FLOOD RISK ASSESSMENTS & DRAINAGE STRATEGIES

Land North Of Milton Road, Adderbury

Phase 1 - Surface Water Drainage Design

Reference: 0202 - DD- 001

Nov-21

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SURFACE WATER DRAINAGE SCHEME

Purpose of this report

1.1 The following report has been prepared to reflect changes to the drainage design submitted as part of clearing the planning condition 3 of planning approval 18/00220/F.

Condition 3 was cleared for the phase 1- Natural sport fields. The drainage proposal was considered relevant at the time of clearing the planning condition. It has now become apparent that the natural sport fields drainage design needs to be updated to fit within the overall site drainage design and the updated proposals for the sport hall.

1.2 This report only related to phase 1 of the development. Phase 2 surface water drainage design will be complete using a different report.

Existing and Proposed Site

2.1 The site is bordered by a housing development and fields to east and Ball Colegrave on the west. The north section is bordered by an unnamed ditch and fields. The Milton Road borders the site on the south. See picture 1 below.

The estimated lifetime of this development is: 100 years

Existing Site

Proposed Site

2.2 Hydraulically all the phase 1 greenfield run-off for the site is being intercepted by the unnamed ditch located to the north of the site. The distribution of catchment areas for existing and proposed site is as per table 1 below.

Table 1 : Surface Type distribution for positively drained areas in hectares

Description	Existing Site	Proposed Site
Impermeable Surface	0.000	0.000
Permeable Surface	2.700	2.700
Total Area positively drained	0.000	0.675

2.3 There is no increase in impermeable areas between the existing and proposed site. The total area of the sport field is 2.7ha, of this area only the lower part is considered to be positively drained. This is because the upper section is has a good infiltration in which all water is being infiltrated. This lower part is ¼ of the total area. The catchment area is considered to be 0.675Ha. Due to the infiltration of the field, the actual run-off coefficient is 10%. These values has been used to create the model.

Site Characteristics

2.5 The site background is clearly identified through answers to the questions below

TOPIC	QUESTION	ANSWER
Protected species or habitat	Is the site near to designated sites and priority habitats?	No
Flood Plain	Is the site located in the flood plain?	No
Topography	Sited on a flat site?	No
	Sited on a steep slope (5-15%)	Yes
	Sited on a very steep slope (>15%)	No
Groundwater	Is ground Water less that 3m bgl?	No
Runoff characteristics	Is the development in a high risk flooding area?	No

Evaluation of Discharge Point

2.6 The SuDS design takes into account Building Regulations Section H3. Rainwater from roofs and paved areas is carried away from the surface to discharge to one of the following in order of priority:

Discharge to:	**Site Assessment**
Adequate infiltration system	The site has potential for infiltration. See site investigation. The natural sport pitches areas have two distinctive ground profiles in which the upper section has a relative better infiltration rate than the lower part of the site. Due to the size and type of the development, a variability on the soil permeability can be allowed. As part of the worst case scenario for the site an infiltration rate of 1.58 x 10-4m/s or 0.5705 m/hr has been allowed for the site. See appendix B.
a watercourse	There is an existing ditch running parallel to the site. The Oxfordshire County Council (LLFA) states that sites should discharge at greenfield run-off when infiltration is not possible. This statement is applicable in build-up areas; however for the sport fields the allowance for greenfield run-off should be allowed as it will provide base flow for fauna and flora within corridors of these ditches. As part of this design a discharge rate 1 l/s is also allowed for. This rate is unlikely to increase the risk of downstream flooding.
a surface water sewer	There are not public drains in the proximity to the site

Peak Runoff and Attenuation Volumes 3

Peak Run-off Rate

3.1 The peak runoff rate for the existing site was calculated as per table 3. Calculation results are in table 5 and appendix C.

Table 3: Peak run-off rate calculation method for existing site

Method Used	Calculation Method
✓	This is a greenfield site, as the proposed development area is less than 50ha, the Institute of Hydrology(IoH). Report124 Flood Estimation for Small Catchments method has been used to estimate the site peak flow rates
	This is a brownfield site, runoff rates are calculated in accordance with best practice simulation modelling
	This is a brownfield site where the pre-development drainage isn't known therefore the runoff rates are calculated using the Greenfield run-off model (above) but using soil type 5

3.2 The runoff flow produced by the development will be controlled as per table 4.

Table 4: Runoff discharge rate control

Control Used	Description of runoff discharge
✓	Water will be discharged into the ground via a SuDS as described in table 6 below
✓	The peak discharge rate has been reduced to pre-development Qbar flow
	The limiting discharge rate requires a flow rate less than 5l/s at discharge point, therefore a rate of 5l/s is used
	The peak discharge rate has been agreed with the local water company to be 1:30 storm event flow rate

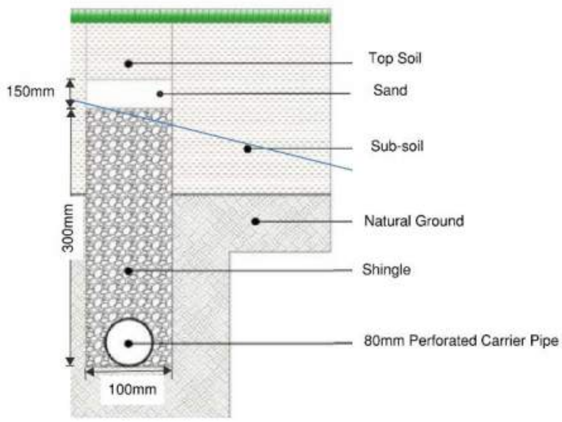
Attenuation Volumes

3.3 Natural sport fields with underdrains provide good attenuation for storms with longer rainfall intensities. The Loughborough University research paper "Drainage behaviour of sport pitches - findings from a research study" states that this attenuation varies from 30 to 90% of a 1 in 100 storm event. This is due to head loss on the corrugated lateral pipes, the loss of rainfall due to evaporation and the reservoirs available within the gravel and perforated pipes. See appendix A for the sport field research paper.

Proposed Sustainable Drainage 4

4.1 The proposals shows that the sport field is being drained using a 100mm perforated pipework discharging to a carrier pipe than then discharges to the infiltration basin. The infiltration basin size is 2m x 3m (at base) x 1m dp. The basin provides a storage volume of 12.33m3. From this infiltration basin, water will be discharged to the ditch at a rate of 1l/s. See drainage layout in appendix D.

The details of the carrier pipes are as per the drawing below.



3.3 Micro Drainage was used to calculate the size of the attenuation based on the available infiltration rate, areas and the drains already installed on site. The calculations as for all events up to the 1 in 100 including an allowance for climate change of 30%. See table 5 for value and appendix C for calculations.

Table 5: Peak discharge rates and anticipated attenuation volumes for SuDS

Return Period Event	Peak Discharge Rate (l/s)		Infiltration Rate (m/hr)	Attenuated Storage Volume (m3)
	Existing	Proposed		
Qbar(1 in 2)	2.10	1.0	0.5705	
1 in 30	4.80	1.0	0.5705	
1 in 100	6.80	1.0	0.5705	
1 in 100 + CC	[redacted]	1.0	0.5705	

The model shows that there is not flooding for any of the storm events and therefore the infiltration basin and the proposals have sufficient capacity to accommodate the flows from the sports field.

4.2 The location and details of the SuDS can be seen drainage layouts in appendix D. Calculations are in appendix C.

4.3 The drainage calculations demonstrate:
- The post development runoff volumes have been reduced to the pre-development runoff values by infiltrating all the run-off produced by the development.
- No flooding occurs for the 1 in 30 storm events.
- Any flooding for the 1 in 100 year +30% climate change event can be safely contained on site

Management of Exceedance Flows

4.5 The drainage network has been designed to attenuate surface runoff for all events up to and including the 1% AEP + CC(1 in 100 years). However consideration has been given to what may happen when the design capacity of the surface water drainage network is exceeded. Surface water will flow to the ditch as it is currently happening.. The flood risk remains low. See appendix D.

Maintenance and Management plan responsibility

6.1 The SuDS will be maintained by the owner of the site

Maintenance and Management plan for proposed SuDS

6.2 The maintenance and Management Plan Guidance from the SuDS Manual, CIRIA C753 (CIRIA, 2015) is to be followed for the effective maintenance of the proposed SuDS techniques outlined above. The maintenance for SuDS structures are as follow:

TABLE 13.2 Operation and maintenance requirements for infiltration basins

Maintenance schedule	Required action	Typical frequency
Regular maintenance	Remove litter, debris and trash	Monthly
	Cut grass – for landscaped areas and access routes	Monthly (during growing season) or as required
	Cut grass – meadow grass in and around basin	Half yearly: spring (before nesting season) and autumn
	Manage other vegetation and remove nuisance plants	Monthly at start, then as required
Occasional maintenance	Reseed areas of poor vegetation growth	Annually, or as required
	Prune and trim trees and remove cuttings	As required
	Remove sediment from pre-treatment system when 50% full	As required
Remedial actions	Repair erosion or other damage by reseeding or re-turfing	As required
	Realign the rip-rap	As required
	Repair or rehabilitate inlets, outlets and overflows	As required
	Rehabilitate infiltration surface using scarifying and spiking techniques if performance deteriorates	As required
	Relevel uneven surfaces and reinstate design levels	As required
Monitoring	Inspect inlets, outlets and overflows for blockages, and clear if required	Monthly
	Inspect banksides, structures, pipework etc for evidence of physical damage	Monthly
	Inspect inlets and pre-treatment systems for silt accumulation; establish appropriate silt removal frequencies	Half yearly
	Inspect infiltration surfaces for compaction and ponding	Monthly

Conclusions 6

The proposals manage and eliminate flood risk for phase 1. It demonstrates that there is not increase risk to properties downstream of the site.

