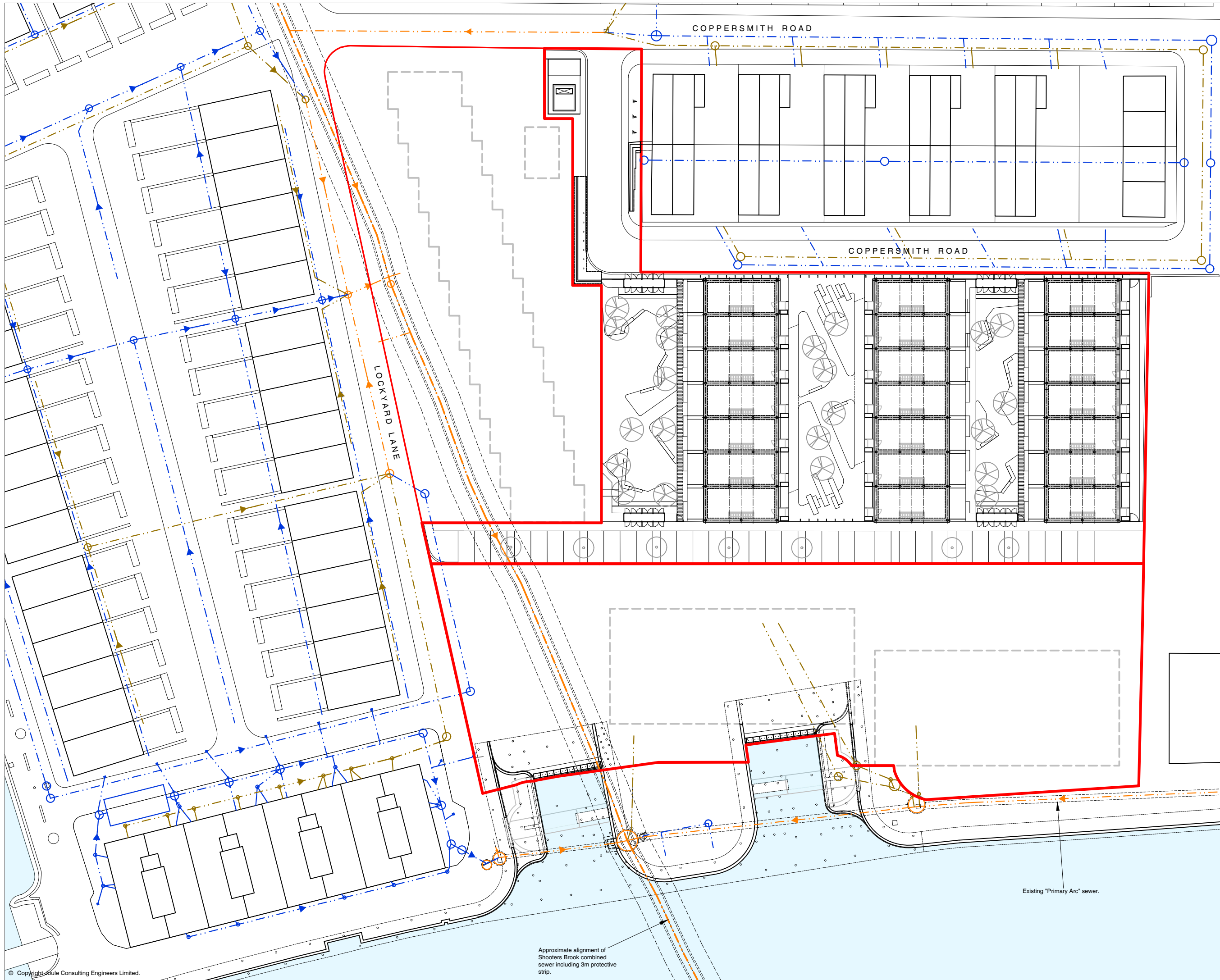
Pollution protection will be provided in the conventional manner by a pattern of trapped road gullies at regular positions along road gutters.

APPENDIX A - POST DEVELOPMENT SITE LAYOUT

- 01 town house 3-storey
- 02 mansion house
- 03 refuse store
- 04 sub station



APPENDIX B – EXISTING AND PROPOSED SITE DRAINAGE



Notes:

1. This drawing is to be read in conjunction with the site survey and all relevant Architects and Engineers drawings and the specification.
2. For general notes refer to drawing no. 284-05-....
3. For setting out refer to Architects details.
4. For assumed site build ups over formation level, refer to drawing 284-05-410 to 412.

Key:

- Phase 3A Site Boundary.
- Phase 3 Site Boundary.
- Phase 3 Later Construction Building Outline.

INFORMATION

Date	Issue	Rev.	Revised	Checked

joule

Joule Consulting Engineers Ltd
T: 0161 832 3467
W: www.jouleconsultingengineers.co.uk
E: mail@jouleconsultingengineers.co.uk

Timber Wharf
40 Worsley Street
Castlefield
Manchester
M15 4LD

Project:
N13,
New Islington,
Manchester.

Title:
Existing,
Phase 3,
Drainage,
Overall Site.

Job No: 284-05	Orig No: 30	Rev: -
Scale: 1:500 @ A3	Date: 07.20	Drawn: TP

APPENDIX C – SITE INVESTIGATION REPORT, LKC 16 1334

APPENDIX D – MARTIN STOCKLEY ASSOCIATES REPORT, SUMMARY OF DESK
STUDY AND SITE INVESTIGATION ON SHOOTERS BROOK

APPENDIX E - FOUL AND SURFACE WATER DRAINAGE CALCULATIONS

Job No	284/05	Sheet No	FOUL 01		
Project	NEW ULLINGTON PHASE 3	Engineer	jin	Date	09/18
Element	FOUL DRAINAGE.	Checked by		Date	

CALCULATE FOW DRAINAGE FLOW IN ACCORDANCE WITH BS EN 752.

DISCHARGE UNIT METHOD.

HOUSE.	No	DU	ΣDU
Wash basin/shower	3	0.45	1.35
Bath/kitchen sink	2	1.0	2.0
Dishwasher	1	0.5	0.5
Washing machine	1	0.65	0.65
WC.	2	1.8	3.6
			<u>8.1 DU</u>

$$33 \text{ houses} \times 8.1 = \underline{\underline{267 DU}}$$

APARTMENT.

As above by inspection. $8.1 \times 60 = \underline{\underline{486 DU}}$.

MANION HOUSE.

GROUND FLOOR.

COMMERCIAL UNIT.

Say: -

Sink	2	1.0	2.0
WC.	3	1.8	5.4
			<u>7.4 DU.</u>

$$6 \text{ Commercial Units} \times 7.4 = \underline{\underline{44.4 DU}}$$

$$\text{Flow drains. say } 6 \times 2 \times 1.0 DU = \underline{\underline{12.0 DU}}$$

$$\boxed{\Sigma DU = 810 DU}$$

Job No 284/05	Sheet No Form 02	
Project NEW ULLINGTON PHASE 3	Engineer j/w	Date 09/18.
Element FOUL DRAINAGE.	Checked by	Date

$$\text{MAX FLOW } Q = kdu \sqrt{Sdu}$$

$$\text{Take } kdu \text{ Residential} = 0.5$$

$$kdu \text{ Commercial} = 1.0$$

$$Q = (0.5 \sqrt{753}) + (1.0 \sqrt{56.4})$$

$$Q = \underline{\underline{21.2 \text{ l/s.}}}$$

Job No 284/05

Sheet No SURFACE 01

Project NEW ULLINGTON PHASE 3

Engineer SJW

Date 01/19

Element SURFACE WATER DRAINAGE.

