

DRAINAGE STRATEGY

PROPERTY ADDRESS

Victoria Mill
10 Lower Vickers Street
Manchester
M40 7LJ

CLIENT

Report Prepared for
Virtu Homes Ltd

REPORT REFERENCE

23-/SWH

DATE

October 2023

1. Introduction

- 1.1.** ADS structural have been appointed to prepare a drainage strategy in support of a proposed residential development of a section of the ground floor at Victoria Mill, 10 Lower Vickers Street, Manchester, M40 7LJ.
- 1.2.** The application site is rectangular in shape and was previously noted as a previously re-developed mill conversion.
- 1.3.** The national Grid reference for the site is 385879,399235 and the nearest postcode to the site is M40 7LJ.
- 1.4.** The site extends to approximately 3195m² and is a brownfield site.
- 1.5.** The scheme is for the conversion of part of the lower ground floor into residential apartments along with associated infrastructure.

2. Proposed Drainage Strategy

- 2.1.** In order to demonstrate the development is of low risk the report should show:
 - That it will be feasible to balance surface water run-off to the existing run-off rate in terms of a brownfield development for all events up to the 1 in 100-year storm (including a 45% allowance for climate change) and set out how this will be achieved
 - What sustainable drainage techniques (SuDS) will be used with any obstacles to their use clearly justified
- 2.2.** The primary flood risk generated by the new development is most likely to be the risk posed to others by surface water run-off. The proposed development will marginally increase the amount of hardstanding across the site which will therefore generate similar surface water run-off rates/volumes which may need to be attenuated within the site boundary.
- 2.3.** The surface water drainage developments for any development site should aspire to be such that the volumes and peak flow rates of surface water leaving a developed site are no greater than the rates prior to the development.
- 2.4.** For a new development it may be necessary to provide surface water storage and/or infiltration to limit and reduce both the peak rate of discharge from the site and the total volume discharged from the site.
- 2.5.** Climate change (CC) will be considered by increasing the rainfall intensity by 40% in line with the Environment Agency and NPPF Technical Guidance.

3. Existing Site Drainage

- 3.1.** The site is currently unused and has previously occupied by an existing converted mill. There is an existing combined drain located just outside of the curtilage of the dwelling, ultimately out falling into the adjacent adopted drain.

It should also be noted that there is an existing canal and overflow/bypass surface water culvert which runs along the southern boundary of the site, ultimately out falling into a larger diameter culvert following the western boundary of the site.

- 3.2.** Foul drainage is to be discharged via a new wastewater treatment plant on site.
- 3.3.** The existing drainage details can be found in Appendix A

4. Consideration of the Sustainable Drainage Systems

- 4.1.** Following the Hierarchy of Sustainable Drainage Systems (SUDS) the following options must be considered.

- Soakways for roof run-off and permeable surfacing for roads, drives, car parks and private access roads
- Discharge to local watercourse
- Discharge to local sewer
- Discharge to adjacent land drains and ponds

- 4.2.** The first option to be considered must be infiltration into the ground. Even when there are alternative sewer connections or water courses available, infiltration must be utilised unless it is unfeasible.

A site investigation carried out by LK on the site revealed that the sub-soils were noted as potentially permeable sands however given that the previous site was utilised as a petrol filling station we would advise that infiltration drainage could potentially effect the underlying aquifer and as such we would rule out the use of this type of drainage system

- 4.3.** The proposed storage would be catered for within the oversized pipes to cater for storms up to and including the 1 in 100 yr. 6-hour storm event +45% CCF.

5. Proposed Drainage Design

- 5.1.** The solution being considered to provide the attenuation via the use of a crate system located in the rear car park area water pipes within the system followed by an orifice plate on the outfall manhole to restrict the flows.
- 5.2.** The design seeks to utilise a traditional piped drainage system. Whilst not the most sustainable solution, at this stage it is one that we are confident could be delivered To enable completion of the works
- 5.3.** The outline drawing shows the surface & foul water on separate systems. The surface water flows will be directed to the culverted water course running along the southern boundary. Foul water runs will discharge to a new waste water treatment plant.
- 5.4.** In calculating the run-off at this stage, to ensure robustness, we have assumed the following:
- No rainwater harvesting / Water butts.
- 5.5.** In order to ensure a robust approach, it has been assumed that 100% of the rainwater falling on the proposed impermeable areas for the roof and parking areas only to enter the system.
- 5.6.** Below ground storage will be provided up to and including the 1 in 100-year storm event plus an allowance of 45% increase for climate change.
- 5.7.** Consultation has be made with United Utilities to discuss the implications of connecting into their system.