Appendix A

EXISTING SITE
NTS

PROPOSED SITE
1:1000

Drawing Scale Bar

Drawing scale	Line length	Drawing scale	Line length
1:5	= 0.25 metres	1:200	= 10.0 metres
1:10	= 0.5 metres	1:250	= 12.5 metres
1:20	= 1.0 metres	1:500	= 25.0 metres
1:25	= 1.25 metres	1:1000	= 50.0 metres
1:50	= 2.5 metres	1:1250	= 62.5 metres
1:100	= 5.0 metres	1:2500	= 125 metres

Measure length of line above for checking of scale

GENERAL NOTES

KEY

POSITIVELY DRAINED AREAS

STUDY AREA

Rev	Details	Date	By	Chk'd

Drawing Status:

PRELIMINARY

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Client:

Project:
Land North Of Milton Road, Adderbury

Drawing:
Existing and Proposed Areas
Permeable and Impermeable - Phase 1

Print Size:	Project No:	Drawing No:	Revision:
A1	0202	002	P1

Drainage behaviour of sport pitches - findings from a research study

DRAINAGE BEHAVIOUR OF SPORT PITCHES - FINDINGS FROM A RESEARCH STUDY

This report presents the key activities and outcomes of a 3 year PhD sponsored by Loughborough University in collaboration with an industry steering group comprised of the IOG, SAPCA, Sport England, and the STRI. Additional funding for fieldwork was provided by IOG and Sport England, and collaboration on natural turf sites with Agripower.

Prepared by: Dr Paul R Fleming, Dr Matthew W Frost, and Dr Murray Simpson, Loughborough University.

Prepared for: Industry Stakeholders

Date: 23-08-2016

SUMMARY. The drainage design of sports pitches has traditionally been based on experience and can be considered an inexact science. Whilst the sport surface can be adequately drained to meet specific criteria, estimating outflows at the discharge point is more challenging. The hydraulic performance of sports pitches has not previously been measured in detail prior to this study.

Within the wider industry and regulatory bodies there is a perceived contribution to local flood risk of the storm water and run off from sport pitches. It is also apparent that artificial pitches have in some cases been treated in planning consents as impermeable.

Observations from industry have suggested that in reality the pitch drainage systems discharge low volumes of water and low peak flow rates, with limited surface runoff (especially from porous artificial pitches). However, in some cases, for artificial pitches in particular, at planning stage the drainage design has required to include off-line tanks to provide storm water storage and attenuation. A lack of technical guidance on sport pitch design and drainage benefits may be leading to overdesign, and prompted this study.

This 3 year study comprised field measurements of weather and discharge behaviour at a range of artificial and natural turf pitches in England; laboratory physical model testing of pitch component hydraulic behaviour; and mathematical modelling to predict how a pitch system may be expected to perform hydraulically. Bespoke field monitoring apparatus was developed as part of the research to measure across a large range of flow rates and volumes.

The experimental work in this study has provided the evidence to demonstrate that the porous pitch designs provide high attenuation of peak rainfall events and large capacity for water storage, similar to the requirements of SuDs based ‘source control’ designs required in new urban developments.

The field monitoring observations suggest that in reality the drainage system behaviour is not as consistent or predictable as might be expected from assumptions made in design software and that in all cases the measured outflow water volume was far less than that estimated from rainfall as the total water volume flowing into the pitch drainage system.

The experimental work, combined with the mathematical modelling, has highlighted the key mechanisms that provide resistance to flow and explain the attenuation behaviour observed. It is considered that in most cases insufficient head is created in the sub-surface layers to drive water to the lateral drainage pipes, and that the high frictional resistance to flow in the corrugated collector pipes provide large ‘head’ losses under the low hydraulic gradients.

The research findings support the claims by many in the industry that in some cases planning approvals, where a lack of understanding or evidence on how pitches can attenuate and store water exists, may be causing the over-design of pitch drainage systems requiring unnecessary offline storage tanks.

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1.0 BACKGROUND TO THE PROBLEM

The occurrence and severity of flooding is increasing annually; emerging research reinforces the need for improved drainage infrastructure to reduce flood risk (IPCC, 2015). Government and planning authorities are imposing restrictions on surface water discharges from new developments into existing infrastructure and watercourses.

New sports facilities have been subject to such drainage restrictions. In particular, the large coverage of sports surfaces such as natural and synthetic pitches (typically >7500m^2 for a full-sized pitch) has resulted in the anticipation of large volumes of rainwater entering local watercourses in a potentially unconstrained way. To manage these perceived large volume yields of storm-water many facilities invest in large (separate) attenuation tanks designed to store the storm water for controlled release into the local drainage network without making any use of the properties of the pitch itself. These systems are effective at limiting the impact of drainage discharge, but can represent a large additional cost to a project budget.

It was considered possible to address this issue through better understanding of the hydraulic properties of sports pitch constructions, on the basis that their pervious and porous designs lend themselves to an intrinsic storage and attenuation as in found in Sustainable Urban Drainage Systems (SuDs) such as green roofs or permeable car parks. SuDs form part of the latest building regulations and aim to capture, store and attenuate storm water at source, in most cases returning it to the ground. If SuDs principles can be integrated more effectively within sports pitch drainage design many opportunities exist for the industry to enhance current construction and regulatory practice.

The study was formulated in collaboration with industry, and funded a three year PHD programme at Loughborough University. The aim of the research study was to measure and understand the hydraulic performance of sport pitches. The aim was broken into specific key objectives as follow:

1. Critically appraise literature in relation to current practice in sports pitch design, sustainable drainage techniques and hydrology.

2. Investigate the drainage performance of existing sports pitch drainage systems at selected field locations.

3. Investigate the hydraulic characteristics of pitch component materials under laboratory conditions.

4. Explore the key drainage mechanisms (identified in objectives two and three) through mathematical modelling.

This report presents an overview of the background and findings from the study (the full study report is in the form of a PhD thesis).

1.1 Introduction to Urban Drainage

Growth in urban areas has led to an increase in impervious surfaces such as roads, car parks and roofs (Mansell, 2003). These surfaces act as barriers that limit the natural infiltration of rainfall into the ground where it lands. The resulting impact is an increase in surface water volume conveyed through storm-water drains to a receiving watercourse. One impact of urbanisation has been a reduction in the infiltration capacity of the land and an increase in the speed at which runoff reaches local watercourses, the 'lag time', the time between the peak rainfall intensity and peak discharge, reduced by a factor of 8 (Mansell 2003). This can greatly increase the risk of flooding locally. In contrast, allowing infiltration of stormwater into the ground ensures the water flow routes become more convoluted and reduces the rate and volume of runoff from an area – and this is the basis for modern urban sustainable drainage practice termed sustainable drainage systems (SuDs).

SuDs is the general term for dynamic flood water management systems, by utilising and enhancing the environment's natural ability to attenuate surface water flooding as close to the source as possible – often termed source control.

Modern permeable paving is an example of integrating SuDs into design. Surface rainwater directly infiltrates into the ground below the paving blocks and the foundation offers some degree of storage (and filtration) in an open textured granular material or for more storage capacity voided structures are provided such as geo-cellular boxes (CIRIA, 2007) or pipes and tanks can be used.

Key legislative drivers for SuDs originate at European level, for example from the Water Framework Directive (WFD) (2000/60/EC) that aims to safeguard the environment for future generations and achieve a good ecological status in all watercourses by 2015. UK planning and policy guidance includes Planning Policy Statement 25 (2010) for urban developments which promote SuDs philosophy. SuDs design and construction guidance is set out in the Suds Manual (CIRIA, 2007) which is being updated currently.

The key SuDs Design principle is to provide sufficient storage, and usually some form of outflow control, to mimic that of antecedent conditions prior to development and prevent runoff from entering watercourses at a rate greater than the Greenfield conditions. This is calculated using various methods and is normally imposed as a condition of planning (CIRIA, 2007). Dependent on the design life and risk the storage requirement is estimated form the predicted rainfall data for the area in question for a specific storm duration and return period, usually using the HR Wallingford Procedure (1981) or recent adaptations of that approach (CIRIA, 2007). Normal practice is to add a percentage surplus onto design storm intensities to account for the influence of climate change (e.g. 20%).

There is no standard value for a permitted discharge /greenfield runoff rate (as this is agreed in planning), though during the project discussion with installation contractors suggested figures around 5-7 L/s/ha were typical.

Flood risk and drainage design are based on statistical probability of a rainfall event occurring based on past records, hence there is a probability that the capacity of a drainage system will be exceeded during its design life. Thus a balance must be achieved between the cost of the drainage system and the risk of a flood exceeding the system design capacity. In general the longer the return period selected for design the lower the probability of exceeding the capacity but greater the potential impact and cost.

The design return period is again normally specified as a planning constraint based on the potential risk and impact of any flooding or run off. If the flooding could affect property a higher design return period will be specified, compared to where there is a minimal risk, (CIRIA, 2007 - SuDs manual).

The Conceptual Framework for Effective Storage and Attenuation in a SuDs system is shown pictorially in Figure 1.