Checked by

Date

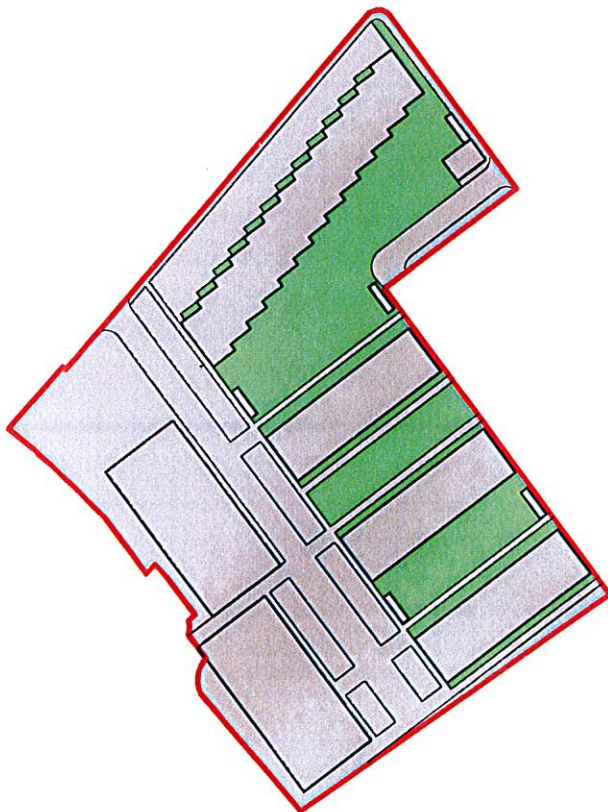


CALVERTON ESTATE - PRE DEMOLITION

$$Area = 8900 \text{ m}^2$$

$$Pimp = 0.70.$$

Consider original layout to determine max former flows and allowable discharge based on a 50% reduction in flow.



PROPOSED.

$$Area = 8900 \text{ m}^2$$

$$Pimp = 0.75.$$

We proposed layout and allowable discharge to demonstrate required storage volume.

Job No 284/05	Sheet No SURFACE 02	
Project NEW ISLINGTON PHASE 3	Engineer SJW	Date 09/18
Element SURFACE WATER DRAINAGE	Checked by	Date

SURFACE WATER ATTENUATION CALCULATIONS.

Overall Phase 3 site area = 8900 m^2

$\text{PIMP} = 0.70$.

CALCULATE EXISTING (FORMER) SITE PEAK SURFACE WATER FLOWS USING THE WATLINGFORD PROCEDURE.

REFER TO ATTACHED TPODS CALC.

M1-5 storm	$Q = 83 \text{ l/s}$
M2-5 storm	$Q = 106 \text{ l/s}$
M5-5 storm	$Q = 138 \text{ l/s}$
M30-5 storm	$Q = 195 \text{ l/s}$
M100-5 storm	$Q = 246 \text{ l/s}$

 Joule Consulting Engineers Timber Wharf, 40 Worsley Street Manchester M15 4LD	Project New Islington Phase 3				Job no. 284/05	
	Calcs for Existing M1-5				Start page no./Revision 1	
	Calcs by SW	Calcs date 13/09/2018	Checked by	Checked date	Approved by	Approved date

DESIGN RAINFALL

In accordance with the Wallingford Procedure

Tedd's calculation version 2.0.00

Design rainfall intensity

Location of catchment area	Manchester
Storm duration	D = 5 min
Return period	Period = 1 yr
Ratio 60 min to 2 day rainfall of 5 yr return period	r = 0.360
5-year return period rainfall of 60 minutes duration	M5_60min = 18.0 mm
Increase of rainfall intensity due to global warming	p _{climate} = 0 %
Factor Z1 (Wallingford procedure)	Z1 = 0.36
Rainfall for 5min storm with 5 year return period	M5_5min _i = Z1 × M5_60min = 6.5 mm
Factor Z2 (Wallingford procedure)	Z2 = 0.62
Rainfall for 5min storm with 1 year return period	M1_5min = Z2 × M5_5min _i = 4.0 mm
Design rainfall intensity	I _{max} = M1_5min / D = 48.0 mm/hr

Maximum surface water runoff

Catchment area	A _{catch} = 8900 m ²
Percentage of area that is impermeable	p = 70 %
Maximum surface water runoff	Q _{max} = A _{catch} × p × I _{max} = 83.0 l/s

SURFACE 04

 Joule Consulting Engineers Timber Wharf, 40 Worsley Street Manchester M15 4LD	Project New Islington Phase 3				Job no. 284/04	
	Calcs for Existing M2-5				Start page no./Revision 1	
	Calcs by SW	Calcs date 13/09/2018	Checked by	Checked date	Approved by	Approved date

DESIGN RAINFALL

In accordance with the Wallingford Procedure

Tedds calculation version 2.0.00

Design rainfall intensity

Location of catchment area	Manchester
Storm duration	D = 5 min
Return period	Period = 2 yr
Ratio 60 min to 2 day rainfall of 5 yr return period	r = 0.360
5-year return period rainfall of 60 minutes duration	M5_60min = 18.0 mm
Increase of rainfall intensity due to global warming	p _{climate} = 0 %
Factor Z1 (Wallingford procedure)	Z1 = 0.36
Rainfall for 5min storm with 5 year return period	M5_5min _i = Z1 × M5_60min = 6.5 mm
Factor Z2 (Wallingford procedure)	Z2 = 0.79
Rainfall for 5min storm with 2 year return period	M2_5min = Z2 × M5_5min _i = 5.1 mm
Design rainfall intensity	I _{max} = M2_5min / D = 61.4 mm/hr

Maximum surface water runoff

Catchment area	A _{catch} = 8900 m ²
Percentage of area that is impermeable	p = 70 %
Maximum surface water runoff	Q _{max} = A _{catch} × p × I _{max} = 106.3 l/s

 Joule Consulting Engineers Timber Wharf, 40 Worsley Street Manchester M15 4LD	Project New Islington Phase 3				Job no. 284/04	
	Calcs for Existing M5-5				Start page no./Revision 1	
	Calcs by SW	Calcs date 13/09/2018	Checked by	Checked date	Approved by	Approved date

DESIGN RAINFALL

In accordance with the Wallingford Procedure

Tedds calculation version 2.0.00

Design rainfall intensity

Location of catchment area	Manchester
Storm duration	D = 5 min
Return period	Period = 5 yr
Ratio 60 min to 2 day rainfall of 5 yr return period	r = 0.360
5-year return period rainfall of 60 minutes duration	M5_60min = 18.0 mm
Increase of rainfall intensity due to global warming	p _{climate} = 0 %
Factor Z1 (Wallingford procedure)	Z1 = 0.36
Rainfall for 5min storm with 5 year return period	M5_5min _i = Z1 × M5_60min = 6.5 mm
Factor Z2 (Wallingford procedure)	Z2 = 1.02
Rainfall for 5min storm with 5 year return period	M5_5min = Z2 × M5_5min _i = 6.6 mm
Design rainfall intensity	I _{max} = M5_5min / D = 79.5 mm/hr

Maximum surface water runoff

Catchment area	A _{catch} = 8900 m ²
Percentage of area that is impermeable	p = 70 %
Maximum surface water runoff	Q _{max} = A _{catch} × p × I _{max} = 137.7 l/s

 Joule Consulting Engineers Timber Wharf, 40 Worsley Street Manchester M15 4LD	Project				Job no.	
	New Islington Phase 3				284/04	
	Calcs for				Start page no./Revision	
	Existing M30-5				1	
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	SW	13/09/2018				