6. Summary

- The development is currently a brownfield site.
- Surface water run-off from the roof and parking area within the proposed development will be via an attenuated system discharging into the existing culverted water course.
- Storage for the impermeable areas will be provided up to and including the 1 in 100 year + 40% climate change events within the site boundary. The attenuation will be in the form of storage crates located in the rear of the carpark.
- The drainage proposals will be designed in accordance with the techniques set out within the SUDS manual.

7. Conclusion

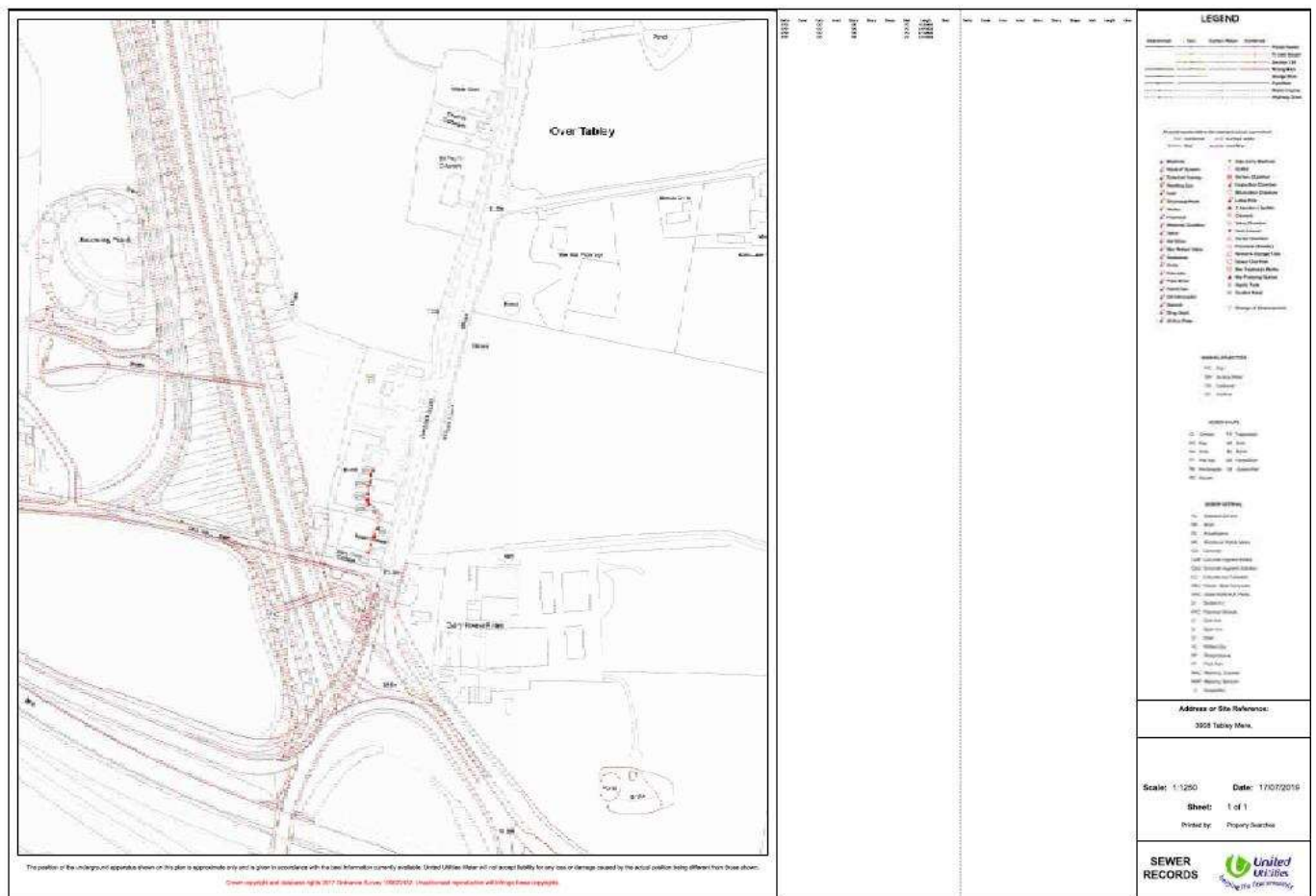
- The drainage strategy presented has investigated the Suds hierarchy in relation to preferred drainage discharge and it can be confirmed that connection to the existing culverted watercourse is the only preferred option give the potential for contamination within the underlying sub-soils
- The on-site drainage has been split between foul and surface water runs, combining into the existing combined manhole as it exits the site. This will allow a later discharge to a separate system when available.