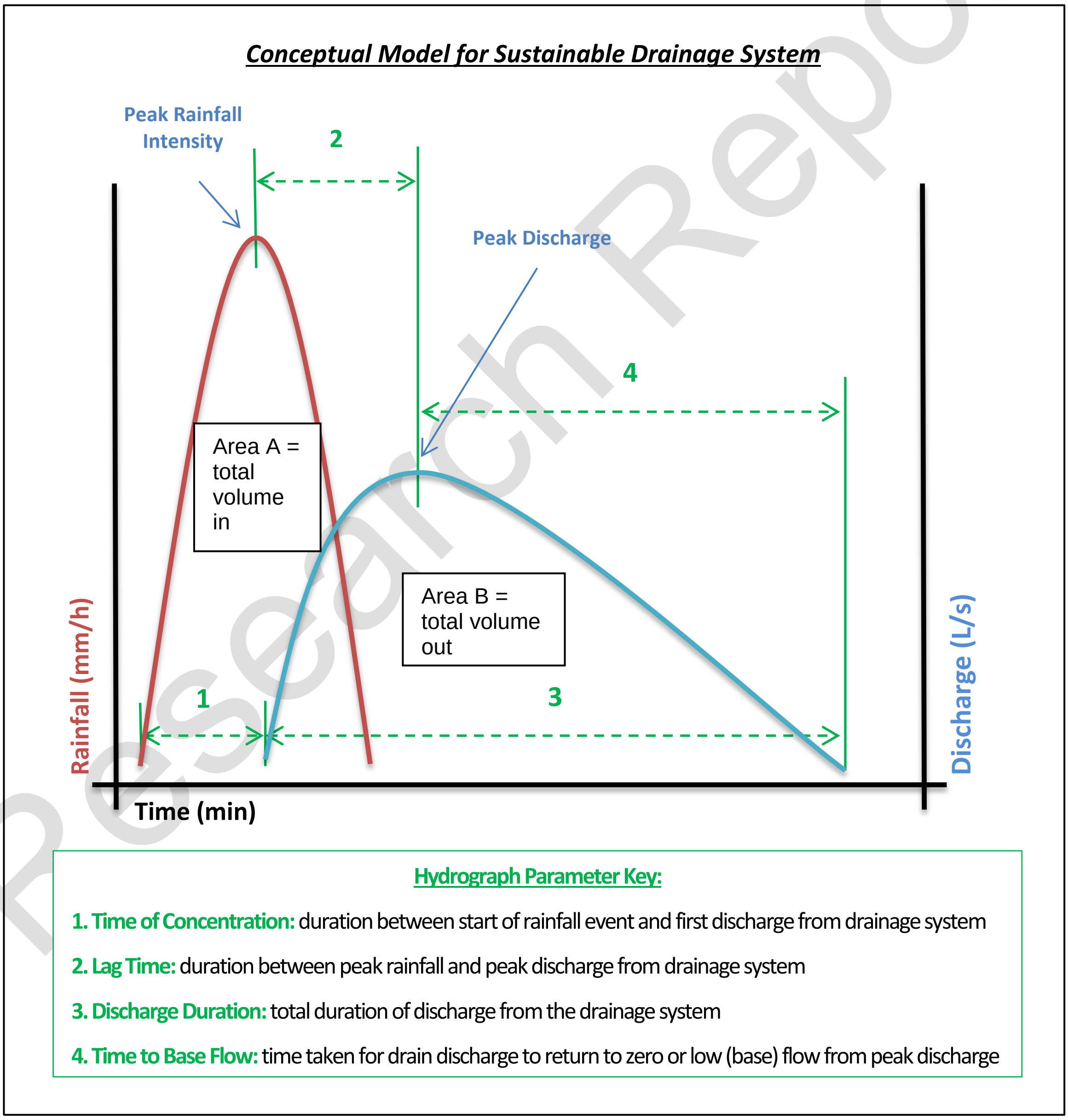

Figure 1. Conceptual model of a drainage system, showing the hyetograph of rainfall intensity (mm/h) and the outflow hydrograph, plotted against time.

In addition to the key input parameters of peak rainfall intensity and duration, and drainage system discharge rate, additional parameters to describe the system behaviour include:

- The time taken from start of rainfall to the point when a first response is recorded at the drainage outfall (1. Time of Concentration, ToC); *Note: ToC has also been expressed differently by practitioners as the time to peak discharge.*
- The duration between the highest intensity rainfall and the corresponding peak in discharge (2. Lag Time);
- The total length of time that water drains or is yielded from the pitch (3. Discharge Duration);
- The time taken for the rainfall event to dissipate and drainage discharge to reach baseline conditions (4. Time to Base Flow).

From Figure 1 it can be observed that the total volume of rainfall is estimated by Area A, the intensity versus time. The Area B represents the total volume discharged at the outfall. An effective attenuation and storage system would behave such that there is effective resistance to water flow leading to a time interval between the rainfall event and discharge flow, and a time lag between peak rainfall intensity (inflow over time) and the largest rate of discharge. In any piped drainage system there is a time lag between rainfall and drain discharge depending on the length of drainage run, type of drain (i.e. roughness coefficient) and gradient. In a sports pitch the time lag is further extended by the resistance to flow through the pitch layers and horizontally to the sub-surface interceptor drains.

The outflow peak discharge, relative to the peak rainfall intensity is termed the 'attenuation' of the system, a ratio expressed as a %. A larger attenuation is positive in reducing flood risk downstream, however it is also important to consider the actual discharge flow rate. There is a consequence of high attenuation however in the need for effective storage of the volume of water being 'held' in the system.

To attenuate very high storm intensities to some appropriate level of outflow rate requires adequate storage within the system. If insufficient storage is provided the water level will back up through the system and cause ponding and uncontrolled surface water runoff at the facility. The volume of storage required is a balance of the storm return period, the outflow constraint and the design storm that requires most attenuation. In Figure 1 storage requirement is assessed from the area of the hyetograph (Volume A, total rainfall volume) from which the overlapping area of hydrograph B is subtracted (total water volume that has discharged before the end of the storm). If the water collected within the pitch is also designed to drain through a porous subgrade soil, the storage volume required will be further decreased. The design of appropriate attenuation and storage for a sports pitch is an iterative calculation and can be laborious by hand. A number of proprietary software systems have been developed with various add-on packages and graphical interfaces for design procedures. These require inputs of a range of typical design storms, locations and return periods, and an outflow hydrograph based on a design flow control (based on the planning constraints, as described above). These design constraints are then run through mathematical models of the designed drainage and storage system making allowance for infiltration rates, time of concentration and time of flow in pipe networks, to calculate a design storage volume required to attenuate flows (Ciria, 2007 - SuDs Manual).

While there are currently no packages with specific sports pitch modules, designs have been undertaken using an approximation of the systems using either green roof or permeable pavement modelling packages where the input parameters and performance constraints of the drainage system are observed to be similar.

1.2 Sports Pitch Construction & Drainage

Key elements of an artificial sports pitch are shown in Figure 2. In addition to showing the typical UK layered construction, labelled on the left of the diagram, the drainage characteristics and mechanisms are shown on the right of the diagram. It is clear that the drainage design of pitches, utilising materials with high voids such as the porous asphalt and low fines sub-base, provide in principle a low resistance to water flow (relative to soils or densely compacted well graded aggregates). In addition, the void spaces provide storage potential for water volumes. Artificial carpets are often manufactured in such a way that the backing is impermeable although drainage holes are then added to promote surface water infiltration.

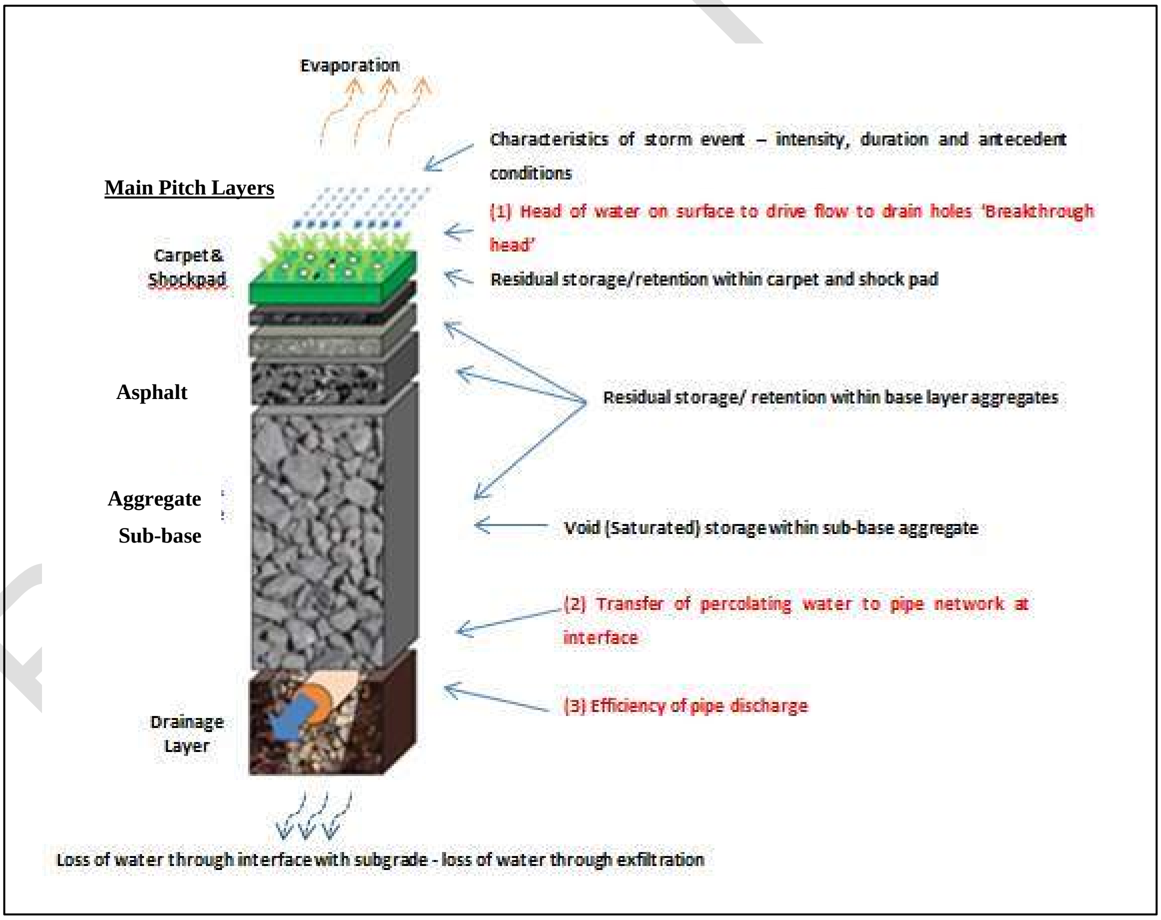

Figure 2. Schematic of a typical artificial sports pitch construction, identifying the construction layers and possible drainage mechanisms and characteristics expected and similar to a SuDs drainage system.