DESIGN RAINFALL

In accordance with the Wallingford Procedure

Tedds calculation version 2.0.00

Design rainfall intensity

Location of catchment area	Manchester
Storm duration	D = 5 min
Return period	Period = 30 yr
Ratio 60 min to 2 day rainfall of 5 yr return period	r = 0.360
5-year return period rainfall of 60 minutes duration	M5_60min = 18.0 mm
Increase of rainfall intensity due to global warming	p _{climate} = 0 %
Factor Z1 (Wallingford procedure)	Z1 = 0.36
Rainfall for 5min storm with 5 year return period	M5_5min _i = Z1 × M5_60min = 6.5 mm
Factor Z2 (Wallingford procedure)	Z2 = 1.45
Rainfall for 5min storm with 30 year return period	M30_5min = Z2 × M5_5min _i = 9.4 mm
Design rainfall intensity	I _{max} = M30_5min / D = 112.4 mm/hr

Maximum surface water runoff

Catchment area	A _{catch} = 8900 m ²
Percentage of area that is impermeable	p = 70 %
Maximum surface water runoff	Q _{max} = A _{catch} × p × I _{max} = 194.5 l/s

SURFACE 07

 Joule Consulting Engineers Timber Wharf, 40 Worsley Street Manchester M15 4LD	Project New Islington Phase 3				Job no. 284/04	
	Calcs for Existing M100-5				Start page no./Revision 1	
	Calcs by SW	Calcs date 13/09/2018	Checked by	Checked date	Approved by	Approved date

DESIGN RAINFALL**In accordance with the Wallingford Procedure**

Tedds calculation version 2.0.00

Design rainfall intensity

Location of catchment area	Manchester
Storm duration	D = 5 min
Return period	Period = 100 yr
Ratio 60 min to 2 day rainfall of 5 yr return period	r = 0.360
5-year return period rainfall of 60 minutes duration	M5_60min = 18.0 mm
Increase of rainfall intensity due to global warming	p _{climate} = 0 %
Factor Z1 (Wallingford procedure)	Z1 = 0.36
Rainfall for 5min storm with 5 year return period	M5_5min _i = Z1 × M5_60min = 6.5 mm
Factor Z2 (Wallingford procedure)	Z2 = 1.83
Rainfall for 5min storm with 100 year return period	M100_5min = Z2 × M5_5min _i = 11.8 mm
Design rainfall intensity	I _{max} = M100_5min / D = 142.0 mm/hr

Maximum surface water runoff

Catchment area	A _{catch} = 8900 m ²
Percentage of area that is impermeable	p = 70 %
Maximum surface water runoff	Q _{max} = A _{catch} × p × I _{max} = 245.7 l/s

Job No	284/05	Sheet No	SURFACE 08.		
Project	N13	Engineer	SJW	Date	07/20.
Element	SURFACE WATER DRAINAGE		Checked by	Date	

From the previous calculations the existing flow from the site, prior to its clearance to create this development plot is 83.0 l/s.

In accordance with the requirements of the Manchester City, Salford City and Salford Councils Level 2 SFRA a minimum reduction of surface water runoff of 50% with the aim of reducing further where this is reasonably practical, towards the greenfield runoff rate.

The maximum allowable post development flow should therefore be $0.50 \times 83.0 \text{ l/s} = \underline{\underline{41.5 \text{ l/s}}}$.

This would give a theoretical required minimum attenuation storage volume of $= \underline{\underline{170 \text{ m}^3}}$ assuming +40% for climate change.

Following this a further assessment of the site is undertaken, to explore how flows could be reduced still further, towards the greenfield runoff rate.

As shown on the attached sketch, the site is characterised by its topography, approximately level with the nearby canal promenade at one end but falling steeply away at the opposite end, associated with the re-profiling and upfills placed at the time of the recent canal development onto a site that otherwise followed a "valley" profile. This affects

Job No 284/05	Sheet No SURFACE 09	
Project N13	Engineer SJW	Date 07/10
Element SURFACE WATER DRAINAGE.	Checked by	Date

what can be practically achieved given that storage capacity at the low level of the site is limited.

Considering the low level area of the site, Phase 3C and the adjacent road.

As shown on the following sketch, available storage at 3C is limited due to the site arrangement and levels.

The maximum volume that can practically be stored is around 40m^3 .

The connected area from site area 3C and the adjacent road total, $1476 + 310 = \underline{\underline{1786\text{m}^2}}$

this suggests a proposed flow at this point limited to 12L/s .

Considering the high level area of the site, Phase 3A and Phase 3B

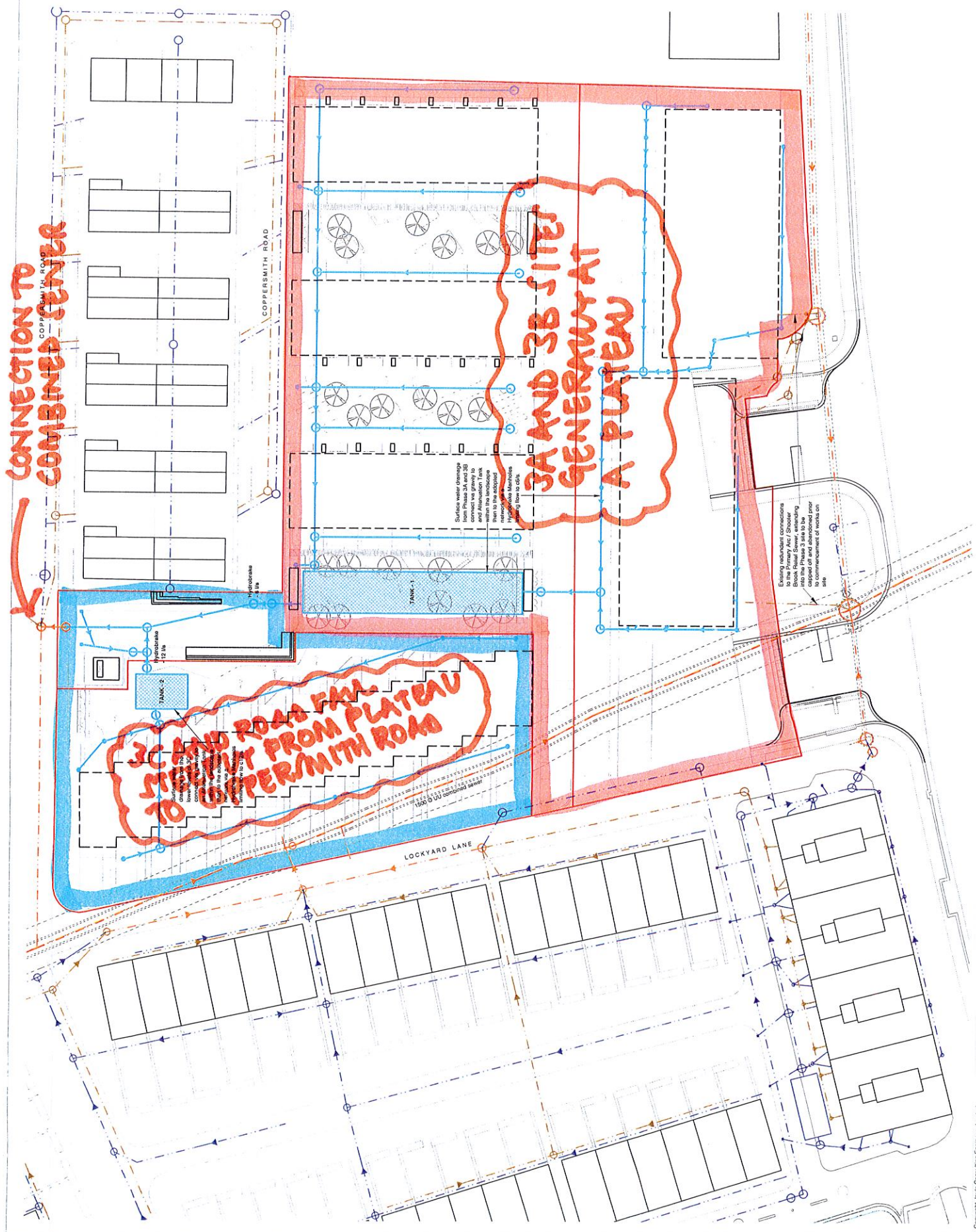
Due to the site topography, available storage in this area is more readily accommodated.

