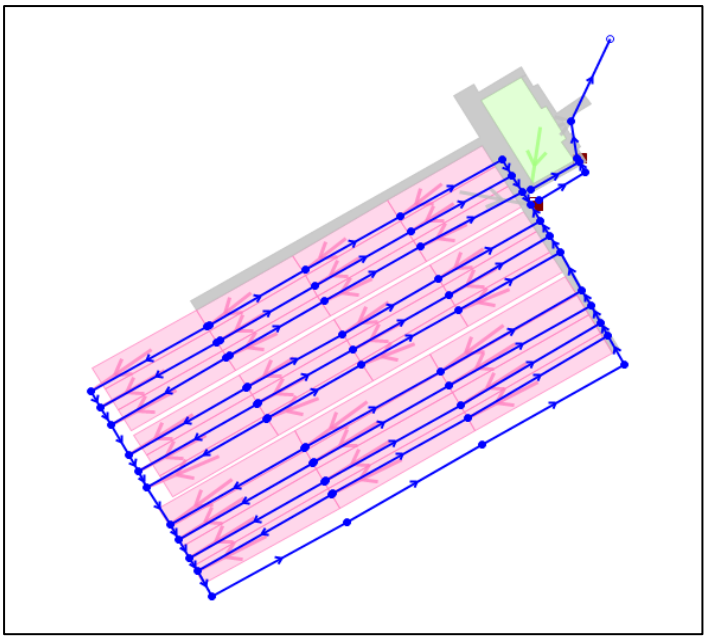
caleywater

Surface Water Drainage Assessment for Goals 3G, Manchester



Goals 3G, Manchester Development Site

January 2022

Report No. 800100_34_DG_158

Surface Water Drainage Assessment for Goals 3G, Manchester

Goals 3G, Manchester

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1. Introduction

Caley Water was instructed in December 2021 to carry out modelling works for the storm water network on a site at Goals 3G, Manchester. The purpose of this modelling exercise was to determine the additional attenuation volume required for a 1 in 100-year storm event with Climate Change (CC) when a further 3 pitches are added to the development.

In order to do this, 1 in 100 year + CC storm events were run to determine the maximum predicted flood volume, thus allowing a minimum attenuation volume to be calculated.

The model was developed using InfoWorks ICM2021 and was created from AutoCAD drawings provided by the client (FT Consulting).

This report provides a detailed methodology and overview of the results of the simulations completed. The report also aims to provide the optimal drainage strategy for the additional pitches.

2. Scope

The following steps were carried out for this project:

1. Build InfoWorks ICM model of proposed drainage network with run off areas
2. Prepare appropriate hydrology for range of storm events
3. Run simulations and extract results
4. Determine storm attenuation required
5. Create brief report and provide maintenance recommendations

3. Resources

FT Consulting provided Caley Water with the following documents:

- AutoCAD drawings of the drainage layout:

  Drainage Survey 3rd June.dwg

- As built drawings:

  A101 Existing Layout.pdf

  A102 Existing Layout.pdf

  A103 Details 1.pdf

  A104 Details 2.pdf

  A105 Details 3.pdf

  A106 Proposed Layout.pdf

  A107 Proposed Layout.pdf

4. Model Build Methodology

The section below details the steps taken to create the model and rainfall data for this project. Two scenarios have been created, the existing model scenario and the extended model scenario. The baseline scenario replicates existing drainage that is currently in place, whereas the future model scenario accounts for the additional 3 pitches and the proposed drainage route for these pitches).

4.1. Creating the Baseline Model Scenario

The following describes the steps taken to create the baseline model. The baseline model represents the 'existing' sewer arrangement which consists of drainage for the main building, the pathways around this building and 8 football pitches (of which 6 are 5-a-side and 2 are 7-a-side). Figure 1 below provides an overview of this existing network.

For more a more detailed view of the existing drainage network, please refer to Appendix 1 of this document.

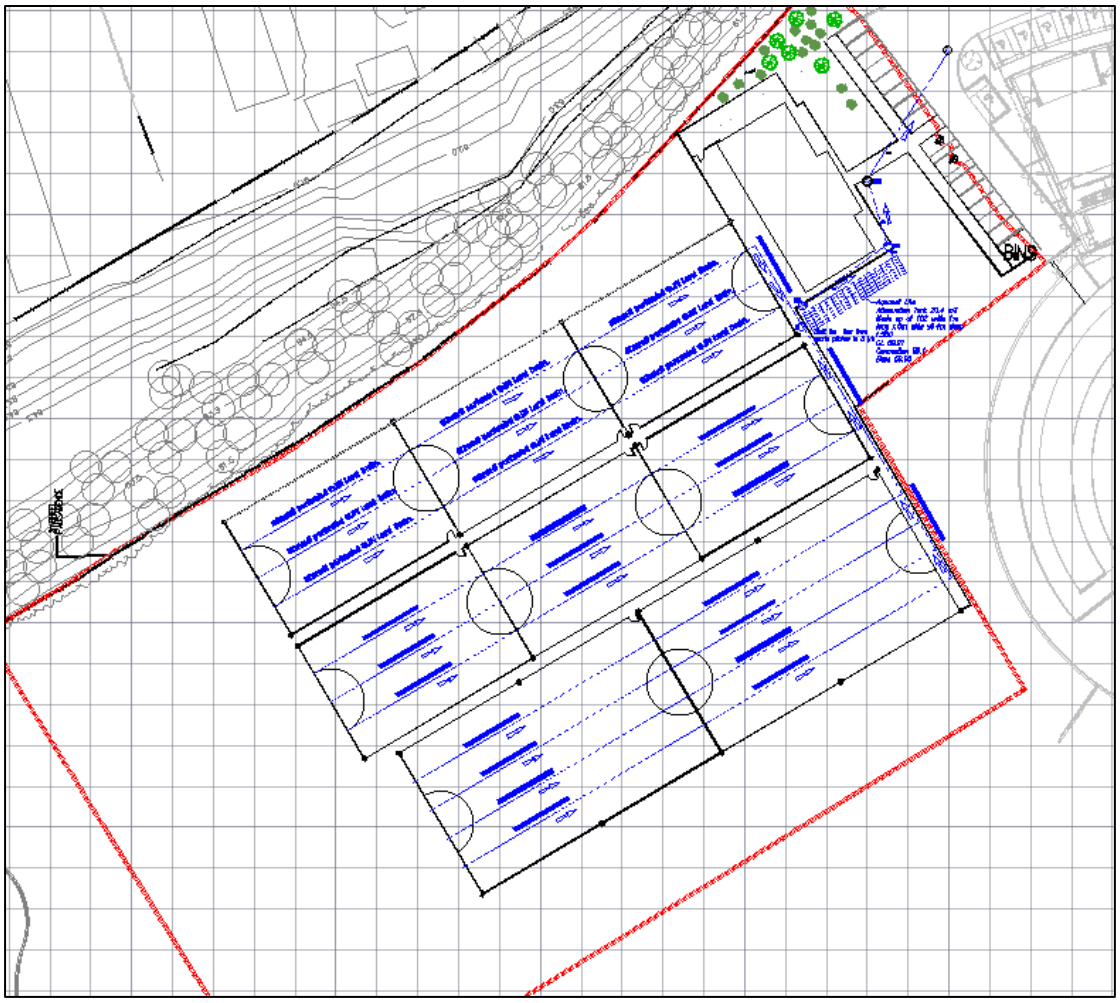


Figure 1 - Design drawing showing the existing drainage network

4.1.1. Storm Water Network

The as built drawings were used to infer the position of the 80mm sewers serving the pitches. Some cover and invert levels were provided in A103 Details 1. Alongside the conduit gradients and diameters, these were applied manually to the model. DSM data was provided for the site, which was used to infer average cover levels for the three pitches.

The information provided does not confirm pipe depth or gradient for the 3 pipes that run the length of each pitch. Assumptions have therefore been made to effectively drain flows from the upstream of the pipes towards the storage tank in the eastern part of the site. These assumptions will directly impact the overall drainage capacity for each pitch.