Report Prepared By: S W Hornby

A handwritten signature in black ink, appearing to read 'S W Hornby'.

For and on behalf of
ADS Structural Limited.

APPENDIX A – EXISTING DRAINAGE LAYOUT



APPENDIX B- ATTENUATION DESIGN.

ATTENUATION DESIGN

Attenuation design in accordance with CIRIA publication C753 - The SUDS Manual

Tedds calculation version 1.0.04

EA_Defra method

Site characteristics

Location;	Manchester
Hydrological region;	10
Soil type (Wallingford Procedure W.R.A.P map);	4
Standard percentage runoff;	SPR = 0.47
Average annual rainfall;	SAAR = 880 mm
5 year return period rainfall of 60 minute duration;	M5_60min = 18.0 mm
Ratio 60-minute to 2 day rainfalls of 5 year return;	r = 0.36
Rainfall intensity increase due to global warming;	p _{climate} = 30 %
Impervious area req. attenuation storage;	α = 100.0 %

Catchment details

Subcatchment	Name	Area (ha)	PIMP (%);	Impermeable. area (ha)
1;	SITE;	0.10;	80.0;	0.08;
Total		0.10;	80.0;	0.08;

Greenfield runoff rates

Catchment area;	AREA = 50.00 hectare
Greenfield runoff rate (50 hectare site);	$\bar{Q}_{\text{rural}} = 0.00108 \text{ m}^3/\text{s} \times (\text{AREA}/1 \text{ km}^2)^{0.89} \times (\text{SAAR}/1 \text{ mm})^{1.17} \times \text{SPR}^{2.17} = \mathbf{315.5 \text{ l/s}}$
Greenfield runoff rate;	$\bar{Q} = \bar{Q}_{\text{rural}} / \text{AREA} \times A = \mathbf{0.6 \text{ l/s}}$
Greenfield runoff rate per unit area;	$\bar{Q}_A = \bar{Q} / A = \mathbf{6.3 \text{ l/s/ha}}$

Estimated site discharges

FSR growth rate (1 year);	FSR _{1yr} = 0.87
Discharge (1 year);	Q _{1yr} = $\bar{Q} \times \text{FSR}_{1\text{yr}} = \mathbf{0.5 \text{ l/s}}$
FSR growth rate (10 year);	FSR _{10yr} = 1.38
Discharge (10 year);	Q _{10yr} = $\bar{Q} \times \text{FSR}_{10\text{yr}} = \mathbf{0.9 \text{ l/s}}$
FSR growth rate (100 year);	FSR _{100yr} = 2.08
Discharge (100 year);	Q _{100yr} = $\bar{Q} \times \text{FSR}_{100\text{yr}} = \mathbf{1.3 \text{ l/s}}$