Assuming an allowable flow of 6L/s .

The connected area from site 3A and 3B total, $1639\text{m}^2 + 3590\text{m}^2 = \underline{\underline{5229\text{m}^2}}$

this gives a required minimum storage volume of 285m^3 .

CONNECTION TO
COMBINED SEWER



1. This drawing is to be used in conjunction with the site survey and as a reference for the drainage and the specification.
 2. For general notes refer to drawings no. 204-05-N01 & N02.
 3. For setting out refer to Architects details.
 4. For assumed site built ups over formation level, refer to drawing 204-05-N01 to N12.
- Key
- Phase 3 Site Boundary
 - Existing Drainage
 - Existing Drainage (Foul)
 - Existing Drainage
 - Proposed Drainage
 - Proposed Drainage (Surface Water)
 - Proposed Drainage Alternation (Surface Water)
 - Proposed Drainage (Foul)
 - Proposed Drainage (Combined)

INFORMATION

Project Name	Lockyard Lane
Client	JKS
Location	Newington, Manchester
Scale	1:500
Drawn by	JKS
Checked by	JKS
Approved by	JKS

joule

JKS Consulting Engineers Ltd
101-103, Newington, Manchester M14 6LJ
Tel: 0161 832 3427
Email: info@joule-engineers.co.uk
Website: www.joule-engineers.co.uk

Project 3: Strategic Drainage Overview, Site Plan

Sheet No.	204-05	Scale	SK 300	Drawn by	JKS
Sheet No.	1 500E A3	Scale	07 20	Drawn by	TP

SURFACE 10

Wavin Stormwater Management

Project Details

Project Name New Islington Phase 3 - 3C and Road

Produced By simon waite

Client Name

Site Address , M15 4LD

Date 08/07/2020

Modular Units and Tank Details - Attenuation Storage Tank

Length	6m	6 units	Void	95%
Width	6m	12 units	Cover Depth	0.7m
Depth	1.2m	3 units	Installation Depth	1.9m
Total Number of Units	216 units			

Recommended Unit

Wavin recommend using the following unit for your installation.



AquaCell Core R

Product Number: 6LB100

AquaCell Core has been designed for use in deep applications, subject to both regular and heavy traffic loadings, such as cars and HGV's (for vehicles up to 44 tonnes).

Drainage Details

Area	<u>1786m²</u>	Allowable Discharge	<u>12 ltr/sec</u>
Catchment Type	Roof + Hardstanding	Climate Change	40%
Area Reduction Factor	0.9	Effective Area	2250.36m ²
Soil Type	Silty sandy clay	Factor of Safety	1

Rainfall Data

R Value	0.33	Storm Return Period	1 in 100 years
----------------	------	----------------------------	----------------

M5-60

18 County

Greater Manchester

Time	Z1 Value	y mm	Z2 Value	p mm	Inflow	Outflow	Storage Volume
5min	0.35	6.3	1.81	11.4	25.66	3.6	22.06
10min	0.5	9	1.89	17.01	38.28	7.2	31.08
15min	0.61	10.98	1.91	20.97	47.19	10.8	36.39
30min	0.78	14.04	1.91	26.82	60.35	21.6	38.75
1hr	1	18	1.99	35.82	80.61	43.2	37.41
2hr	1.22	21.96	2.03	44.58	100.32	86.4	13.92
4hr	1.56	28.08	2.01	56.44	127.01	172.8	-45.79
6hr	1.72	30.96	1.97	60.99	137.25	259.2	-121.95
10hr	2	36	1.97	70.92	159.6	432	-272.4
24hr	2.6	46.8	1.89	88.45	199.05	1036.8	-837.75
48hr	3.25	58.5	1.81	105.89	238.28	2073.6	-1835.32

Maximum
volume that
can be stored
at 3C due
to site
constraints.

Critical storm duration (hrs) 30min

Required Storage Height 38.75m

Volume from Dimensions 41.04m³

Note critical storm duration = 30 min.
Note time to drain down = 2hr +

Wavin Stormwater Management

Project Details

Project Name New Islington Phase 3 - 3A and 3B

Produced By simon waite

Client Name

Site Address , M15 4LD

Date 08/07/2020

Modular Units and Tank Details - Attenuation Storage Tank

Length	24m	24 units	Void	95%
Width	8m	16 units	Cover Depth	0.7m
Depth	1.6m	4 units	Installation Depth	2.3m
Total Number of Units	1536 units			

Recommended Unit

Wavin recommend using the following unit for your installation.



AquaCell Core R

Product Number: 6LB100

AquaCell Core has been designed for use in deep applications, subject to both regular and heavy traffic loadings, such as cars and HGV's (for vehicles up to 44 tonnes).

Drainage Details

Area	<u>5229m²</u>	Allowable Discharge	<u>6 ltr/sec</u>
Catchment Type	Roof + Hardstanding	Climate Change	40%
Area Reduction Factor	0.9	Effective Area	6588.54m ²
Soil Type	Silty sandy clay	Factor of Safety	1

Rainfall Data

R Value	0.33	Storm Return Period	1 in 100 years
----------------	------	----------------------------	----------------

M5-60

18 County

Greater Manchester

Time	Z1 Value	y mm	Z2 Value	p mm	Inflow	Outflow	Storage Volume
5min	0.35	6.3	1.81	11.4	75.13	1.8	73.33
10min	0.5	9	1.89	17.01	112.07	3.6	108.47
15min	0.61	10.98	1.91	20.97	138.17	5.4	132.77
30min	0.78	14.04	1.91	26.82	176.68	10.8	165.88
1hr	1	18	1.99	35.82	236	21.6	214.4
2hr	1.22	21.96	2.03	44.58	293.71	43.2	250.51
4hr	1.56	28.08	2.01	56.44	371.86	86.4	285.46
6hr	1.72	30.96	1.97	60.99	401.84	129.6	272.24
10hr	2	36	1.97	70.92	467.26	216	251.26
24hr	2.6	46.8	1.89	88.45	582.77	518.4	64.37
48hr	3.25	58.5	1.81	105.89	697.63	1036.8	-339.17

Critical storm duration (hrs) 4hr

Required Storage Height 285.46m

Volume from Dimensions 291.84m³

NOTE critical storm duration = 4hr
 NOTE time to drain down = 24hr +

Job No	284/05	Sheet No	SURFACE 15		
Project	N13	Engineer	siw	Date	07/20.
Element	SURFACE WATER DRAINAGE.		Checked by	Date	

Overall.

Combining the overall low level and high level flow.

The combined maximum flow is $12 + 6 = \underline{18 \text{ l/s}}$

This compares favourably with the existing flow at 83.0 l/s and represents a reduction of around 79% which is obviously greater than the minimum 50% reduction required by the MTFRA.

From the calculations it is noted that:-
Phase 3C and Adjacent Road.

Critical storm duration for attenuation = 30 minutes.

Time for tank to discharge fully = 120 mins +

Flow at 4 hours = 3.75 l/s .