A storage tank providing 20.4m^3 of storage is located downstream of the pitches. This has been modelled with a rectangular pipe with the relevant dimensions taken from the drawing to provide the adequate amount of storage.

Figures 2 to 4 show the model layout of the existing drainage system.

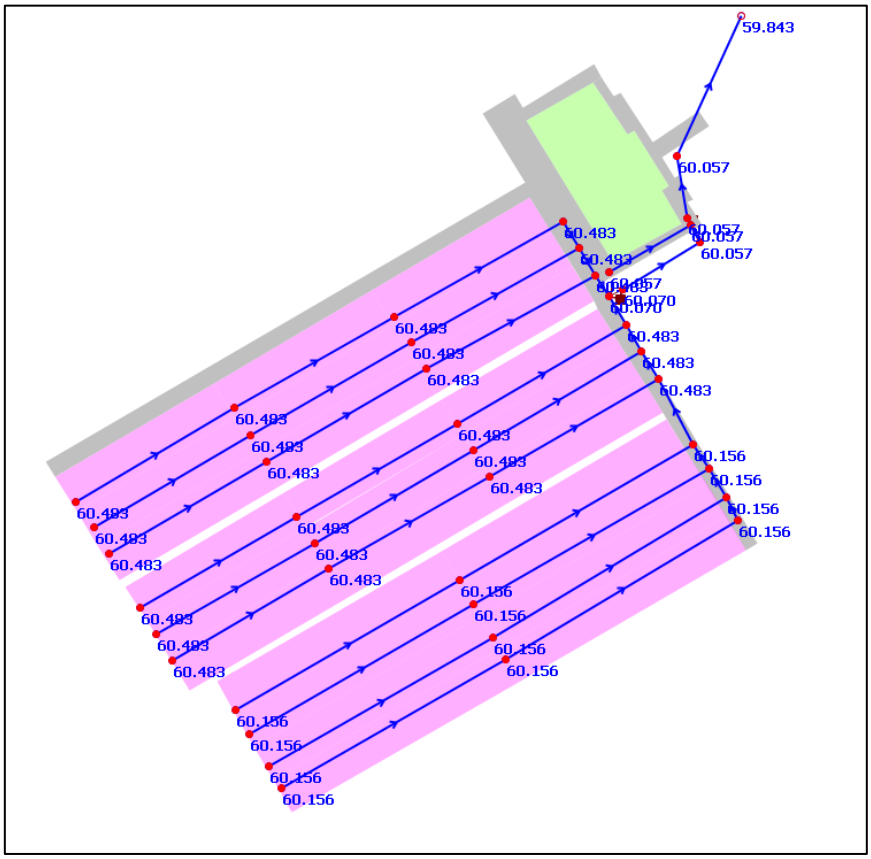


Figure 2 – Overview of Overall Modelled Drainage Layout for the Goals 3G site with cover levels labelled (note that the cover levels have been inferred from DSM)

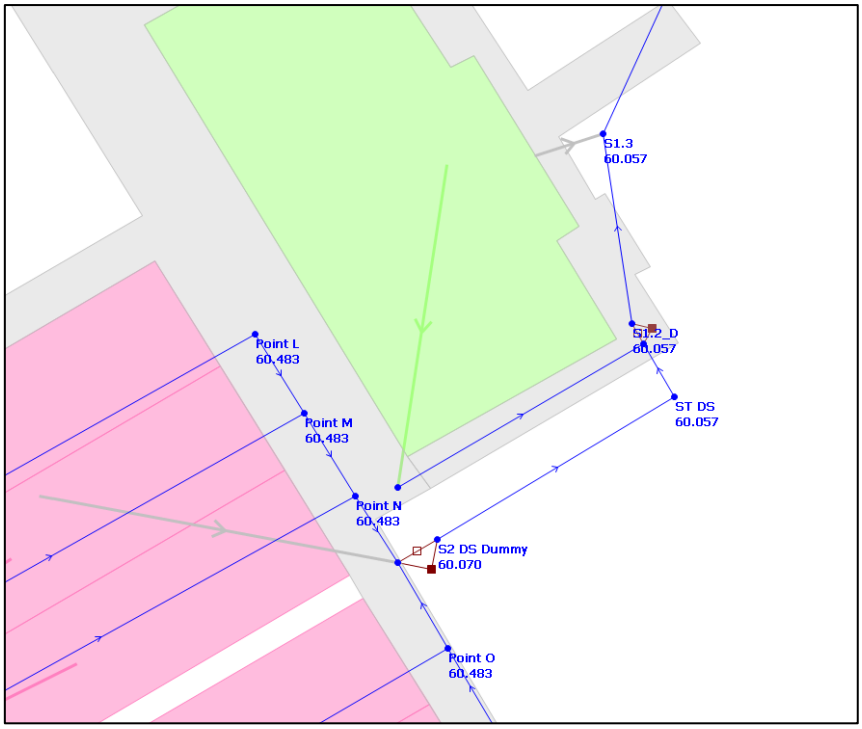


Figure 3 – Close up of Modelled Drainage Layout for the Goals 3G site around storage tank. Labels show the node references as well as the cover levels (assumed using DSM data or taken from the drawings).

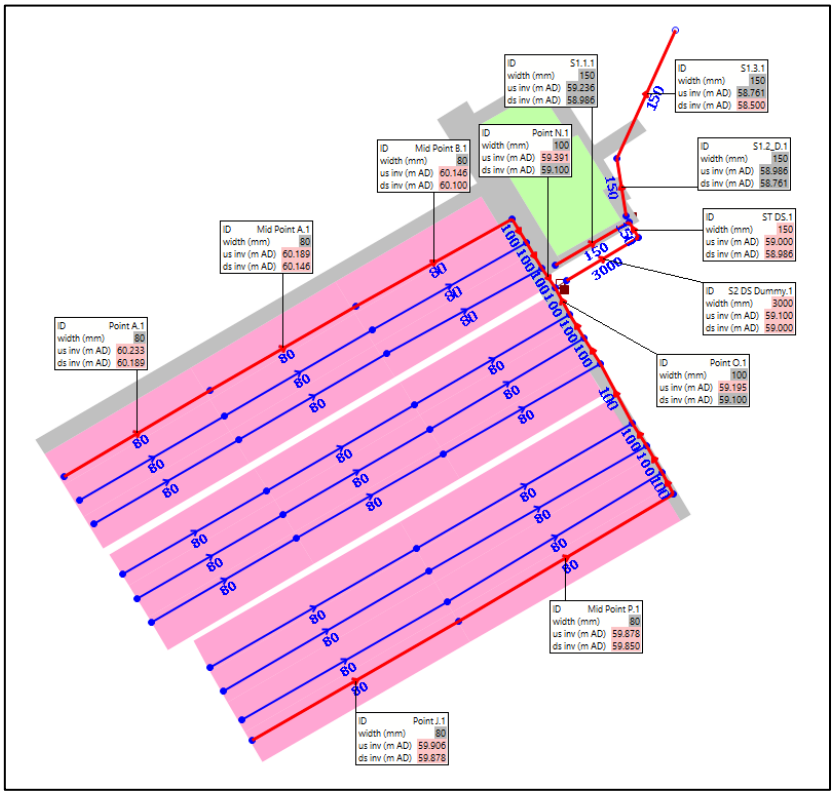


Figure 4 – Overview of Overall Modelled Drainage Layout for the Goals 3G site with pipes labelled. The highlighted 80mm sewers serving the pitches have been labelled as examples, it is noted that the inverts are mirrored in all the other pipes within the same pitch or pitch size.

4.1.2. Sub-catchments

The next stage of the model build was to create sub-catchments that contain runoff surfaces which produce rainfall runoff draining to the relevant nodes. This was completed by using industry best practice to determine the various areas (road, roof and permeable) from drawings provided by the client.