The design normally required the carpet and supporting shockpad and asphalt to have a high infiltration rate (150mm/hr) often assessed using a ring infiltrometer with a head of 100mm. The head and infiltration far exceed typical storms and make allowance for reduction in infiltration over time due to wear and clogging of the carpet.

Drainage of Natural Turf Pitches (NTP) was also considered within the project scope, and Figure 3 shows a typical cross-section through such a pitch. The sand slit drains, excavated slits backfilled with fine gravel and sand/sand rootzone materials are typically 50mm wide, are at up to 1m centres to a depth of 300mm. These are perpendicular to the lateral gravel-pipe drains at 3-4m centres, which deliver the water flow into collectors/main drain. In addition, natural turf pitches are usually laid with a surface fall to further assist surface water runoff (Artificial Turf Pitches (ATP's) in contrast are usually relatively flat).

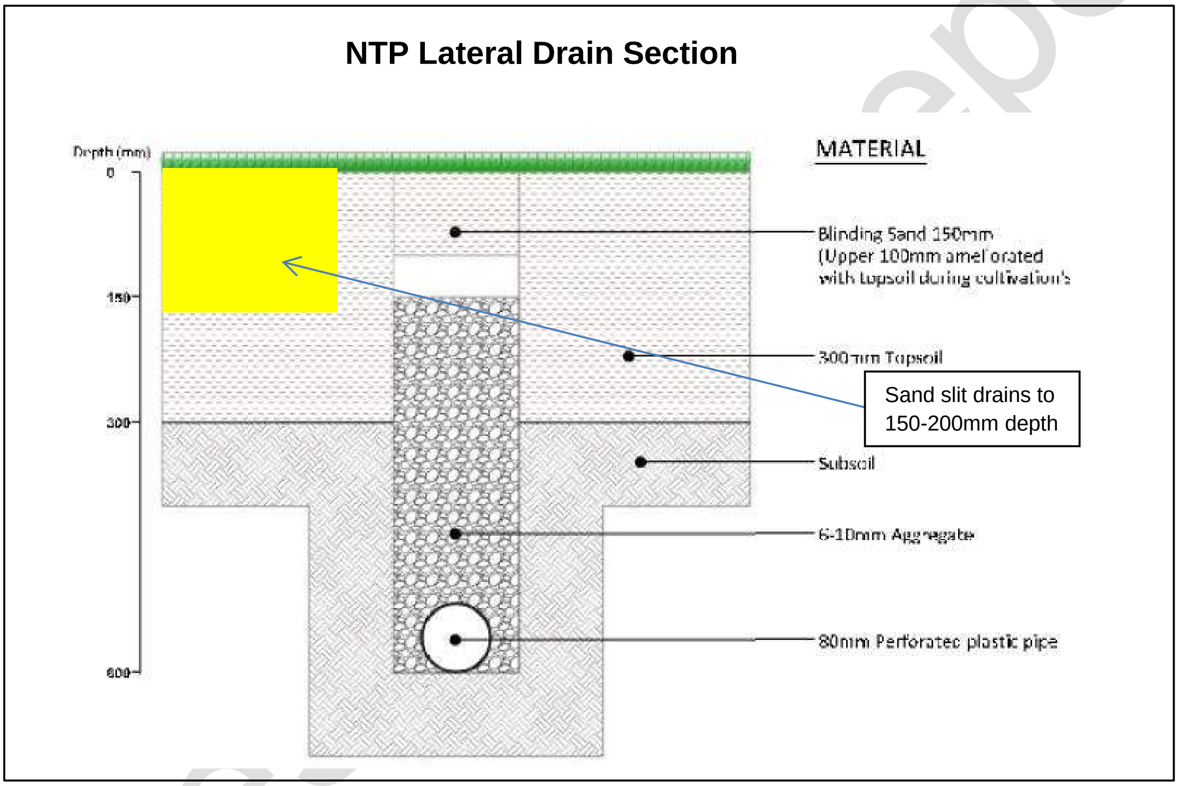

Figure 3. Typical cross section of a lateral drain at a NTP, also showing the sand slits running perpendicular. Main drain detail is similar to the lateral with a wider trench and larger 150mm diameter pipe.

It was considered that in essence the same principles applied to both pitch construction types. However there was a natural bias toward artificial pitches within the project scope, partly due to their expected more uniform and consistent inorganic materials. The study set out to investigate the hypothesised mechanisms and drainage performance explained in Figure 2.

2.0 RESEARCH FINDINGS

2.1 Introduction

The study incorporated three methods of research, fieldwork, laboratory work and mathematical modelling. These are summarised in the sub-sections below.

2.2 Fieldwork – collected evidence of pitch drainage performance

The monitoring of in-service sport pitches was undertaken during the study, around England. A site selection screening process helped identify suitable sites including requirements for full ground information, drainage plans, single outfalls and suitable monitoring chambers. Initial screening of suitable sites produced 28 for further appraisal through visits and data mining. From these sites 8 were identified as most suitable and monitored for varying periods of time between late 2011 and 2014.

Early work identified problems with the industry standard flow devices in achieving detailed flow rate records across a full range of flows and for extended periods (to avoid high frequency of site visits). As a consequence the project team devised and built in-house bespoke flow measurement devices, termed Flo-pods, to continuously log flow rates to a resolution of 0.01 L/min. At the natural turf pitches, calibrated flumes were installed to continuously log flow rates to a resolution of 0.001 L/min. Weather stations were also installed at the sites to collect the local environmental conditions, detailed local rainfall records were a priority.

Fieldwork is always beset by challenges for research, whereby controlled and consistent conditions are near impossible and the harsh measurement environment challenges the most rugged of technologies. Regardless, the project achieved a good set of data over many months at each site across both artificial and natural pitches. This large data set is represented only in brief here to illustrate the observations and trends recorded.

2.2.1 Artificial Turf Pitches (ATP)

An example of a short sequence of rainfall events and associated unconstrained drainage flows from an ATP is shown in Figure 4.

The ATP monitored comprised a typical 3G pitch comprising: 40mm monofilament pile with sand/rubber infill; 40mm porous asphalt base; 300mm aggregate sub-base: plastic clay subgrade (classified as weathered M Mudstone, some cut and fill, low permeability, low to medium plasticity). Collector drains discharged freely to a single outfall. Figure 4 analysis shows that a total rainfall of 13mm, or approximately 95000 litres volume of water, landed on the pitch (pitch area 7530m^2). Peak rainfall intensity was recorded as 2.5mm/hr, and 7mm of rain had fallen in the 5 days preceding the events shown here. During this 6-7 day long period of rain the discharged outflow showed generally a low continuous discharge with occasional peaks. The outflow peaks clearly follow the rainfall peaks as expected, with a lag time of typically 4-7 hours. The peak outflow recorded was low, at 0.2 L/s (equivalent to 0.27 L/s/ha). The total outflow volume was measured as close to 30,000 litres, or approx. 32% of the total rainfall volume. The discharged outflow was spread over a further 35-70 hours. The peak discharge outflow is estimated as an equivalent rainfall of 0.08 mm/hr, and when then compared to the 2.5mm/hr peak rainfall can be expressed as 'attenuation', in this case around 95%. This represents a large reduction in the potential discharge intensity if the rainfall were collected more efficiently (e.g. for an impervious hard-standing area). The shape of each discharge event closely resembles the expected pattern shown in the conceptual model in Figure 1. The data clearly demonstrate resistance to flow of infiltrating rain water provided by the porous pitch constituent layers and drainage system components.

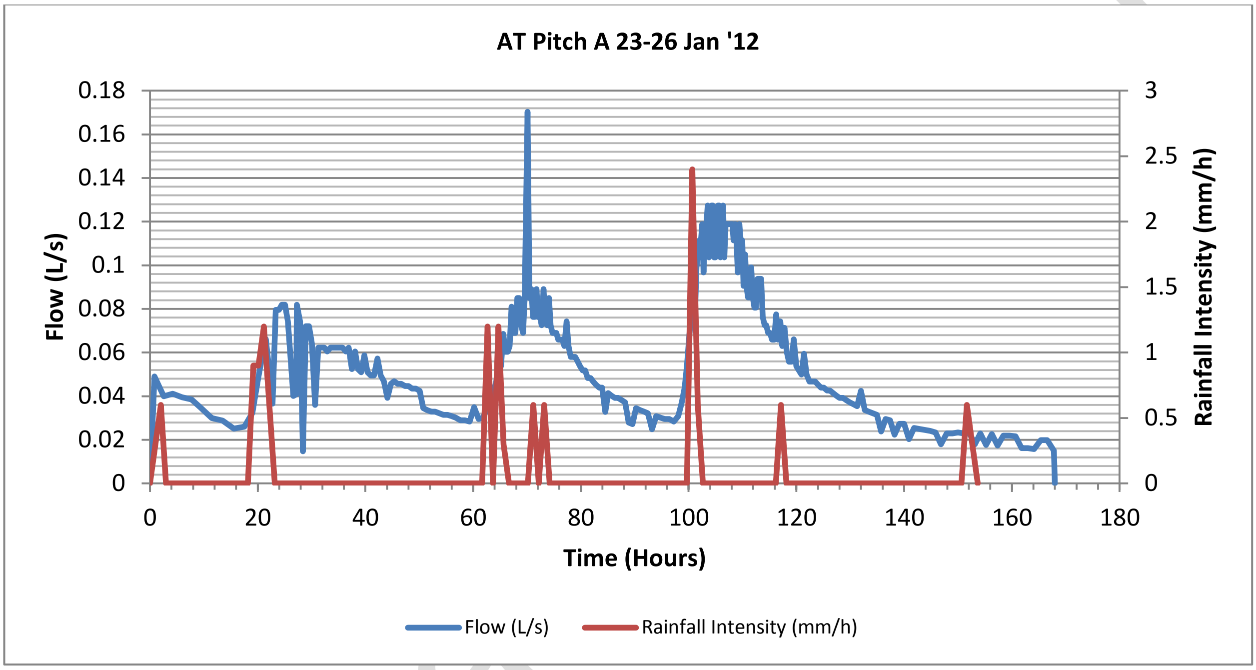

Figure 4. A typical series of short rainfall events and the associated pitch discharge at an ATP. (Note: 1000 minutes = ~16.7 hours, 3000 = 2 days).