Flow at 6 hours = 1.40 l/s

Phase 3A and 3B

Critical storm duration for attenuation = 4 hours

Time for tank to discharge fully = 24 hours +

Job No	284/05	Sheet No	SURFACE 16		
Project	N13	Engineer	SJW	Date	07/20
Element	SURFACE WATER DRAINAGE.	Checked by		Date	

In conclusion therefore,

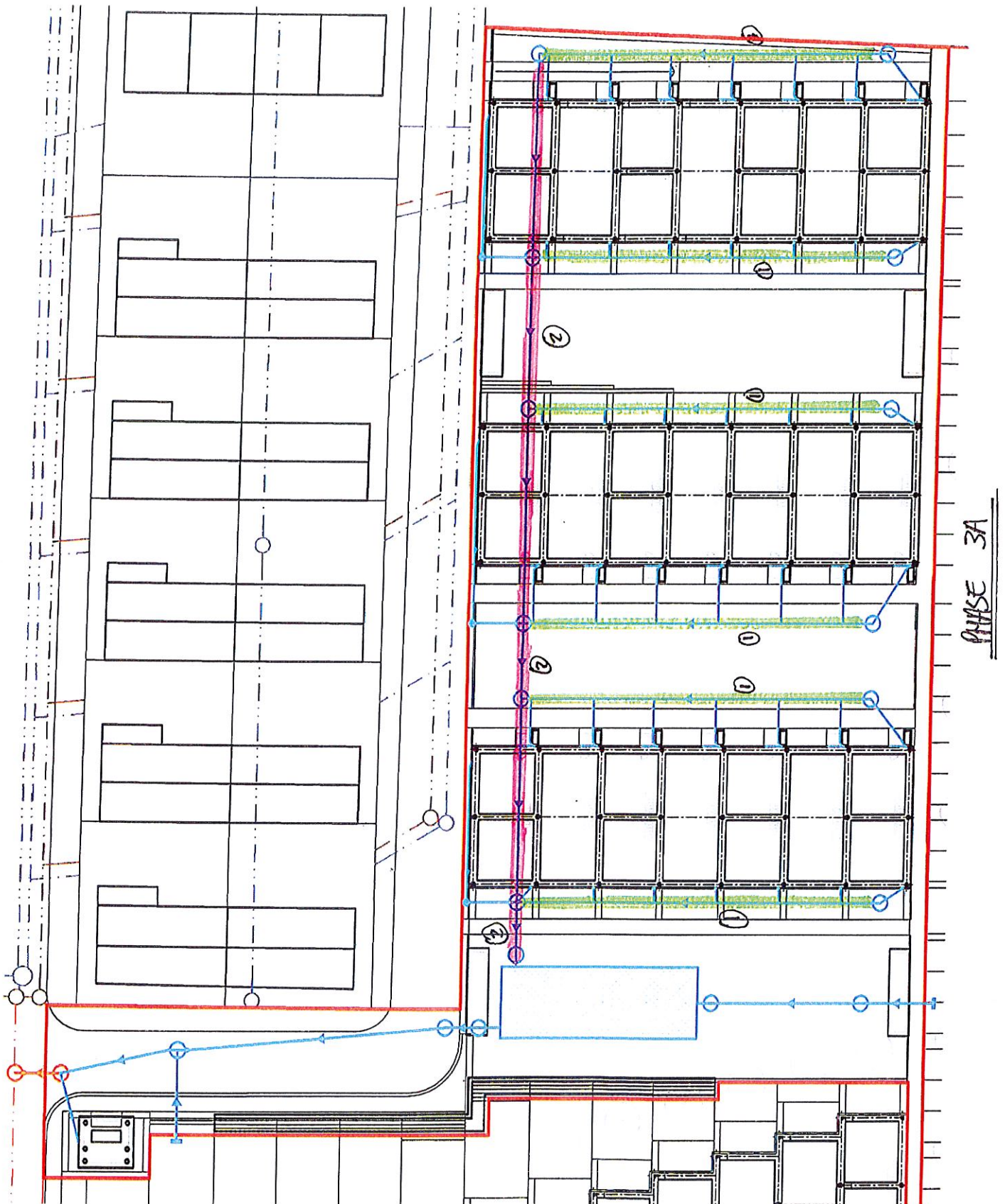
For 100 year storm + 40% climate change.

Allowable flow up to 2 hours = $12.0 + 6.0 = 18 \text{ l/s}$

at 4 hours = $3.75 + 6.0 = 9.75 \text{ l/s}$

at 6 hours = $1.40 + 6.0 = 7.40 \text{ l/s}$

SURFACE 17.



Job No	244-05	Sheet No	SURFACE 18.
Project	N13	Engineer	NR
		Date	13/03/20
Element	SURFACE WATER DRAINAGE	Checked by	Date

PHASE 3A SURFACE WATER DRAINAGE.

- ① Half of 1 No terrace block plus associated handstanding.

$$\text{Area per run (maximum)} = 315 \text{ m}^2$$

$$\text{Runoff} = 0.014 \text{ l/s/m}^2 \quad (\text{From Part H Building Regs})$$

$$\text{Flow rate} = 4.4 \text{ l/s}$$

$$\underline{\text{Use 100mm } \phi \text{ @ 1:80}} \Rightarrow \text{capacity}_{(\text{from Part H})} = 6.5 \text{ l/s} > 4.4 \text{ l/s} \therefore \text{OK.}$$

- ② Full attenuated area of Phase 3A side.

$$\text{Total area} = 1639 \text{ m}^2$$

$$\text{Runoff} = 0.014 \text{ l/s/m}^2$$

$$\text{Flow rate} = 0.014 \times 1639 = 23.0 \text{ l/s}$$

$$\text{Capacity of 150mm } \phi \text{ at 1:80} = 19.8 \text{ l/s} \quad (\text{Tedds calc})$$

$$\text{Capacity of 225mm } \phi \text{ at 1:80} = 54.0 \text{ l/s} \quad (\text{Tedds calc})$$

- Use 150mm ϕ at 1:80 up to penultimate chamber.

$$\hookrightarrow \text{flow} = 23.0 - 4.4 = 18.6 \text{ l/s} \leq 19.8 \text{ l/s} \therefore \text{OK.}$$

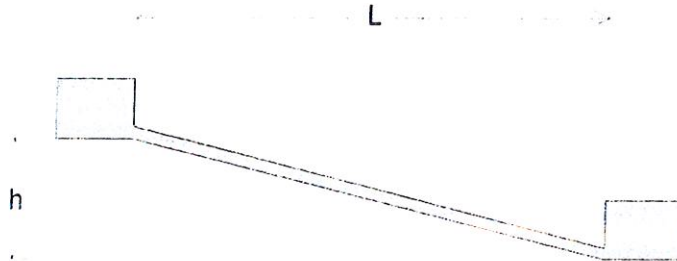
- Use 225mm ϕ at 1:80 between penultimate chamber and last chamber tank.

$$\hookrightarrow \text{flow} = 23.0 \text{ l/s} \leq 54.0 \text{ l/s} \therefore \text{OK}$$

joule Joule Consulting Engineers Ltd Timber Wharf 40 Worsley Street Manchester, M15 4LD	Project		New Islington - Phase 3A		Job no.		384-05	
	Calcs for		150mm Pipe at 1 in 80		Start page no./Revision			
	Calcs by		Calcs date		Checked by		Checked date	
	NR		13/03/2020					
					Approved by		Approved date	

DESIGN OF A SURFACE WATER DRAIN

TEDDS calculation version 1.0.04



Drain design details

Design flow rate $Q_{\text{design}} = 10.0 \times 10^{-3} \text{ m}^3/\text{s}$
 Length of the drain $L = 80.0 \text{ m}$
 Fall along length of drain $h = 1.000 \text{ m}$
 Gradient of drain $i = h / L = 0.013 \text{ (1 in 80)}$