The components of the astroturf were reviewed within the “A103 Details 1.pdf”. It has been assumed that the sports pitches are drained entirely via the 80mm perforated pipes, and therefore these areas have been modelled as fully contributing. A standard permeable runoff surface with a modified routing value of 1 has been used to represent flows to the network from the astroturf pitches. All impermeable surfaces have been modelled using a fixed runoff coefficient of 100%.

Figure 5 below illustrates the sub-catchment arrangement in the InfoWorks model. In this model runoff areas were assigned on an absolute value basis. Contributing areas of hardstanding surfaces and permeable landscaped areas were then assigned to each sub-catchment. Appropriate simulation parameters were attributed to each sub-catchment.

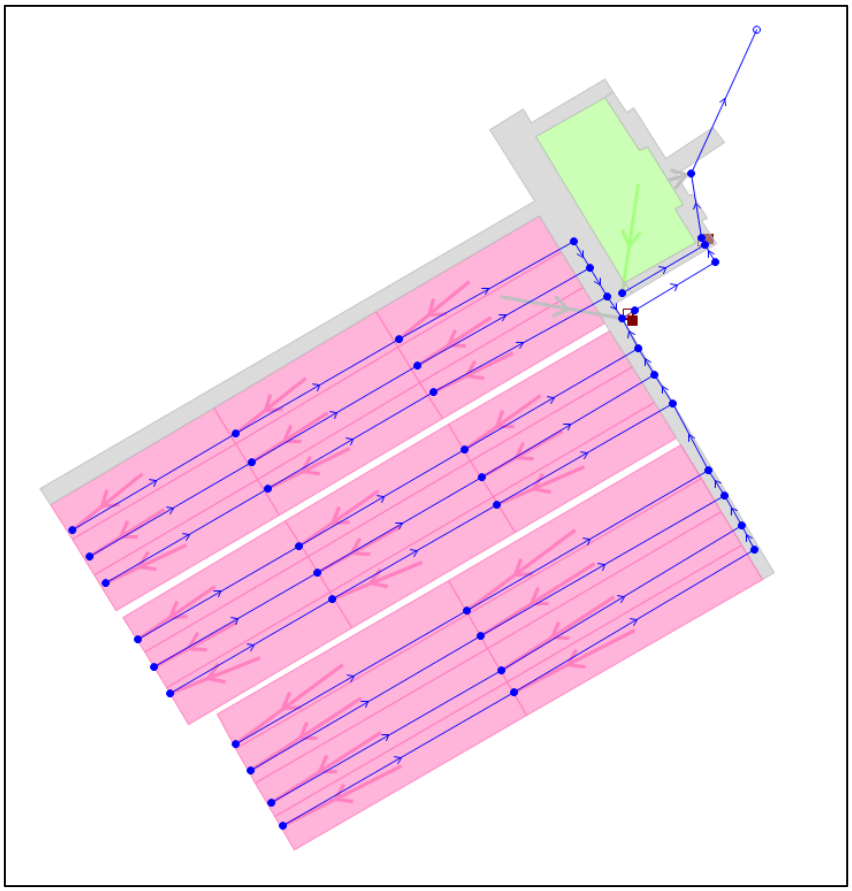


Figure 5 – Storm Water Model for the existing Goals 3G site (Roof sub-catchments shown in green, road sub-catchments shown in grey, sports pitch sub-catchments shown in pink)

4.1.3. Limiting Discharge Devices

There are two hydro-brakes in place: one upstream of the storage tank and one downstream of the entire system. The first hydro-brake limits flow from the football pitches to 3l/s and the second one limits site flows to 5l/s. Both flow control devices contain an over-topping weir.

No head discharge curve has been provided detailing the operation of the either the hydro-brake at manholes S2 or S1.2. These hydro-brakes have been modelled using an orifice link with a fixed discharge rate of 3l/s and 5l/s respectively. Relief weirs have been modelled at both locations, in both instances set to 59.5m AD which corresponds with the top level of the storage tank. The invert level of the weirs as well as the widths have been assumed as there is currently no information available detailing the exact weir dimensions.

4.1.4. Downstream Boundary Condition

The model is set up to discharge to an outfall at the bottom of the car park. This outfall represents the connection point to what is believed to be the United Utilities (UU) public sewer. Information has not been provided regarding the invert level of this UU manhole. It is therefore assumed that the downstream network hydraulic conditions will not impact the site flows from entering the UU network.

4.2. Creating the Future Model Scenario

The client wishes to extend the site footprint by building 2 5-a-side pitches and 1 7-a-side pitch. Figure 6 below indicates the location for these pitches.

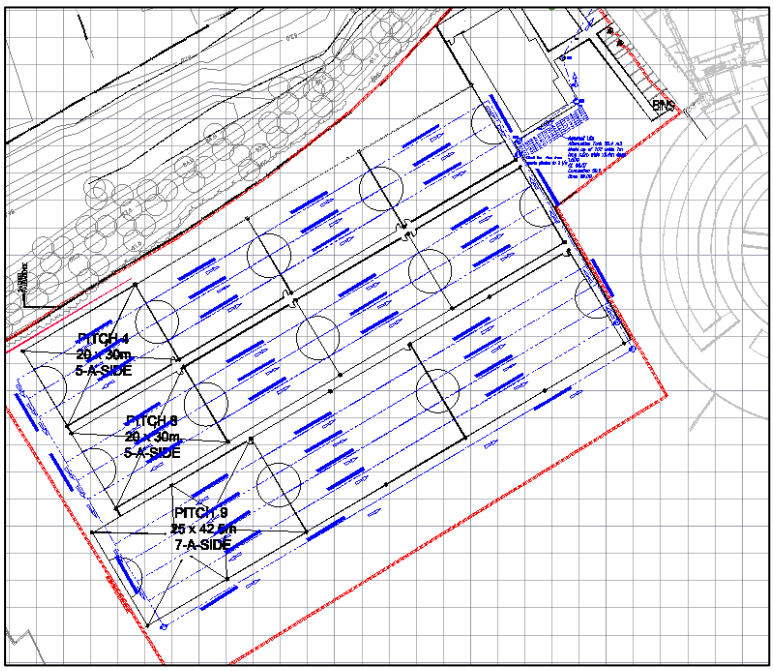


Figure 6 – Storm Water Design Layout for the proposed additional 3 pitches

A new model scenario was created with these new pitches included, see Figure 7 below. The drainage for the new pitches conveys flows initially west, before looping back to the front of the property via a new pipe that runs the length of the pitches.

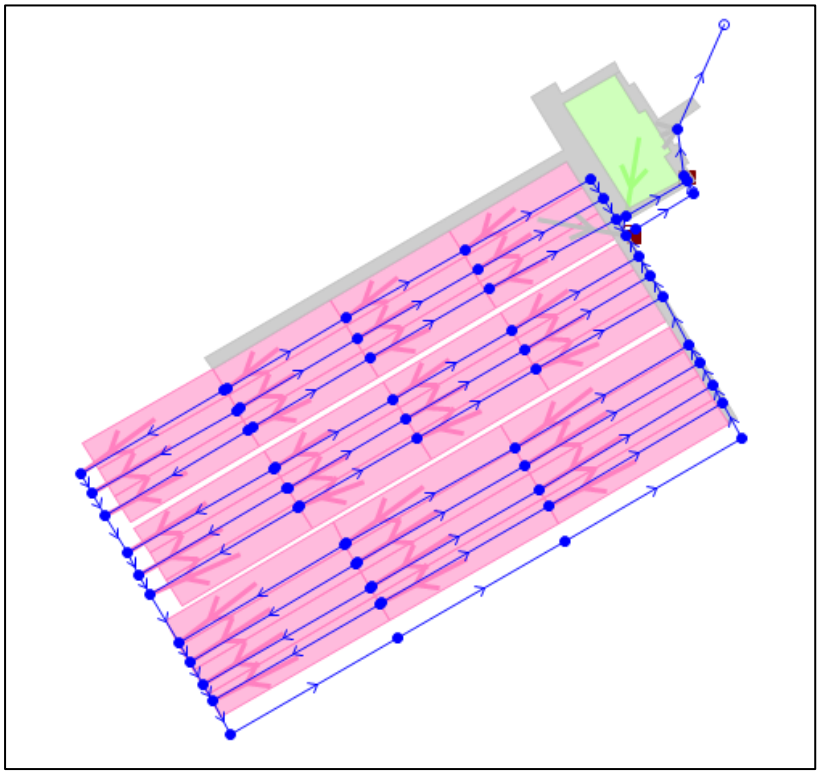


Figure 7 - Storm water model for the proposed Goals 3G site extension (roof sub-catchments shown in green, road sub-catchments shown in grey, and sports pitch sub-catchments shown in pink)

4.3. Contributing Areas

The site layout drawings were used to determine which sub-catchment would drain to each manhole. Table 1 provides an overview of the runoff contributing to the surface water network for each scenario. It can be seen that the additional pitches will cause an increase in contributing area of 0.226ha.