Table 1 presents a series of rainfall events from this and other artificial turf porous pitch builds. The antecedent precipitation (AP5) parameter represents the amount of rain in mm that fell up to 5 days prior to the monitored event. The AP5 values are included as it was expected that the previous rainfall may affect the yields and flow rates generated in subsequent events. However no clear relationship emerged. The main points that emerged from the monitoring of ATPs in this study are that low yield figures and low peak flow rates were observed. This was in spite of the intensity of the storm event monitored and any pitch specific design.

For the sites monitored the ground conditions were such that little exfiltration was expected into the subgrade, and little losses due to evaporation were likely.

Table 1. Selected rainfall and discharge events from a series of artificial turf pitches.

Rainfall Event	ATP1	ATP2	ATP3	ATP4	ATP5	ATP6	ATP7
	Dec 2011	Dec 2012	Jan 2012	Jan 2012	Jan 2012	July2013	Apr 2014
Antecedent Precipitation (5 Days) – AP5 (mm)	0.4	15	7	6	8	5.4	6
Total Rainfall (mm)	8.0	4.8	4.2	3.9	4.5	19.6	7.4
Total Rainfall Volume (L)	60,208	36,125	31,609	29,351	33,867	69,600	26,300
Rainfall Duration (h)	~110	~9	5	3	3	3	4
Peak Rainfall Intensity mm/h	1.6	1.6	1.2	1.2	2.4	5.8	2.2
Total Volume Discharged from Pitch Drain (L)	11,669	483	4,040	2,086	11,823	8133	113
Peak Flow Rate (L/s)	0.130	0.007	0.082	0.170	0.127	0.1	0.01
% Yield	19.4	1.3	12.8	7.1	34.9	12	0.5
Time of Concentration (h)	1.7	62.4	17.55	0.7	0.18	1	6
Lag Time (h)	13.3	65.0	3.9	7.0	2.8	12	38
Discharge Duration (h)	131.1	138.4	45.0	34.6	76.2	41	25
Time to Base Flow (h)	23.8	134.6	34.0	30.6	62.3	60	23
Antecedent dry period (h):	21.5	40.4	48.5	40.5	1.0	25	25
Attenuation of Peak Flow (%):	96	99	97	93	97	98	99

2.2.2 Natural Turf Pitches

Examples of rainfall events and associated drainage flows from one of the monitored NTPs are shown in Figures 5 and 6, selected to demonstrate varying yield % of discharge volume versus rainfall volume.

The event in Figure 5 gave a total rainfall of 7.6mm, at a peak intensity of 4mm/hr, over a period of approximately 4 hours. The peak drainage outflow reached 6.9 l/s (4 L/s/hectare) approximately 2 hours after the peak rainfall. The pitch outflow continued for around 7 hours. The volume out recorded was approximately 74% (yield) of the rainfall volume, with an attenuation of 65%.

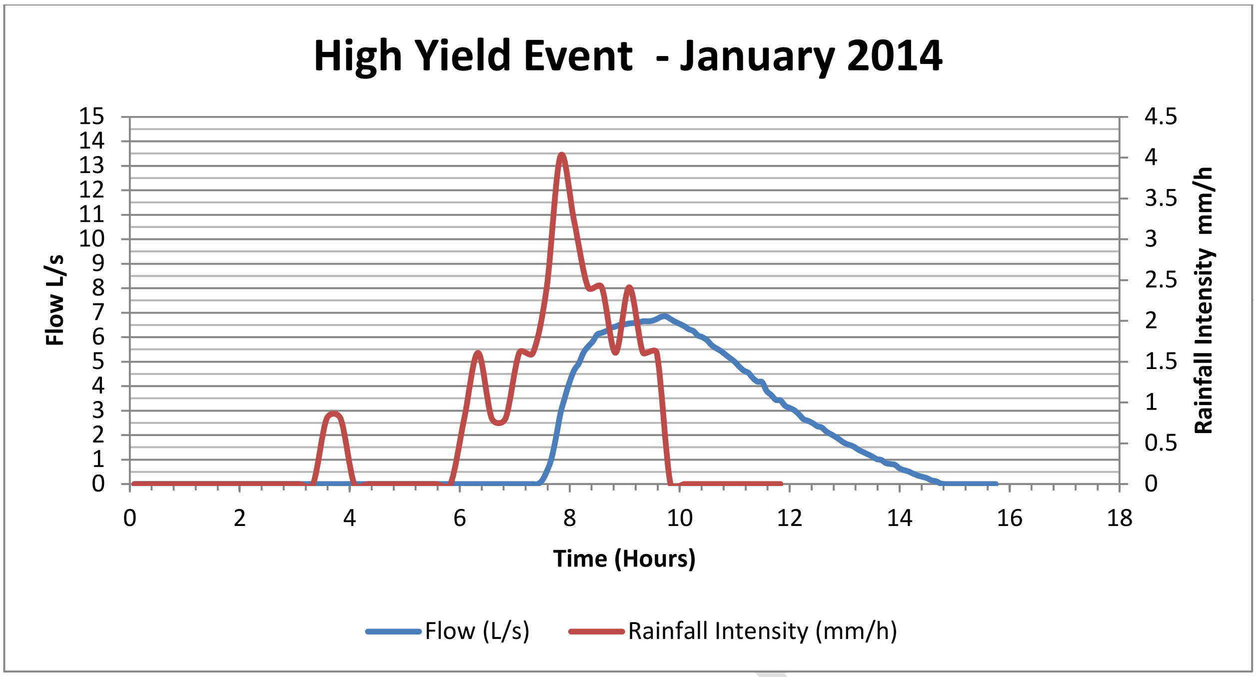

Figure 5. Example of a rainfall event which produced a high yield. In this case antecedent conditions were similar to the event. Note the similarity to the conceptual drainage diagram in Figure 1.

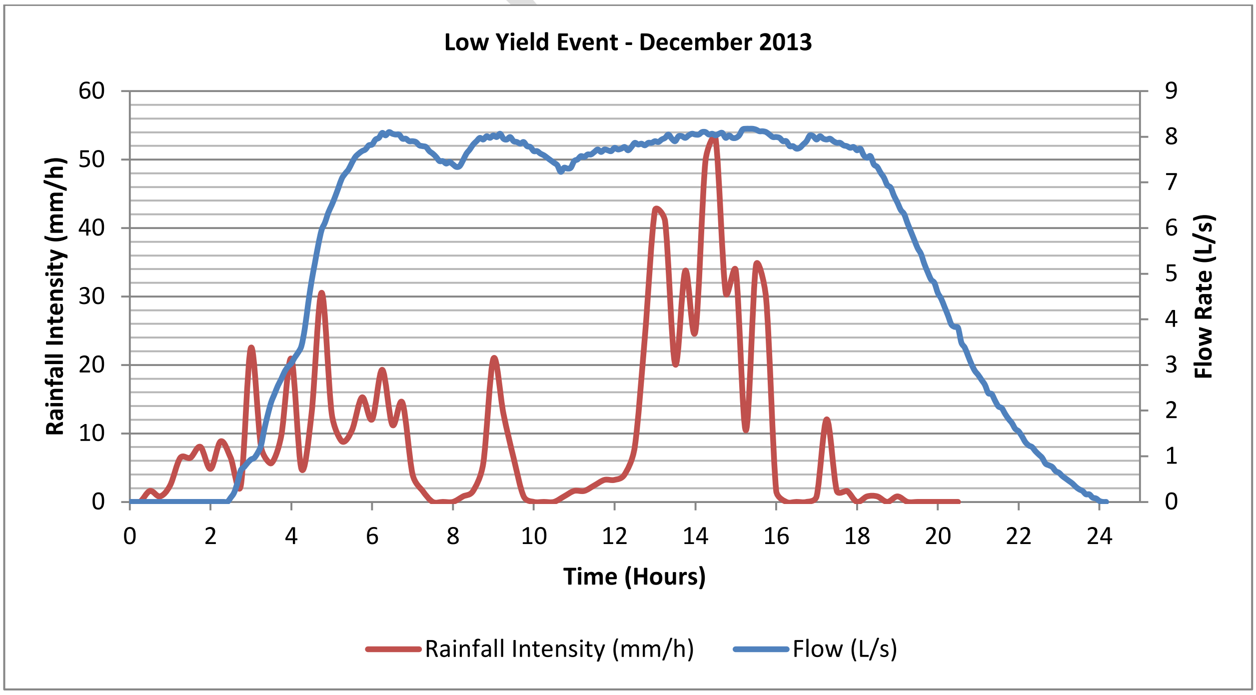

Figure 6. An NTP event recorded from December 2013, showing a very high large storm event.

The key outcomes from analysis of Figure 6 are: a total rainfall of 199mm over 18 hours; a peak rainfall intensity of 53mm/hr; a peak discharge flow rate of 8.2 L/s (4.8 l/s/ha) was reached at the pitch outfall and this was sustained for around 11 hours. In contrast to the event in Figure 5 the total volume out gave a yield of 14% and an attenuation of 97%. (Note the flume capacity for full bore flow is around 12 L/s such that the flow rates seen here are not yet at the limit of measurement)

Whilst Figures 5 and 6 show very contrasting behaviour in terms of yield (% ratio of volume out/in), the discharge flow rates are high (relative to those monitored at the artificial pitches). Nonetheless the data demonstrate a clear resistance to flow and reduction in discharge intensity relative to the storm. These higher flow rates are below the industry reported greenfield run off rates of around 5-7L/s/ha.

A summary of the most notable events at the NTPs monitored are given in Table 2. The largest rainfall events measured occurred in the winter months.

Note: Pitch NAT1-4 data is from two grass pitches of total area 17160m^2 with a single drainage outfall. Pitch NAT 5-7 comprises two grass pitches *and* an (artificial) athletics track, to a single outfall. The total area is approximately 2100m^2.