(Library item Drain design details)

Minimum pipe diameter $D_{\text{min}} = 150 \text{ mm}$
 Surface roughness $k_s = 0.6 \text{ mm}$
 Mean hydraulic depth factor $m = 0.25$
 Kinematic viscosity of fluid $\nu = 1.31 \times 10^{-6} \text{ m}^2/\text{s}$

Using the Chezy equation

Constant $c = 56$
 Diameter of pipe required $D = ((Q_{\text{design}}^2 \times 16) / (\pi^2 \times m \times c^2 \times i \times 1 \text{ m/s}^2))^{0.2} = 111 \text{ mm}$
 Nearest pipe diameter $D_{\text{chezy}} = 150 \text{ mm}$
 Flow velocity using Chezy $V_{\text{chezy}} = c \times \sqrt{(m \times D_{\text{chezy}} \times i \times 1 \text{ m/s}^2)} = 1.212 \text{ m/s}$

Using the Eschitt equation

Diameter of pipe required $D = (Q_{\text{design}} \times 1000 \times \sqrt{(1/i)} / 0.00035 \text{ m}^3/\text{s})^{0.382} \times 1 \text{ mm} = 116 \text{ mm}$
 Nearest pipe diameter $D_{\text{eschitt}} = 150 \text{ mm}$
 Flow velocity using Eschitt $V_{\text{eschitt}} = 26.738 \times (D_{\text{eschitt}} / 1 \text{ mm})^{0.62} \times 1 \text{ m/s} / (\sqrt{(1/i)} \times 60) = 1.113 \text{ m/s}$

Using the Colebrook-White Equation for pipe running full and partially full

Design pipe diameter $D_{\text{design}} = \max(D_{\text{chezy}}, D_{\text{eschitt}}, D_{\text{min}}) = 150 \text{ mm}$
 Constant $Z = \sqrt{(2 \times (g_{\text{acc}} / 1 \text{ m/s}^2) \times (D_{\text{design}} / 1000 \text{ mm}) \times i)} = 0.192$
 Flow velocity $V_{\text{full}} = -2 \times Z \times \log((k_s / (3.7 \times D_{\text{design}})) + ((2.51 \times \nu) / (D_{\text{design}} \times Z \times 1 \text{ m/s}))) \times 1 \text{ m/s}$
 $V_{\text{full}} = 1.121 \text{ m/s}$
 Flow rate running full $Q_{\text{full}} = V_{\text{full}} \times \pi \times D_{\text{design}}^2 / 4 = 19.8 \times 10^{-3} \text{ m}^3/\text{s}$

PASS - Maximum flow rate is greater than design flow rate

From Hydraulics Research Tables 35 and 36

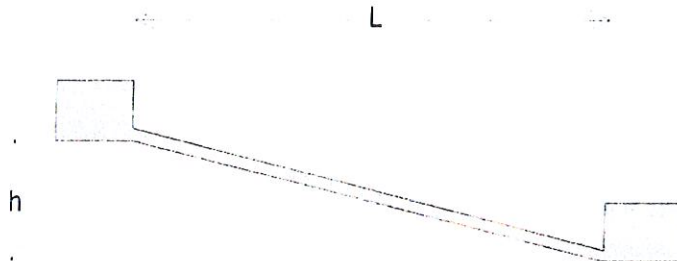
Depth as proportion of D $x = 0.503$
 Flow velocity at design flow rate $V_{\text{design}} = 1.123 \text{ m/s}$

joule Joule Consulting Engineers Ltd Timber Wharf 40 Worsley Street Manchester, M15 4LD	Project		New Islington - Phase 3A		Job no.		384-05	
	Calcs for		225mm Pipe at 1 in 80		Start page no./Revision			
	Calcs by		Calcs date		Checked by		Checked date	
	NR		13/03/2020					
					Approved by		Approved date	

SURFACE 10

DESIGN OF A SURFACE WATER DRAIN

TEDDS calculation version 1.0.04



Drain design details

Design flow rate $Q_{\text{design}} = 10.0 \times 10^{-3} \text{ m}^3/\text{s}$
 Length of the drain $L = 80.0 \text{ m}$
 Fall along length of drain $h = 1.000 \text{ m}$
 Gradient of drain $i = h / L = 0.013 \text{ (1 in 80)}$

(Library item Drain design details)

Minimum pipe diameter $D_{\text{min}} = 225 \text{ mm}$
 Surface roughness $k_s = 0.6 \text{ mm}$
 Mean hydraulic depth factor $m = 0.25$
 Kinematic viscosity of fluid $\nu = 1.31 \times 10^{-6} \text{ m}^2/\text{s}$

Using the Chezy equation

Constant $c = 56$
 Diameter of pipe required $D = ((Q_{\text{design}}^2 \times 16) / (\pi^2 \times m \times c^2 \times i \times 1 \text{ m/s}^2))^{0.2} = 111 \text{ mm}$
 Nearest pipe diameter $D_{\text{chezy}} = 225 \text{ mm}$
 Flow velocity using Chezy $V_{\text{chezy}} = c \times \sqrt{(m \times D_{\text{chezy}} \times i \times 1 \text{ m/s}^2)} = 1.485 \text{ m/s}$

Using the Eschritt equation

Diameter of pipe required $D = (Q_{\text{design}} \times 1000 \times \sqrt{(1 / i) / 0.00035 \text{ m}^3/\text{s}})^{0.382} \times 1 \text{ mm} = 116 \text{ mm}$
 Nearest pipe diameter $D_{\text{eschritt}} = 225 \text{ mm}$
 Flow velocity using Eschritt $V_{\text{eschritt}} = 26.738 \times (D_{\text{eschritt}} / 1 \text{ mm})^{0.62} \times 1 \text{ m/s} / (\sqrt{(1 / i) \times 60}) = 1.431 \text{ m/s}$

Using the Colebrook-White Equation for pipe running full and partially full

Design pipe diameter $D_{\text{design}} = \max(D_{\text{chezy}}, D_{\text{eschritt}}, D_{\text{min}}) = 225 \text{ mm}$
 Constant $Z = \sqrt{(2 \times (g_{\text{acc}} / 1 \text{ m/s}^2) \times (D_{\text{design}} / 1000 \text{ mm}) \times i)} = 0.235$
 Flow velocity $V_{\text{full}} = -2 \times Z \times \log((k_s / (3.7 \times D_{\text{design}})) + ((2.51 \times \nu) / (D_{\text{design}} \times Z \times 1 \text{ m/s}))) \times 1 \text{ m/s}$
 $V_{\text{full}} = 1.459 \text{ m/s}$
 Flow rate running full $Q_{\text{full}} = V_{\text{full}} \times \pi \times D_{\text{design}}^2 / 4 = 58.0 \times 10^{-3} \text{ m}^3/\text{s}$

PASS - Maximum flow rate is greater than design flow rate

From Hydraulics Research Tables 35 and 36

Depth as proportion of D $x = 0.280$

Flow velocity at design flow rate $V_{\text{design}} = 1.095 \text{ m/s}$

Job No	284-05	Sheet No	SURFACE 21
Project	N13	Engineer	NR
		Date	13/03/20
Element	SURFACE WATER DRAINAGE	Checked by	Date