System Type	Existing Model Scenario Contributing Areas (ha)	Future Model Scenario Contributing Areas (ha)
Roof	0.036	0.036
Road	0.071	0.071
Sports pitches	0.573	0.799
Total Development Runoff	**0.680**	**0.906**

Table 1 – Sub-catchment Parameters for each scenario

4.4. Creating the Rainfall Data

Caley Water was instructed to create 4 rainfall scenarios: 1 in 30-year event, 1 in 30-year event plus 30% uplift to allow for climatic change, 1 in 100-year event and 1 in 100-year event plus 30% uplift for climatic change.

Rainfall data was created using the Flood Estimation Handbook Web Service (https://fehweb.ceh.ac.uk/). The website provides point descriptors that are then used by InfoWorks and ReFH2 software to generate desired design event:

- For the 1 in 30-year event plus climatic change, a multiplying factor of 1.3 was applied to a 1 in 30-year events using an inbuilt function in InfoWorks.
- For the 1 in 100-year event plus climatic change, a multiplying factor of 1.3 was applied to a 1 in 100-year events using an inbuilt function in InfoWorks.

5. Model Results

The next stage of the project involved determining the existing system's capacity and whether any predicted flood volumes or surcharge would impact the sewer network. Figure 8 below provides pitch number which has been used throughout this report.

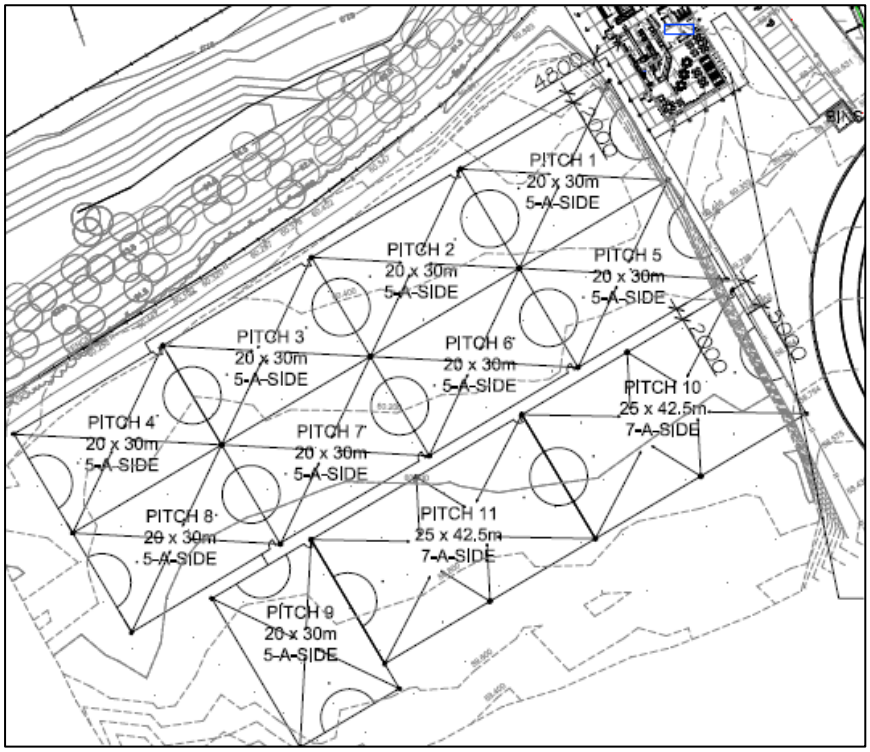


Figure 8 – Pitch referencing from drawing provided by client.

Flood risk has been assessed over a 1 in 30-year return period with and without climatic change (+CC), and a 1 in 100-year return period with and without +CC over a range of winter and summer storm durations (60, 120, 180, 240, 300, 600 and 1440 minute storms). The following section provides the results of these simulations. Table 2 below shows the ground levels, maximum flood levels and flood volumes for all manholes located in the existing system during critical duration storm events.

		1 in 30		1 in 30 + CC		1 in 100		1 in 100 + CC	
Site location	**Average of Ground Level (m AD)**	**Max Level (m AD)**	**Flood Volume (m^3)**	**Max Level (m AD)**	**Flood Volume (m^3)**	**Max Level (m AD)**	**Flood Volume (m^3)**	**Max Level (m AD)**	**Flood Volume (m^3)**
Off pitch	60.232	60.130	0	60.257	0	60.344	0	60.597	2.8
Pitch 1	60.483	60.261	0	60.361	0	60.398	0	60.550	0.3
Pitch 2	60.483	60.310	0	60.451	0	60.501	0	60.689	1.2
Pitch 3	60.483	60.326	0	60.479	0	60.531	0.3	60.728	1.5
Pitch 5	60.483	60.262	0	60.363	0	60.399	0	60.624	0.4
Pitch 6	60.483	60.311	0	60.453	0	60.503	0	60.716	1.3
Pitch 7	60.483	60.327	0	60.480	0	60.533	0.3	60.743	1.6
Pitch 10	60.156	60.064	0	60.325	0	60.418	0	60.709	1.9
Pitch 11	60.156	60.082	0	60.346	0	60.441	0	60.746	2.4
Grand Total			**0**		**0**		**0.6**		**13.4**

Table 2 – Ground Levels, Water Levels and Flood Volumes for all MHs with the existing attenuation storage. Refer to Figure 8 which indicates the locations of each pitch.

Table 3 below shows the ground levels, maximum flood levels and flood volumes for all the site areas under the scenario including the proposed expansion of the Goals 3G site.

		1 in 30		1 in 30 + CC		1 in 100		1 in 100 + CC	
Site location	**Average of Ground Level (m AD)**	**Max Level (m AD)**	**Flood Volume (m^3)**	**Max Level (m AD)**	**Flood Volume (m^3)**	**Max Level (m AD)**	**Flood Volume (m^3)**	**Max Level (m AD)**	**Flood Volume (m^3)**
Off pitch	60.218	60.238	0	60.483	0	60.512	0.1	60.599	5.7
Pitch 1	60.483	60.261	0	60.361	0	60.399	0	60.550	0.1
Pitch 2	60.483	60.310	0	60.451	0	60.503	0	60.689	1.2
Pitch 3	60.483	60.326	0	60.479	0	60.534	0.3	60.728	1.5
Pitch 4	60.483	60.275	0	60.523	0.6	60.545	1.4	60.615	8.3
Pitch 5	60.483	60.262	0	60.363	0	60.401	0	60.622	0.4
Pitch 6	60.483	60.311	0	60.453	0	60.505	0	60.715	1.3
Pitch 7	60.483	60.327	0	60.480	0	60.535	0.3	60.743	1.6
Pitch 8	60.483	60.272	0	60.518	0.6	60.541	1.2	60.612	7.7
Pitch 9	60.156	60.260	0	60.507	0	60.531	0.9	60.606	9.1
Pitch 10	60.156	60.110	0	60.380	0	60.468	0	60.709	1.9
Pitch 11	60.156	60.118	0	60.395	0	60.485	0	60.749	2.6
Grand Total		**60.327**	**0**	**60.523**	**1.2**	**60.545**	**4.2**	**60.749**	**41.6**

Table 3 – Ground Levels, Water Levels and Flood Volumes for all MHs with the existing attenuation storage & pipe sizes under a scenario including the proposed development (new pitches shaded in green)

The results provided in Tables 2 and 3 indicate that flooding will increase as a result of the additional pitches. Minor flood volumes are also predicted during the M30+CC event at pitches 4 and 8.