Estimated attenuation volume - 1 year

Attenuation storage vol (fig A7.1 - A7.8);	Uvol _{1yr} = 86.5 m ³ / hectare
Basic storage volume ;	BSV _{1yr} = Uvol _{1yr} × α × A = 8.65 m ³
FEH rainfall factor (figs A11.1, A6.1.1 - A6.3.4);	FF _{1yr} = 1.00
Storage volume ratio (fig A8.1 - A8.8);	SVR _{1yr} = 1.43
Adjusted storage volume;	ASV _{1yr} = SVR _{1yr} × BSV _{1yr} = 12.35 m ³
Hydrological regional volume ratio (fig A9.1);	HR _{1yr} = 1.00
Final estimated attenuation storage;	Vol _{1yr} = HR _{1yr} × ASV _{1yr} = 12.37 m ³

Library item: Estimated attenuation output

Estimated attenuation volume - 10 year

Attenuation storage vol (fig A7.1 - A7.8);	Uvol _{10yr} = 133.7 m ³ / hectare
Basic storage volume ;	BSV _{10yr} = Uvol _{10yr} × α × A = 13.37 m ³

FEH rainfall factor (figs A11.1, A6.1.1 - A6.3.4);
Storage volume ratio (fig A8.1 - A8.8);
Adjusted storage volume;
Hydrological regional volume ratio (fig A9.1);
Final estimated attenuation storage;

$$\begin{aligned}FF_{10yr} &= \mathbf{1.00} \\SVR_{10yr} &= \mathbf{1.43} \\ASV_{10yr} &= SVR_{10yr} \times BSV_{10yr} = \mathbf{19.11 \text{ m}^3} \\HR_{10yr} &= \mathbf{1.05} \\Vol_{10yr} &= HR_{10yr} \times ASV_{10yr} = \mathbf{20.04 \text{ m}^3}\end{aligned}$$

Library item: Estimated attenuation output

Estimated attenuation volume - 100 year

Attenuation storage vol (fig A7.1 - A7.8);
Basic storage volume ;
FEH rainfall factor (figs A11.1, A6.1.1 - A6.3.4);
Storage volume ratio (fig A8.1 - A8.8);
Adjusted storage volume;
Hydrological regional volume ratio (fig A9.1);
Final estimated attenuation storage;

$$\begin{aligned}Uvol_{100yr} &= \mathbf{270.0 \text{ m}^3 / \text{hectare}} \\BSV_{100yr} &= Uvol_{100yr} \times \alpha \times A = \mathbf{27.00 \text{ m}^3} \\FF_{100yr} &= \mathbf{0.90} \\SVR_{100yr} &= \mathbf{1.66} \\ASV_{100yr} &= SVR_{100yr} \times BSV_{100yr} = \mathbf{44.83 \text{ m}^3} \\HR_{100yr} &= \mathbf{1.14} \\Vol_{100yr} &= HR_{100yr} \times ASV_{100yr} = \mathbf{51.09 \text{ m}^3}\end{aligned}$$

Library item: Estimated attenuation output

Attenuation storage required

Vol. increase due to head-discharge relationship;
Maximum attenuation storage required;

$$\begin{aligned}p_{hydro} &= \mathbf{1.25} \\V_{req_max} &= Vol_{10yr} \times p_{hydro} = \mathbf{25.0 \text{ m}^3}\end{aligned}$$

Interception storage

Interception rainfall depth;
Volume of interception storage required;

$$\begin{aligned}d_{int} &= \mathbf{5 \text{ mm}} \\V_{int_req} &= 0.8 \times A_{imp} \times d_{int} = \mathbf{3.20 \text{ m}^3}\end{aligned}$$

Long term storage

Proportion of paved area draining in to network;

$$\alpha = \mathbf{1.0}$$

Proportion of pervious area draining in to network;

$$\beta = \mathbf{0.5}$$

Rainfall depth for 100years, 6 hour event;

$$RD = M100_360 = \mathbf{74.1 \text{ mm}}$$

Extra runoff vol of dev.runoff over greenfield runoff;
 $Vol_{xs} = \max(RD \times A \times (PIMP \times \alpha \times 0.8 + ((1 - PIMP) \times \beta \times SPR) - SPR), 0 \text{ m}^3) = \mathbf{16.09 \text{ m}^3}$

Treatment volume

Treatment volume (assume 80% runoff);

$$T_{vol} = 0.8 \times A \times 15\text{mm} \times PIMP = \mathbf{9.60 \text{ m}^3}$$