The drainage design at the two NAT sites is very similar to that shown in Figure 3. From the surface, water can flow through 50mm wide sand slits located at 260mm centres to a depth of 150-200mm. The slits are traversed at 150-300mm depth by gravel trenching up to 450mm deep which houses 80mm lateral perforated pipes at the base laid at 3m centres diagonally across the pitch. The invert of the 80mm perforated pipes are located approximately 600mm from the surface with a fall of 1:100, into a mixture of 90mm and 100mm perimeter and collector drains. The perimeter and collector drains converge to form a terminal 150mm collector pipe at a fall of 1:200. The formation soils at both sites were reported as of very low permeability in the site investigation reports. Furthermore, site measurements using soakaway tests showed little to no subgrade drainage capacity.

The Table 2 summary data demonstrates some interesting behaviour trends observed from the wider data set. In many cases high rainfall intensity led to lower yield, and lower rainfall intensity higher yield. Times of concentration and lag times were both of a few hours, suggesting some water reaches the discharge in relatively short times in relation to the artificial pitches (Table 1). Antecedent conditions bore little discernible relationship to discharge behaviour, as found for the artificial pitches. The highest discharge peak recorded was 9.8 L/s (around 5 L/s/ha), but yields from the larger rainfall events were low whilst smaller rainfall events gave higher yield. Overall the attenuations were high with an average greater than 90%, the smallest was 64%.

Table 2 Selected rainfall and discharge events from a series of natural turf pitches.

Rainfall Event	NAT1 Nov 2013	NAT2 Dec 2013	NAT3 Jan 2014	NAT4 Mar 2014	NAT5 Nov 2013	NAT6 Nov 2013	NAT7 Nov 2013
Antecedent Precipitation (5 Days) – AP5	1	59	7	21	26	56	60
Total Rainfall (mm)	4.2	199.0	7.6	12.1	14.1	36.6	5.1
Total Rainfall Volume (L)	72,432	3.41 M	131,068	206,950	351, 250	915, 750	126, 500
Rainfall Duration (h)	2	18	4	12	7	7	2
Peak Rainfall Intensity mm/h	11.3	53.1	4.0	6.4	24.6	16.6	11.0
Total Volume Discharged from Pitch Drain (L)	9,649	462,472	97,518	128,900	3, 381	115, 029	5, 715
Peak Flow Rate (L/s)	3.7	8.2	6.9	4.9	0.8	9.8	1.8
% Yield	13.3	13.5	74.4	62.3	1	13	4.5
Time of Concentration (h)	1.1	2.4	3.9	1.2	2.3	4.4	2.2
Lag Time (h)	0.5	0.3	1.9	0.8	2.5	2.2	2.2
Discharge Duration (h)	2.2	21.5	7.3	11.9	3.5	11.4	11.4
Time to Base Flow (h)	2.1	3.4	5.1	8.8	3	9	3
Antecedent dry period (h):	40	22	62	46.8	7	5.4	3.6
Attenuation of Peak Flow (%):	93	97	64	84	98	90	96

Note – natural turf pitches founded in low permeability clay soils.

2.2.3 Summary Outcomes from Field Monitoring of Pitches

The general trends in drainage discharge from the monitored sports pitches, regardless of system (AGP or NGP), were anticipated to show greater discharges for higher intensity and longer rainfall events and wetter antecedent conditions. However, the monitoring results showed limited trends which support this expected behaviour.

The fieldwork results were also assessed to determine how the rainfall events observed compared to the typical range of ‘synthetic’ rainfall events used in drainage design from the Wallingford procedure. A range of synthetic rainfall events for two return periods (1 in 5 year and 1 in 100 year), which varied in duration from 5 minutes to 2 days, were considered for average rainfall intensity (mm/h) and total event rainfall depth (mm). This demonstrated that in general the rainfall events observed at the artificial pitch sites rarely exceeded a 1 in 5 year storm whereas a the natural turf pitches some events closely matched 1 in 100 year events. This is unfortunate, and limits the direct comparison of the two types of pitch and the artificial pitch behaviour to design rainfall events (i.e. of a longer return period). It further demonstrates the lack of control afforded in field work and to an extent the element of chance in achieving the desired range of data measurements.

It is clear from the whole data sets that for all field sites monitored that the drainage water volumes discharged were much lower than the surface rainfall water volumes.

When contrasting artificial and natural pitch types, the latter generated a broader range of drainage yields (<1%-85%) compared to artificial (<1%-35%). When comparing drainage designs, it is clear that natural pitch drainage comprises very specific vertical drainage connection pathways, the slits and laterals, in comparison to the porous sub-base foundation of artificial pitches which acts more like a thick ‘raft’ and offers potentially greater dispersion of water horizontally as it percolates downwards from the carpet. Furthermore, the artificial pitch subgrade lateral drains are set at a much wider relative spacing, requiring greater horizontal flow distances to reach them. In contrast however, natural turf pitches are designed to ‘hold’ some of the drainage water to feed the plant growth, and might be expected to be at or close to field capacity during the monitoring periods.

The data from the natural turf pitches does suggest that under very intense rainfall events there may be some surface runoff, encouraged by the surface falls/gradients built into the schemes, and in these specific case studies there were open ditches adjacent to hedgerows that would intercept the runoff water.

The field data demonstrate well the ability of sports pitches to attenuate rainfall peak flows effectively and consistently in line with SUDs principles even without a flow control.

The influence of antecedent conditions was not observed to have the expected effect on the results. Whilst efforts were made to isolate events by considering the AP5 index value and looking for singular events wherever possible, without knowing the actual water/moisture conditions within the pitch it is difficult to provide an accurate representation of the true antecedent conditions.

It should be noted that each site has site-specific natural soil conditions and variability in the detailed construction methodology and accuracy of as built drawings is expected. The field locations were active pitches such that there was limited opportunity for invasive site investigation work that could be carried out in this study. However the site investigation information was more readily available for the natural turf pitches monitored. Perhaps of note is that one artificial turf pitch that was monitored for several months produced no discernible discharge flow at all, and despite some further investigation was abandoned for further study. It has to be assumed at this site that the drainage was either not connected properly or had a significant blockage, it had a reportedly low permeability subgrade. The pitch surface was not observed to flood by the groundsman however.

2.3 Laboratory Results – Storage Capacity and Flow

A series of laboratory experiments were carried out to establish, with some control, the hydraulic behaviour and capacity of the pitch system components, with an emphasis on the artificial pitch construction. The experiments utilised a combination of small and large sample sizes to evaluate individual pitch element behaviour. A large test rig with a pitch section was then also used to test hydraulic performance under simulated rainfall.

2.3.1 Carpet – breakthrough head and retention

The carpet and infills demonstrated some resistance to water entering the sub-surface. This resistance is provided by the tortuous route of water flow across and through the carpet surface and infills to the drainage holes. This resistance is somewhat dependent on the carpet hole size and spacing, for the tufted carpet systems with impermeable backing. Similarly there is resistance to flow through the carpet fabric in needle punched systems. In addition the porous shockpad beneath the carpet also inhibited flow to some extent, dependent on its void space. The depth of water required initiating flow across and down through the carpet/infill system is termed the 'breakthrough head'. Until this head is achieved there is limited flow across the carpet such that runoff is not an issue.

The typical value for breakthrough head for all the carpet systems tested with drainage holes, on an open textured 'insitu' shockpad, was the equivalent to around 5-6mm depth of rainfall within the carpet. The inclusion of a dense prefabricated shockpad beneath was observed to increase this to up to 10mm.

However once flow is initiated the water drained readily into the lower pitch construction. Thereafter, some water is retained in the carpet/infill system to maintain a head to drive flow. The water retention values observed were in the range 2-5mm depth of water.

The magnitude of the breakthrough head and surface water retention does suggest that, if initially dry, for many lower intensity rainfall events there is likelihood that no water will flow across the carpet or down to enter the subsurface. However, if initially moist or very wet already then breakthrough may occur relatively quickly and infiltration commence.

2.3.2 Component Material Storage Capacity

A further series of tests evaluated the potential storage volumes, flow behaviour and residual storage of the pitch component layers.

The summary data provide further evidence of the ability of a sports pitch to retain and store large rainfall volumes, equating to large intensity storm events.

Table 3 provides a simple summary of the maximum available storage if the pitch became fully saturated from dry. In addition, 'water retention' (WR) values are provided that represent the amount of water retained after free drainage of the percolating water through the fully saturated materials under gravity. WR effectively represents the potential volume of water held in the system (by adsorption forces) on the surface of the materials. The values of saturated storage are presented as a total for a typical pitch construction and as litres per m^3 of the component material layer to permit simple calculation of saturated storage potential for other construction designs. The table also includes the potential storage for the pea gravel used as a bedding material in the drainage trenches.

Table 3. Artificial pitch components, showing the typical percentage voids in the layers, volume of water retained after saturation, and estimates of the total storage capacity.

Material	Total Volume of Material (m^3)	(PV) Percentage Voids (%)	(WR) Water Retention (L/m^3)	(WR) Water Retention (L/pitch) 1000s	(SS) Saturated Storage (L/m^3)	(SS) Saturated Storage (L/pitch) 1000s	Storage depth of water (mm)
Carpet/infill	300	30-70	--	--	--	15-37	2-5
Shockpad	113	45	84	9	450	51	9
Asphalt	488	22	66	32	220	107	14
Sub-Base	2250	24	24	54	240	540	73
Pea Gravel	44.0	10	--	--	100	4	1-2
			Totals	**95**		**720**	**97mm**

The storage depth of water is an estimate of the equivalent height of a column of water for the specific material layer at full saturation, and can be compared to rainfall total depths in Table 4. These values however depend on construction thickness values, in this example they were 300mm (low fines, often termed Type 3) sub-base, 65mm asphalt, 15mm shockpad. A typical pitch area of 7500m^2 was assumed to estimate total saturated storage.