An assessment was also carried out to determine whether or not the over-topping weirs were predicted to operate at the hydro-brake at the downstream end of the storage tank. Table 4 provides a comparison of these predicted spill volumes during a worst-case duration.

Storm Event Return Period	Baseline Scenario Spill Volume (m^3)	Baseline Peak Flow Rate (l/s)	Future Scenario Spill Volume (m^3)	Future Peak Flow Rate (l/s)
M30	12	7.9	20	7.8
M30 + CC	42	16.1	59	15.0
M100	50	17.5	67	16.7
M100 + CC	110	20.7	143	19.6

Table 4 - Spill volumes over the relief weir at the downstream hydro-brake

The additional pitches indicate an increase in spill volumes across all return periods. Note that these spill volumes bypass the hydro-brake and are conveyed directly into the United Utilities public sewer.

6. Drainage Strategy

Section 5 describes the impact that the additional pitches will have on both the existing Goals 3G drainage network (flooding results) as well as the impact on the UU sewer system (spills over the relief weir at the downstream end of the storage tank).

The following section of the report details the steps that should be taken to ensure that the drainage is compliant with the requests set out by the LLFA.

6.1. Design Strategy

The site location is constrained by numerous other developments around it. A row of dense trees lines the northern border of the site, whilst the Manchester Fort Shopping Park is located along the western border. A school is located immediately south of the pitches and the Irish World Heritage Centre is adjacent on the eastern side. Furthermore, there are main roads such as the A6010 to the north and a network rail site to the east.

There is increasingly limited space to construct green infrastructure such as infiltration trenches, SUDS ponds or swales. As such it has been determined that the most effective way forward is to assess the existing infrastructure and determine locations where this can be increased to provide added capacity.

This will involve pipe upsizes as well as increases in the Geocellular Storage Tanks already in place.

6.2. Connection Point

As per section 3.2.3 of the CIRIA Report C753 – The SuDS Manual, the surface water discharge location must be prioritised in the following order:

- Infiltration
- Surface waters
- Surface water sewers (or other such drainage system)
- Combined sewers

The existing system is known to connect to the UU surface water sewer to the east of the parking lot on Irish Town Way. It is assumed that as this was previously accepted by UU that no further changes to the connection point would be required.

6.3. Peak Runoff Rates

Site flows are currently being controlled by a hydro-brake upstream of the connection point. This hydro-brake limits flows to 5l/s. The greenfield runoff rate was calculated to determine whether this was acceptable. The following values were used to calculate the rate:

- Total Site Area (ha): 0.906
- SAAR (mm): 916
- Soil Type: 4

Using the IH124 method, an estimated site discharge rate of 5.99l/s is obtained (Appendix 2 provides the calculation report generated by HR Wallingford). This is 0.99l/s greater than the capacity of the hydro-brake. It is possible that a hydro-brake with a greater capacity is required to gain approval from the relevant stakeholders.

For the purposes of this exercise, it has been assumed that the hydro-brakes will **not** be retro-fitted.

6.4. Hydraulic Design Calculations

The model simulations have highlighted the impact that the additional two pitches will have on the existing site drainage. The impact is seen in two phases: flooding at various manholes/pitches and spills over the relief weir downstream of the storage tank.

There are two flooding mechanism at play on this site. Firstly, flows will back up from the hydro-brake upstream of the storage tank. This hydro-brake attenuates flows to 3l/s and contains a relief weir which spills directly to the storage tank. Secondly, pipe diameters are small enough that the predicted inflows from the rainfall runoff exceed the pipe's full-bore capacity.

The following sections of this report provide an overview of site upgrades required to reduce flooding and spills.

6.4.1. Flooding Solution

In line with requirements set out by the LLFA, flooding should not occur during a 1 in 100-year rainfall event with allowance for climate change in any part of a building. Based on the drawings provided, the lowest floor level has assumed to be 60.1m AD. This is less than 50mm higher than the cover levels of the storm network around the premises. As such, any predicted flood volumes around the building are likely to fall foul of the LLFA requirements.

Table 3 of this report indicates that a total of 5.7m^3 of flooding is predicted for a 1 in 100-year return period with allowance for climate change around the building. The model was used to determine potential solutions to eliminate this flooding. The following elements are required to eliminate flooding around the building:

- It is recommended that the pipe between Pitch 9 to Pitch 10 be increased in diameter from 100mm to 225mm. This spans a length of approximately 152m.
- It is recommended that the pipe between Pitch 10 to Manhole reference S2 is increased in diameter from 100mm to 375mm. This spans a length of approximately 54m.
- It has been assumed that any flooding within the pitches will be 'self-contained' and not gravitate anywhere else within the site premises. As such there is no requirement to increase the land drainage underneath the astroturf pitches.

Figure 9 shows the locations that require pipe diameter increases.