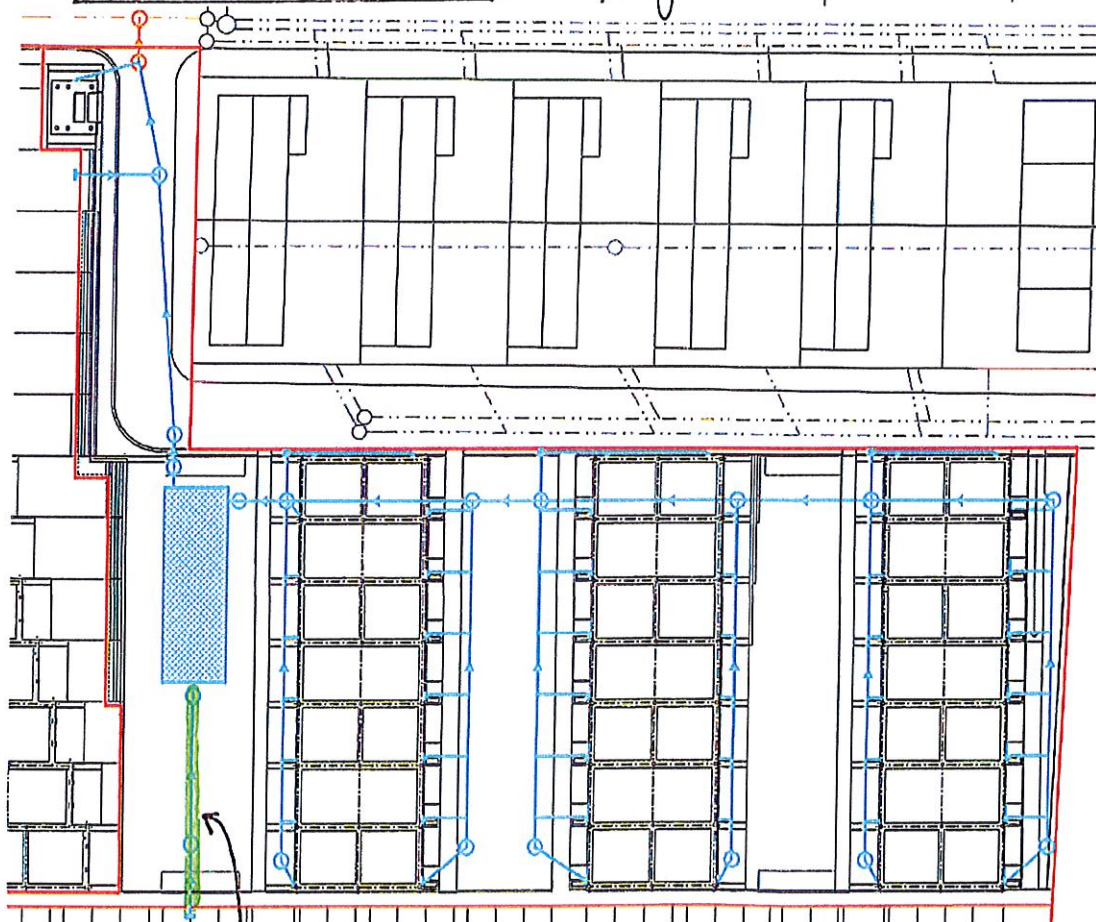
SURFACE WATER FLOW FROM SITE 3B

Site area = 3590 m^2

Runoff = 0.014 l/s/m^2 (from Part H)

Flow rate = $3590 \times 0.014 = 50.3 \text{ l/s}$

Use $225 \text{ mm } \phi @ 1:80 \Rightarrow \text{capacity} = 58.0 \text{ l/s} \gg 50.3 \text{ l/s}$



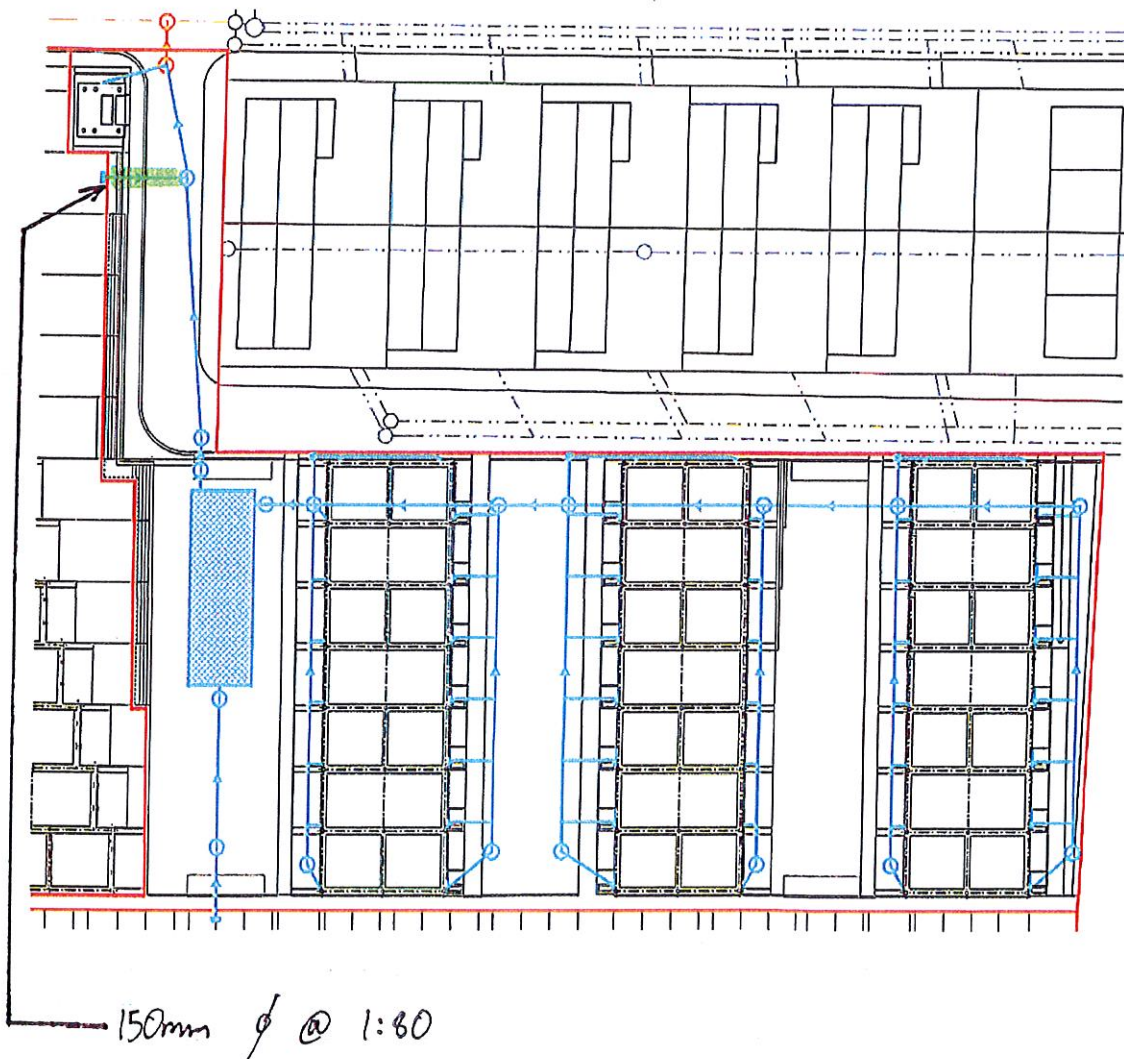
225mm ϕ at 1:80

Job No	244-05	Sheet No	SURFACE 22		
Project	N13	Engineer	NR	Date	13/03/20
Element	SURFACE WATER DRAINAGE.		Checked by	Date	

SURFACE WATER FROM PHASE 3C

From attenuation tank size, surface water from 3C restricted to 12 l/s

Use 150mm ϕ pipe @ 1:40 $\Rightarrow$ capacity = 20 l/s.