Retention and storage are not mutually exclusive, however, such that full storage includes the retention volume. As a consequence if the pitch materials are already at WR through antecedent conditions then the further ***available*** storage is SS-WR, around 650,000 litres (650m^3) in this case.

An artificial pitch represents a construction volume of approximately 2800m^3, and the storage volume estimated represents an overall average void space of nearly 25%. The data show if this were fully utilised during very high intensity storms, and with no flow discharged, the pitch could theoretically hold the water volume from a typical 1 in 100 year storm that lasts for two days, see Table 4.

The thick sub-base layer is clearly a major contributor of the potential storage capacity. For a low fines compacted aggregate the expected percentage voids of around 25% is in accordance with figures used by industry designers. Void space is affected by both the particle size distribution and compactive effort applied, which controls the particle packing (density). The hydraulic behaviour, i.e. flow rate of water, is controlled by the permeability of the soil medium and the 'head' of water driving the flow (for simple saturated flow). The permeability of coarse sub-base materials is not readily measurable but in the study a figure of $7x10^{-3}$ m/s was achieved from a constant head permeameter test. This figure agrees with the expected range of 10^{-1} to 10^{-3} m/s expected for coarse gravels. In contrast fine grained silt and clay soils usually have permeability in the range of 10^{-6} to 10^{-9} m/s (when unstructured). These values are provided to demonstrate the high porosity and low resistance to flow of the sub-base relative to many subgrade soils. Furthermore fine grained soil permeability is especially difficult to measure accurately in the laboratory or the field (e.g. in soakaway tests).

Table 4. Estimated rainfall volumes on a sports pitch area 7500m^2 for a return period of 1 in 100yr year and different duration storm intensities from the Wallingford procedure (for the Loughborough area)

Duration D (min)	5	10	15	30	60	120	240	360	600	1440	2880
M100-D Total Rainfall Depth (mm)*	12	18	23	30	38	46	55	60	69	78	92
Volume in Litres (1000s)	90	135	172.5	225	285	345	412.5	450	517.5	585	690

Note for natural turf pitches, the storage volume has been estimated at around 30m^3 per full-size pitch (so ~60m^3 at each of the two sites presented above in 2.2.2) for the gravel filled lateral drainage channels, assuming 10% void ratio (approx.. 21m^3 void space) and including the 80mm pipe void (approx. 9m^3) but ignoring the main carrier drain. This represents a much smaller potential storage volume than expected for the sub-base in Table 3, although it ignores the potential storage in the soil pore spaces. The detailed mechanism of water flow and storage in partially saturated fine soils (small pore spaces) is relatively complex and is expected to be largely affected by the antecedent rainfall conditions.

2.4 Mathematical Modelling

2.4.1 Model Design

A relatively simple mathematical model was constructed to estimate the theoretical behaviour of the input rainfall through to output discharge from the pitch, and investigate the mechanism at work regarding attenuation further. The model was constructed in Microsoft Excel with open architecture to permit staged analysis of the processes and sensitivity analyses of the key influencing parameters.

The modelling steps made simple assumptions to ensure conservatism, or worst case, such as evaporation of surface was ignored as a potential loss, and the subgrade was modelled as impermeable.

The model was constructed to simulate the typical artificial pitch cross section shown in Figure 7, and included lateral drains (80mm diameter, corrugated and perforated), a single collector drain (150mm diameter, corrugated solid wall) and a single outfall discharge point, see Figure 8. The model stipulates discrete points for calculations, termed nodes, and considered columns of pitch of 1 square metre in area. The lateral drain nodes intercept water from the sub-base at 1m centres, i.e. 60 nodes across the pitch. The model assumed a 5m spacing between the lateral drains, and the 21 lateral drains discharge at end nodes into the collector.

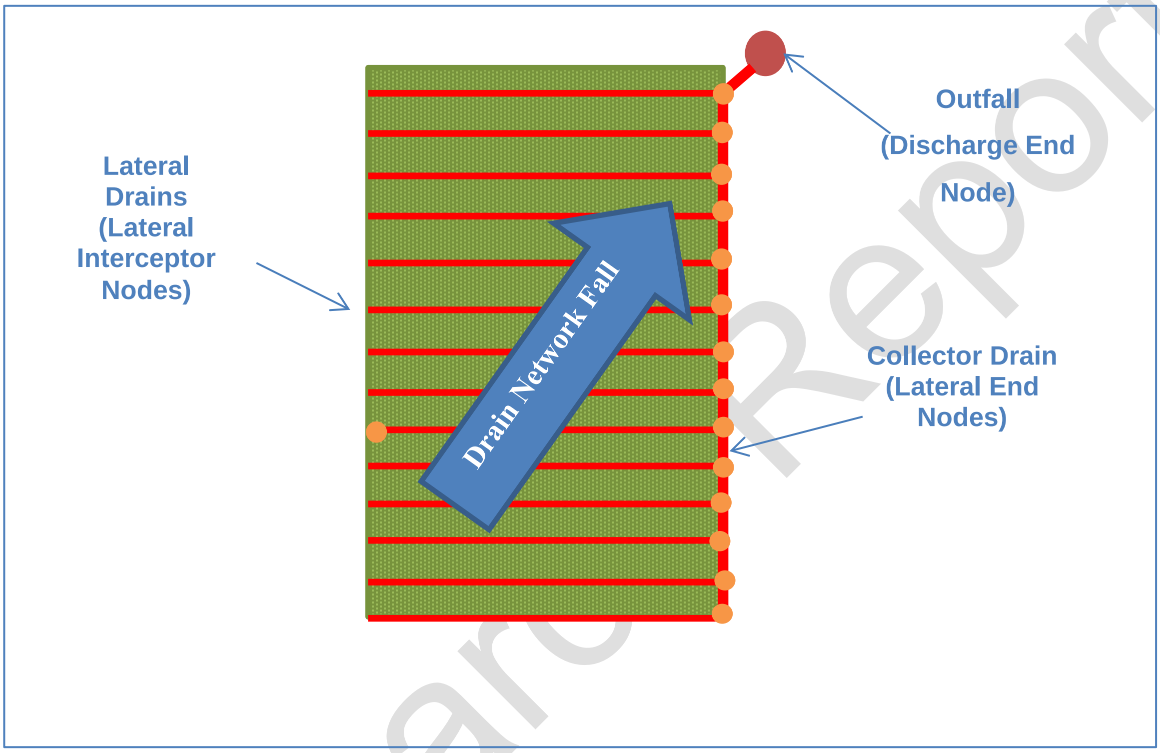

Figure 7. Schematic of the drainage layout evaluated in the mathematical model

Pseudo static and dynamic equations are utilised in the model, the key steps are set out below.

1. Rainfall is simulated, as depth of surface water versus time (using the accepted Wallingford procedure, choosing typical mid-range input data).
2. As water percolates vertically through to the bottom of the sub-base estimates of retention are made through the layers
3. The flow of water from the sub-base horizontally into the lateral drain node (per m length) is determined versus time
4. Cumulative flow is estimated for each 60m lateral drain versus time, at the point of discharge to the collector drain.
5. Cumulative flow from the collector drain nodes is estimated to provide the discharge flow rate (hydrograph) at the outfall.

Step 3 comprises the hydraulics of water flow in soils, using simple flownets and horizontal permeability of the granular sub-base (approximately 10^{-2} m/s). For steps 4 and 5 the time related 'dynamic' nature of the flows in and out at each lateral node comprises the hydraulics of water flow in round pipes with a fall. The cumulative behaviour of flow in the lateral; pipes assumed low pipe friction (despite the perforations and corrugations which is a conservative assumption) whereas the collector pipe included for high pipe friction due to the corrugations.

2.4.2 Key Outcomes

The model predicted, in relation to a standard 1 in 100 year event a total flow out of the system represented as yield % in the range <1% to around 26% for the storm durations of 5 minutes to 2 days, and attenuation figures of between 10 and 50%. This suggests, theoretically that 75% or more of the rainfall volume is 'held' in the system. The model further identified that much of this 'loss' was held in the pipe network due to the high friction resistance. The model also identified that only for very long continuous storm durations of greater than a whole day did more than half the rainfall volume actually find its way into the pipe network due to the resistance in horizontal flow through the sub-base to the lateral drains. The model could not readily simulate complex antecedent conditions however, and was based on first time events.

The predictions do not (and cannot) match the field results to any degree of accuracy due to the many system variables (such as head loss/ system flow friction, wind, humidity, evaporation, subgrade exfiltration etc.). However, the model does show similar trends and patterns identified from the field data observations, and allowed for further analysis of the sensitivity of the outflow parameters to the system hydraulic mechanisms, discussed below.

Once rainfall has landed onto the pitch surface, there is a time delay in the water reaching the breakthrough head required to drive flow through to the sub-surface layers. There is also potential 'loss' of rainfall volume here stored within the carpet and infill. The carpet storage (if relatively dry prior to the rainfall event) is the equivalent volume of a low intensity storm event.

The advancing water flow is then expected to reach the lower sub-base relatively quickly under continued rainfall creating sufficient 'head', though there is again potential for rainfall volume 'loss' in water retained (adsorbed) in the layers of shockpad, asphalt and sub-base. This combined 'residual' storage in the pitch layers is sufficient to limit any discharge from a low to medium intensity storm event (if relatively dry prior to the rainfall event).

A barrier to the rainfall entering the sub-surface lateral pipe network is the resistance to horizontal flow across the interface of the sub-base to subgrade (and within the sub-base itself). This is dependent on the sub-base permeability, the drain spacing and the build-up of sufficient head required forcing flow to the drain. However for typical values of 5-10m spacing the horizontal flow rate and volume is expected to be minimal until the depth of water in the sub-base reaches around 50mm (with 25% void space this means rainfall depth of at least 12.5mm and no residual losses).

After reaching the lateral drains, a key hydraulic factor limiting the surface rainfall water reaching the outfall quickly is the hydraulic pipe frictional resistance. The lateral drainage pipes are generally corrugated (and perforated) and laid on a fall of around 1 in 100 or 1 in 200 in some cases. In addition the collector pipe is laid at a low gradient and may also be corrugated.