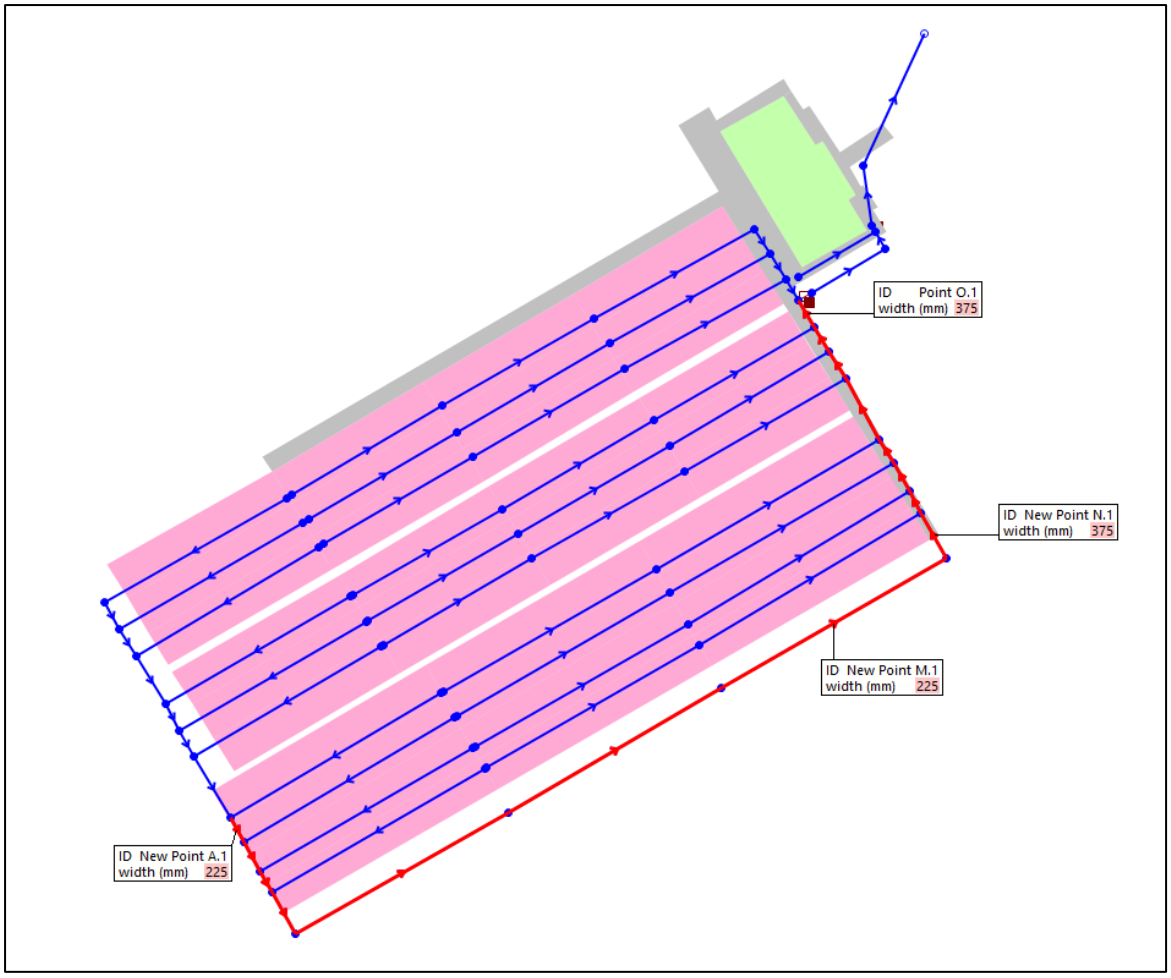


Figure 9 - Plan view of the pipes that should be increased in diameter. Pipes highlighted in red and labelled.

6.4.2. Relief Weir Spill Solution

Upon removing the local incapacity in the network external to the sports pitches, the overtopping spills at manhole S1.2 downstream of the GCS units was reviewed. Table 5 below shows the maximum spill volume recorded at S1.2 during each of the four return periods across the different stages of the solution development.

Storm Event Return Period	Baseline Scenario Spill Volume (m^3)	Future Scenario Spill Volume (m^3)	Flooding Solution Scenario Spill Volume (m^3)
M30	12	20	17
M30 + CC	42	59	61
M100	50	67	71
M100 + CC	110	143	160

Table 5 – Total volume spilled over the relief weir in Hydrobrake MH S1.2 during the critical duration storm for each return period. Comparison of Baseline, Future and Flooding Solution Scenarios

The table above indicates that spills over the relief weir increase from 110m^3 to 160m^3 during the 1 in 100 years plus climate change scenario as a result of both the new pitches plus the increased pipe sizes. In order to reduce these spills back to the baseline scenario, increased storage is recommended. A new model scenario was used to calculate the additional storage requirements. This scenario includes the extra pitches, the increased pipe diameters, and an allowance for additional storage at the GCS units. This assessment found that an additional 45m^3 of storage is required to reduce M100+CC spills back to their baseline value. Table 6 below shows the predicted relief weir spills with this additional storage.

Storm Event Return Period	Baseline Scenario Spill Volume (m^3)	Storage + Flooding Solution Scenario Spill Volume (m^3)	Storage + Flooding Solution with 25% allowance Scenario Spill Volume (m^3)
M30	12	0	0
M30 + CC	42	11	0
M100	50	21	4
M100 + CC	110	110	92

Table 6 - Total volume spilled over the relief weir at Hydro-brake in MHS1.2 during the critical duration storm for each return period. Comparison of Baseline and Flooding + Storage Solution Scenario.

In line with the guidance presented in Section 24.9.4 of the CIRIA SuDS manual, an additional 25% storage requirement will be applied to this volume in order to minimise the risk from future scenarios. This means that a total storage volume of 82m^3 is required at the current location of the GCS (this is comprised of the 20.4m^3 already present plus ~62m^3 of new storage).

Table 7 below provides a review of predicted flood volumes with the new storage and pipes included.

		1 in 30		1 in 30 + CC		1 in 100		1 in 100 + CC	
Site location	**Average of Ground Level (m AD)**	**Max Level (m AD)**	**Flood Volume (m^3)**	**Max Level (m AD)**	**Flood Volume (m^3)**	**Max Level (m AD)**	**Flood Volume (m^3)**	**Max Level (m AD)**	**Flood Volume (m^3)**
Off pitch	60.21767	60.130	0	60.134	0	60.135	0	60.138	0
Pitch 1	60.483	60.261	0	60.361	0	60.398	0	60.550	0.1
Pitch 10	60.156	60.029	0	60.160	0	60.211	0	60.488	0
Pitch 11	60.156	60.064	0	60.227	0	60.289	0	60.614	0.8
Pitch 2	60.483	60.310	0	60.451	0	60.501	0	60.689	1.2
Pitch 3	60.483	60.326	0	60.479	0	60.531	0.3	60.728	1.5
Pitch 4	60.483	60.253	0	60.253	0	60.253	0	60.351	0
Pitch 5	60.483	60.262	0	60.363	0	60.399	0	60.551	0.3
Pitch 6	60.483	60.311	0	60.453	0	60.502	0	60.690	1.2
Pitch 7	60.483	60.327	0	60.480	0	60.532	0.3	60.729	1.5
Pitch 8	60.483	60.253	0	60.253	0	60.253	0	60.288	0
Pitch 9	60.156	59.926	0	59.930	0	59.933	0	59.951	0
Grand Total			**0**		**0**		**0.6**		**6.8**

Table 7 – Ground Levels, Water Levels and Flood Volumes for all site areas under the proposed development with additional storage attenuation and pipe upsizes

7. Maintenance Proposals

The following maintenance regime is proposed based on the latest guidance provided in the CIRIA SuDS Manual.

7.1. SuDS Components

Geocellular Storage Tank

There is only one SuDS component on this site, this is the Geocellular Storage tank. It is intended that this tank will retain enough of the storm flows for large return period events before discharging back in to the UU surface water network further downstream along Irish Heritage Way. The flows both entering and leaving the GCS are regulated by hydro-brakes at either end of the tanks. It should be noted that the GCS tank will only hold rainfall runoff generated from the football pitches and a small area of paved runoff (walkways between pitches and the building).

Catchpit

There is a catchpit at manhole reference S1.1 which will provide treatment of surface water flows from the building roof and the paved areas nearby. The GSC tank is not linked hydraulically to this catchpit.

7.2. Maintenance Schedule

The following table provides a typical maintenance schedule for an attenuation storage tank such as the one recommended for this site. This guidance has been taken from the CIRA SuDS Manual, Section 21.13 and is applicable for this site.

Maintenance Schedule	Required Action	Typical Frequency
Regular Maintenance	Inspect and identify any areas that are not operating correctly. If required, take remedial action.	Monthly for 3 months, then annually
	Remove debris from the catchment surface	Monthly
	Remove sediment from silt trap	Annually
Remedial Actions	Repair/rehabilitate inlets, outlet, overflows, and vents	As and when required
Monitoring	Inspect inlets, outlets, vents, and overflows (including hydro-brakes) to ensure that they are in good condition and operating as designed	Annually and after storm events
	Survey inside of tank for sediment build-up and remove if necessary	Every 5 years

Table 8 - Recommended Maintenance Schedule

8. Conclusions

The following conclusions have been made for this study:

- The model has been created from various PDF drawings provided by the client.
- Best modelling practice has been adopted when creating model sub-catchments and preparing rainfall data.
- Flood risk has been assessed for a full range of storms for the 1 in 100-year return period with climatic change. Critical storm durations have been identified.
- The surface water network at Goals 3G Manchester is predicted to flood for critical storm durations during all the tested return periods with the exception of 1 in 30 year.
    - Total predicted flood volumes are 0.6m^3 and 13.4m^3 for a 1 in 100 year and 1 in 100 year + CC event respectively.
    - Predicted flood volumes will increase as a result of the additional pitches. Total predicted flood volumes of 1.2m^3, 4.2m^3 and 41.6m^3 are predicted during the 1 in 30 + CC, 1 in 100 and 1 in 100 + CC scenarios respectively.
    - **Total flood volume of 5.7m^3 is predicted around the building for a 1 in 100 + CC scenario.**
- The relief weir is predicted to spill a total of 110m^3 during a 1 in 100 + CC event in the baseline scenario. This is predicted to increase to 143m^3 with the addition of the new pitches.
- Hydraulic design assessment has resulted in the following recommendations:
    - **The pipe between Pitch 9 to Pitch 10 should be increased in diameter from 100mm to 225mm. This spans a length of approximately 152m.**
    - **The pipe between Pitch 10 to Manhole reference S2 should be increased in diameter from 100mm to 375mm. This spans a length of approximately 54m.**
    - **The total site storage required is 82m^3, comprised of the existing 20.4m^3 and 62m^3 additional storage:**
        - In order to return to a baseline scenario, the additional storage volume required to attenuate the 1 in 100-year storm event plus climatic change to a restricted discharge flow rate of 5l/s is 45m^3.
        - In line with the latest CIRIA guidance, an extra allowance of 25% should be added, increase the total additional storage by 62m^3.
- The proposed maintenance schedule provided recommends frequent clearing of sediment in the silt trap as well as constant monitoring of the SuDS components to ensure sediment build-up does not adversely impact the site hydraulics.