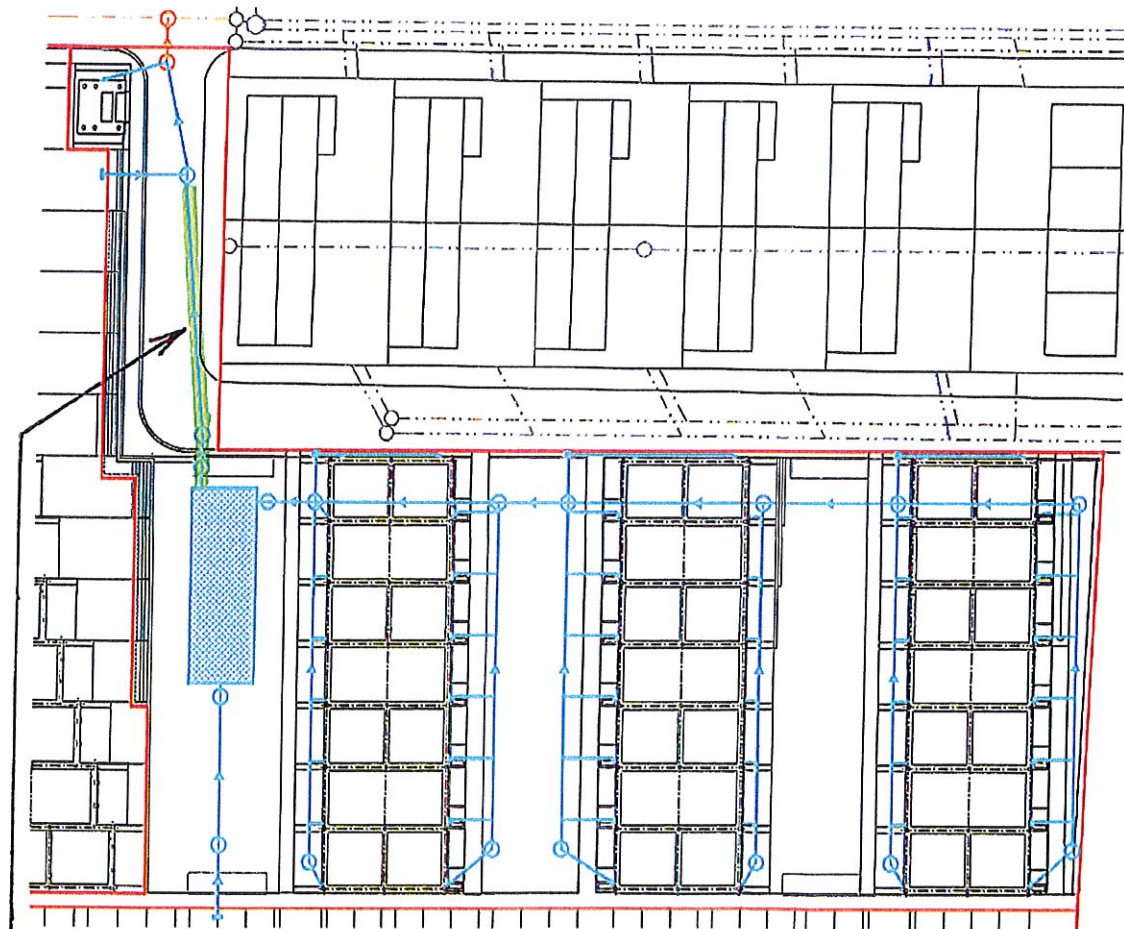


Job No	284-05	Sheet No	SURFACE 23	
Project	N13	Engineer	NR	Date 13/03/20
Element	SURFACE WATER DRAINAGE.		Checked by	Date

ATTENUATED SURFACE WATER FROM 3A & 3B

From tank calc, discharge from tank = 6 L/s

use 150 ϕ pipe at 1 in 80 $\Rightarrow$ capacity = 20 L/s



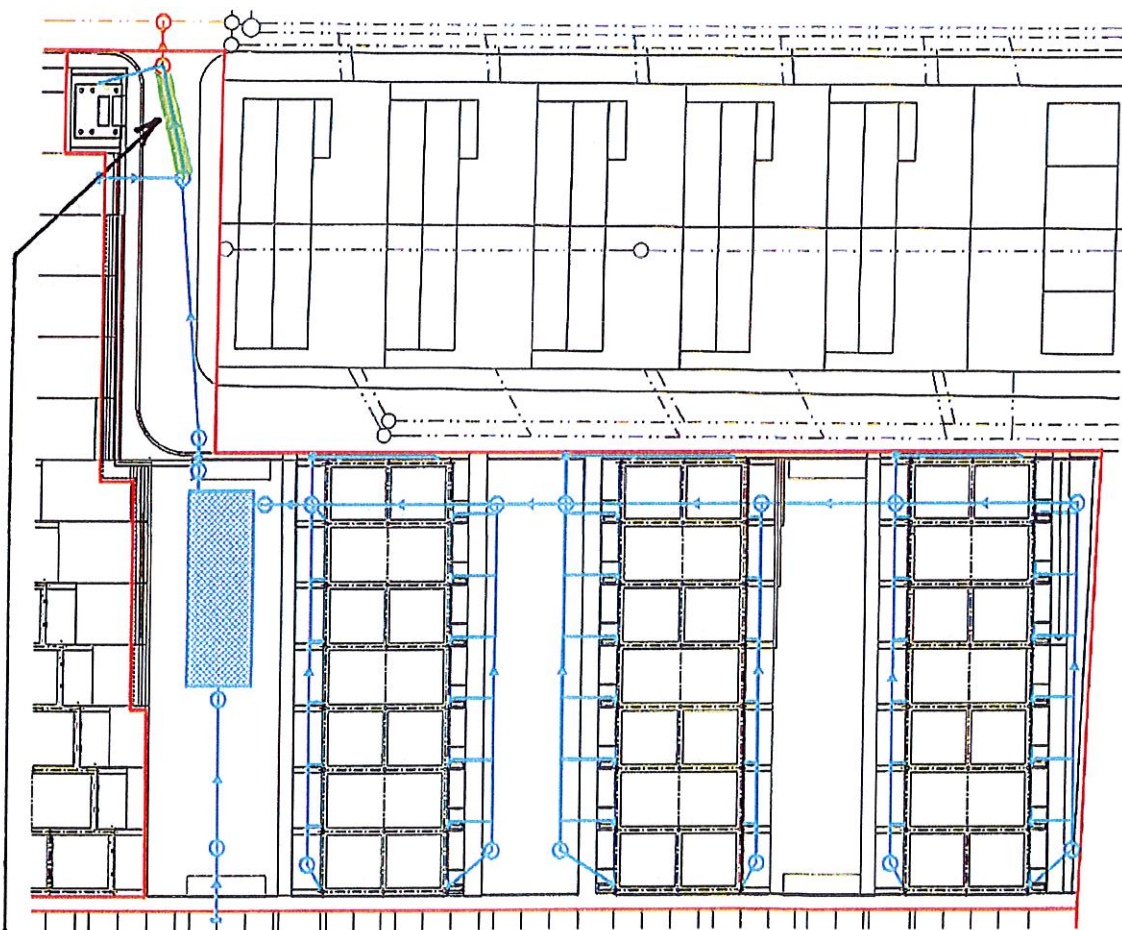
225mm ϕ at 1 in 80

Job No	284-05	Sheet No	SURFACE 2A		
Project	N13	Engineer	NR	Date	13/03/20
Element	SURFACE WATER DRAINAGE		Checked by	Date	

SURFACE WATER FROM 3A, 3B + 3C

Total discharge of surface water = 18 L/s

Use 225mm ϕ pipe at 1 in 80 $\Rightarrow$ capacity 58.0 L/s



225mm ϕ @ 1 in 80.