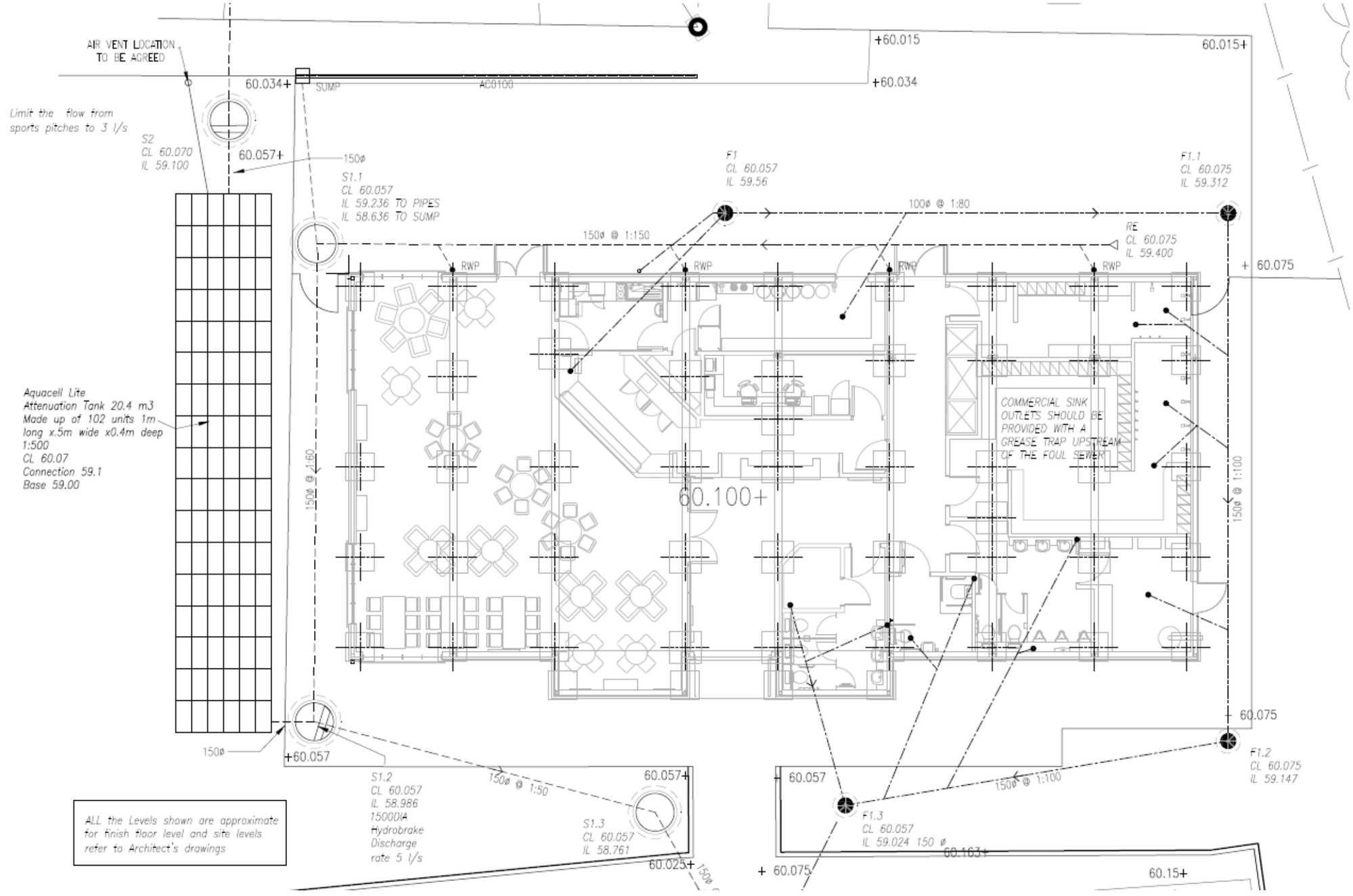
9. Project Assumptions

The following are limitations/points of discussion for this project:

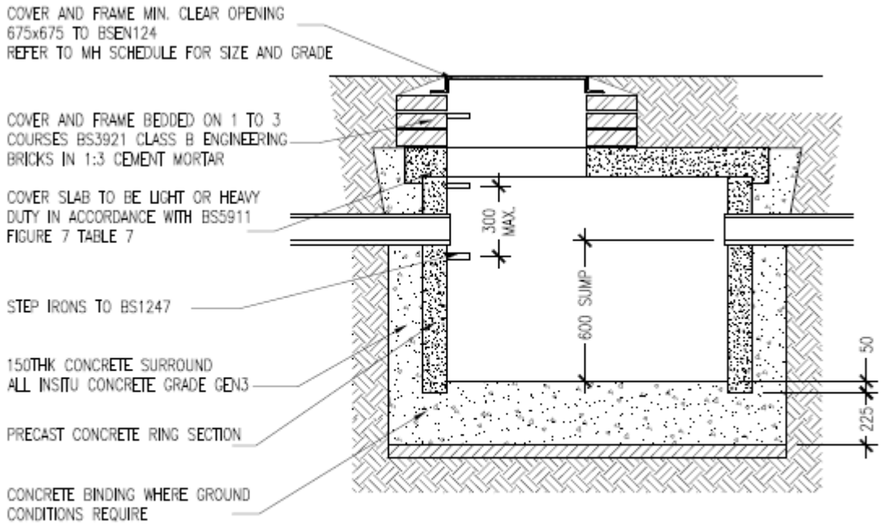
- The sewer boundary conditions have been assumed as a free discharge to the UU network. It is assumed that the downstream network will have enough capacity to accommodate the added flows
- There is little guidance available on best practise for modelling surface runoff from Astro-Turf pitches. As a result, the assumption was made to model the runoff as a simple NewUK permeable surface with a reduced routing value of 1.
    - No ground conditions have been provided (either through borehole testing or otherwise), therefore soil conditions such as soil porosity are unknown.
- The pitch designers were not able to confirm the maximum capacity of the land drainage (i.e., the 3, 80mm diameter pipes that run underneath each pitch). As such, the pipe depths, gradients and capacities have been assumed.
- It is assumed that the onsite flooding which is predicted to occur within the pitches themselves is tolerable. During a 1 in 100-year event with CC minor flooding is predicted within most of the pitches, however as this is an extreme event, and the overall flood depth will remain relatively low.

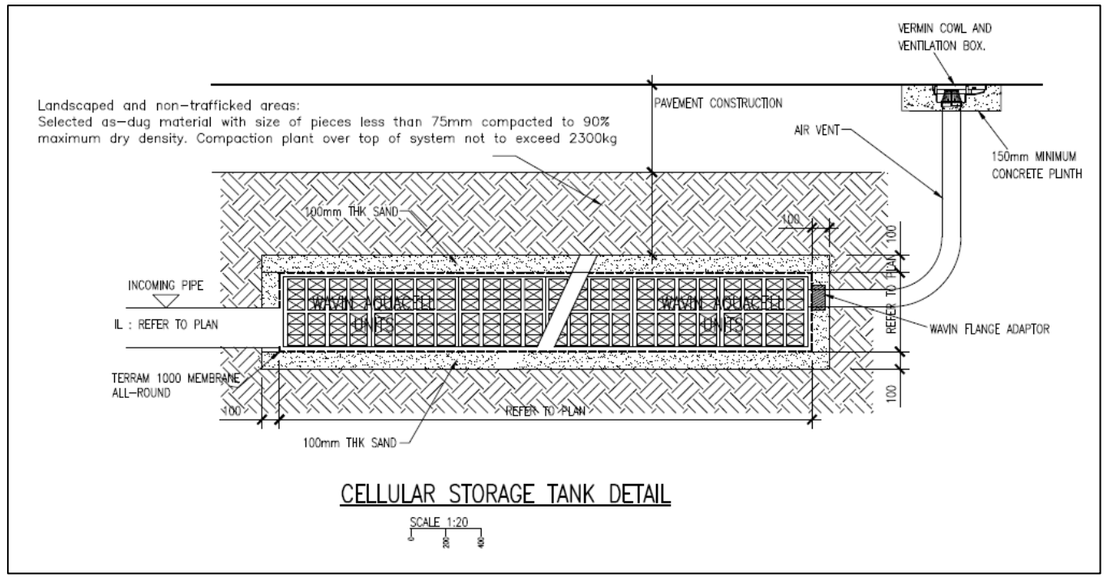
Appendix 1

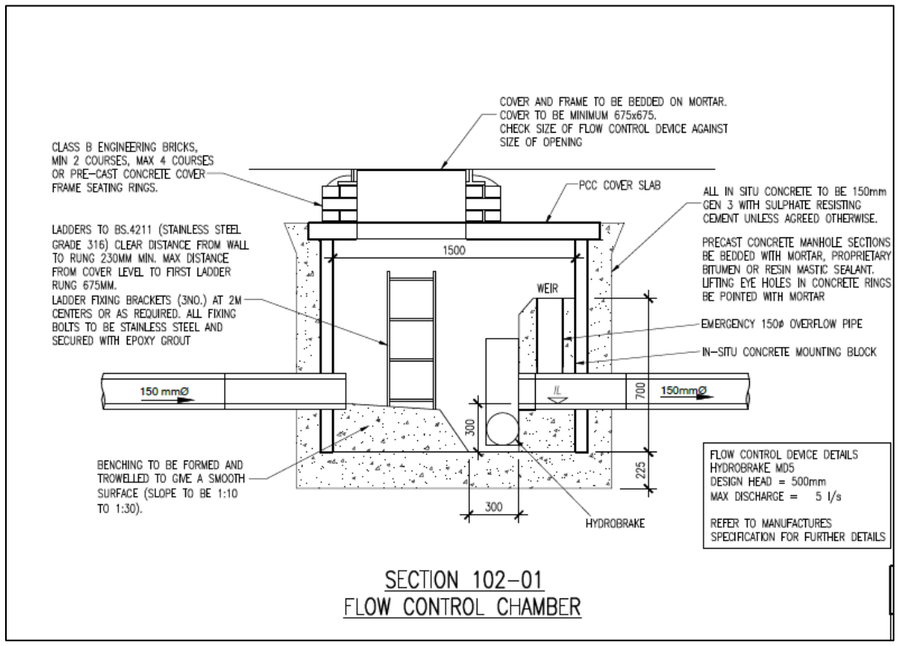
General Arrangement of building site

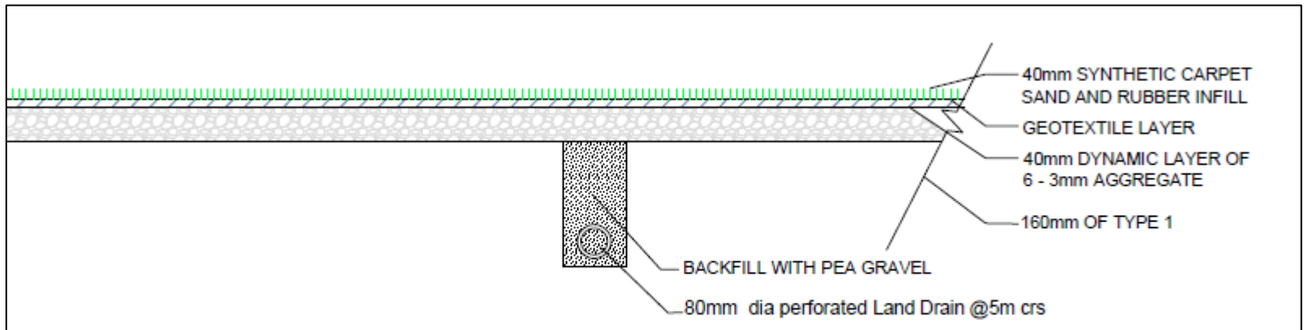


Typical Details of Drainage Elements









Appendix 2

HR Wallingford Report showing greenfield runoff rate estimation calculations

HR Wallingford
Working with water

Greenfield runoff rate estimation for sites

www.uksuds.com | Greenfield runoff tool

Calculated by:	Mark Campbell
Site name:	Goals 3G
Site location:	Manchester

This is an estimation of the greenfield runoff rates that are used to meet normal best practice criteria in line with Environment Agency guidance "Rainfall runoff management for developments", SC030219 (2013) , the SuDS Manual C753 (Ciria, 2015) and the non-statutory standards for SuDS (Defra, 2015). This information on greenfield runoff rates may be the basis for setting consents for the drainage of surface water runoff from sites.

Site Details

Latitude:	53.50038° N
Longitude:	2.23057° W
Reference:	1228714014
Date:	Jan 12 2022 16:08

Runoff estimation approach IH124

Site characteristics

Total site area (ha): 0.906

Methodology

Q_{BAR} estimation method:	Calculate from SPR and SAAR
SPR estimation method:	Calculate from SOIL type

Soil characteristics	Default	Edited
SOIL type:	4	4
HOST class:	N/A	N/A
SPR/SPRHOST:	0.47	0.47

Hydrological characteristics	Default	Edited
SAAR (mm):	916	916
Hydrological region:	10	10
Growth curve factor 1 year:	0.87	0.87
Growth curve factor 30 years:	1.7	1.7
Growth curve factor 100 years:	2.08	2.08
Growth curve factor 200 years:	2.37	2.37

Notes

(1) Is Q_{BAR} < 2.0 l/s/ha?

When Q_{BAR} is < 2.0 l/s/ha then limiting discharge rates are set at 2.0 l/s/ha.

(2) Are flow rates < 5.0 l/s?

Where flow rates are less than 5.0 l/s consent for discharge is usually set at 5.0 l/s if blockage from vegetation and other materials is possible. Lower consent flow rates may be set where the blockage risk is addressed by using appropriate drainage elements.

(3) Is SPR/SPRHOST ≤ 0.3?

Where groundwater levels are low enough the use of soakaways to avoid discharge offsite would normally be preferred for disposal of surface water runoff.

Greenfield runoff rates	Default	Edited
Q_{BAR} (l/s):	5.99	5.99
1 in 1 year (l/s):	5.21	5.21
1 in 30 years (l/s):	10.19	10.19
1 in 100 year (l/s):	12.46	12.46
1 in 200 years (l/s):	14.2	14.2

This report was produced using the greenfield runoff tool developed by HR Wallingford and available at www.uksuds.com. The use of this tool is subject to the UK SuDS terms and conditions and licence agreement , which can both be found at www.uksuds.com/terms-and-conditions.htm. The outputs from this tool are estimates of greenfield runoff rates. The use of these results is the responsibility of the users of this tool. No liability will be accepted by HR Wallingford, the Environment Agency, CEH, Hydrosolutions or any other organisation for the use of this data in the design or operational characteristics of any drainage scheme.