For simplicity it was considered in this modelling that the pea gravel trench around the lateral pipes remained dry. In reality there is a further storage reservoir within the drainage trenches and pipe network. This provides further hydraulic losses through resistance and storage. The pipe network itself offers additional storage capacity (around 6000 litres in the 80mm diameter lateral drains alone for the example given).

It was also considered in the model that no exfiltration into the subgrade occurred to the base or sides of the pitch system. This is a further potential barrier to some of the water flow reaching the collector pipe outfall. The large base area of around 7500m^2 represents a major potential source of water volume 'losses' ***if*** the subgrade has capacity for accommodating some flow (controlled by the subgrade vertical/horizontal permeability and its water content). SUDs experience has shown recently than even in relatively low permeability soils the low rates of flow can help dissipate water volumes over time. This is further discussed in Section 3.

In summary due to hydraulic resistance in the whole system the head of water generated by the rainfall at various layers in the system is, it appears, seldom sufficient to overcome internal flow friction and generate significant flow rates to the outfall. This supports the reduced flows observed in the fieldwork and the attenuation of rainfall, even in the unconstrained systems.

3.0 Discussion Points

The data collected though fieldwork, lab and modelling has met the aim of the project and demonstrated the performance and behaviour of pitch drainage mechanisms.

Whilst there was a focus on artificial turf in the laboratory and modelling, the same principles, in essence, apply to natural turf. However, it is considered that the near surface drainage mechanisms of natural turf are more complex due to their organic nature and finer grained soil systems.

From the fieldwork it is apparent that in general natural turf drainage systems are designed to be more efficient at removing surface water through the combination of relatively closely spaced sand slits and sub-surface drainage pipes.

The artificial pitches (ATPs) monitored produce high attenuation of peak rainfall intensity, and low discharge rates (typically >90% attenuation and <0.1 L/s/ha. However the field rainfall events monitored were of lesser intensity and duration (i.e. less than 1 in 5year return period) than those monitored at the natural pitches. Whilst there were no data collected for extreme storms at ATPs the general trend showed the very porous nature of the pitch sub-structure acted in a similar way to what are termed ‘source control’ sustainable drainage systems, and the laboratory and modelling work confirmed their potential storage capacity.

The natural turf pitches (NTP) were monitored over a much larger range of rainfall intensities than the ATPs. The storms included events that equate to approximately 1 in 100 year events, and as such can be considered to represent worst case design events. The NTP data produced a broader range in attenuation, range 30-90%, and greater peaks flows up to 5 L/s/ha.

It was concluded however, from the low yields measured during the very high intensity storms at NTPs that some surface water was running off to local areas adjacent to the pitch i.e. surface ditches or low lying areas. It is of course prudent to consider the wider site environs at any sport pitch development regarding the possible fate of any surface run off (and local water run-on) and interceptor drains are a feature of good practice.

The pitches monitored had no flow control (i.e. were uncontrolled discharge) and only in exceptional circumstances, at the NTPs, did the outfall flow rate reach or exceed a flowrate that might be considered to require flow control. A simple flow control such as a Hydro-Break vortex flow control is relatively easy and low cost to install at the outfall chamber, and was observed in place at some facilities. It is clear from the study findings that the storage capacity of the sub-surface porous layers can be effectively utilised to store water during high intensity events, and for an ATP the capacity can be in excess of $500m^3$ for the sub-base alone (assuming a $7500m^2$ pitch area, and 300mm depth of sub-base with a 25% void ratio). This potential storage volume equates to a rainfall event total depth of 70mm, i.e. close to that predicted for a 1 in 100 year storm that lasts a day.

There are, however, possible implications for permitting stored water to sit for extended periods in the aggregate sub-base, which need to be considered in design. The most significant consequence could be possible softening and swelling of the subgrade soil, though this is soil type dependent. It is considered that this risk is only relevant if the subgrade is a high plasticity clay soil, and if significant negative pore water pressures exist from the construction (e.g. from excavation, removing trees etc.) or previous site history. However, assuming the pitch construction provides a similar overburden pressure at formation level to that caused by unloading through excavations, and the subgrade is suitably protected during construction, negative pore water pressures generated (suctions) are likely to be small and dissipation of these would occur during early in-service life. Furthermore, slight softening of the subgrade is likely to be occurring in most cases on clay soil subgrades through contact with water regardless of the detailed drainage design. Pitches generally experience small live loads in-service and this is not expected to create a structural problem (nor has it been reported to our knowledge). The effective engineering behaviour of the pitch for drainage and structural loads should be included into the pitch design, and clearly requires a suitably thorough site investigation to fully characterise the subgrade soils.

The exfiltration of rainfall water from the sub-base into the subgrade may have been a factor in the field monitoring observations of low yield. However the monitored sites were reportedly constructed on low permeability clays. Notwithstanding the arguments presented in the paragraph above it is clearly sensible to consider this as a possible drainage solution, further reducing discharge water from the sports facility. This is assuming the water table is at a suitably low depth relative to formation and that there are no restrictions imposed (e.g. on protected aquifers or in relation to contaminated land etc.).

An overall finding of this study is that sport pitches can be harnessed as a sustainable drainage tool for integrated storm water management. In addition to their capacity to act as a method of 'source control' for surface rainfall water volumes that land on the pitch there is potential capacity to permit the pitch to further enable the sustainable drainage of other local amenities such as the clubhouse/sports hall or other neighbouring pitches - if designed appropriately.

4.0 CONCLUSIONS

Sport pitch design incorporates porous substructures and pipe drainage networks to remove surface rainfall and ensure the surface remains playable in adverse weather conditions.

The drainage behaviour and performance of sport pitches has not been documented in detail prior to this study.

It appears that in some cases in planning approvals tight discharge consent limits have been required, leading to potential over-design of the pitch drainage system.

Anecdotal evidence from the industry had suggested that pitch designs can provide some internal storage and attenuation capability that limit high discharge rates.

This study evaluated the field performance of several artificial and natural pitch constructions, and additionally evaluated artificial pitch components in the laboratory.

The experimental work has demonstrated that the porous pitch designs do provide high attenuation of peak rainfall events and large capacity for storage, similar to the requirements of SuDs based 'source control' designs required in new urban developments.

The field monitoring observations suggest that in reality the drainage system behaviour is not as consistent or predictable as might be expected from assumptions made in design software (i.e. the water volume in is not equal to the volume out).

The experimental work, combined with the mathematical modelling, has highlighted the key mechanisms that provide resistance to flow and explain the attenuation behaviour observed. It is considered that in most cases insufficient head may be realised in the sub-base to drive water to the drainage pipes, even for some high intensity storm events. In addition, the high frictional resistance to flow of the corrugated collector pipes provide additional 'head' losses providing further resistance to flow. As a consequence, under typical storms the rainfall volume collected in the pitch cannot all be discharged.

These findings support a need for informing and updating current policy and practice regarding sport pitch drainage design to ensure that future designs are value engineered regarding discharge and runoff flood risk.

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Appendix B

A) Illustration of ball stop fencing example - permanent type.

B) Illustration of typical MUGA fencing.

Rev	Drawn	Revision Description	Date
K		Amendments to site plan	08/04/21
J	GO	Parking spaces added. Binstore moved. Amendments to hardstanding.	15/10/20
I	GO	Site plan amendments. Buidling relocated. Parking and access road revised.	13/10/20
H	JF	Building rotated and aligned to south of site. Minor internal amendments.	06/10/20
G	JF	Building reduced in size to reflect phase 1 build	29/09/20
F	GO	Roller shutter added to the terrace	05/02/20
E	GO	Pedestrian path on Milton Road added. Tactile paving and dropped kerbs added. Ball stop net added. Cycle parking revised. Parking layout amended. Landscape buffer amended.	04/02/20
D	GO	Minor amendments to the carpark layout, access, binstore following tracking comments. Low level bollards incorporated.	29/10/19
C	GO	Planning issue	25/09/19
B	GO	True north orientation. Minor amendments	20/09/19
A	GO	First issue for comments	17/09/19

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Lathams
ARCHITECTURE + URBANISM

Project
Adderbury Sports & Community Building

Client
Adderbury Parish Council

Drawing Title
Site Plan As Proposed

Drawing Number	Revision
7354 7354(08)02	J
Scale	**Date**
1 : 500	29/04/2019
Drawn	**Checked**
GO	CT
Lathams Job Number	**Original Size**
7354	A1

Proposed Site Plan
1 : 500

Parking allowance: 55 spaces + 7 disabled
Edge of existing vegetation margin as per topo survey - shrubs/trees indicative only.
Carpark and MUGA lighting in accordance with Ansell Lighting report.
All new planting and hedges consist of native species.

Project No.	FEDS- 218132	By:	AK	Chkd:	DKP
Title	Recreation Ground, Milton Road, Adderbury, Banbury, OX15				
Sheet No.	10	Date:	October 2018		

Notes: Tests carried out on 3rd June 2018

Weather conditions: Damp

Trial Pit: Two

1. INPUTS

Trial Pit Dimensions		
Length	1.500	m
Width	0.700	m
Depth	1.000	m
Inlet Depth	0.335	m
Effective Depth	0.665	m

Soil Infiltration Rate = $\frac{Vp_{75\text{-}25}}{ap_{50} \times tp_{75\text{-}25}}$

$Vp_{75\text{-}25}$	the effective storage volume of water in the trial pit between 75% and 25% effective depth =		0.349 m^3	
ap_{50}	the internal surface area of the trial pit up to 50% effective depth and including the base =		2.513 m^2	
$tp_{75\text{-}25}$	the time for the water level to fall from 75% and 25% effective depth =	14.6 minutes	876.66 seconds	(lowest)

f =	Soil Infiltration Rate for Design	=	**1.6E-04** m/s	(lowest)
		=	**0.5705** m/hr	(lowest)

2. INPUT OF PERMEABILITY TEST DATA

TEST 1

Time	Water level	Water Depth
0	383	617
10	683	317
20	896	104
30	978	22

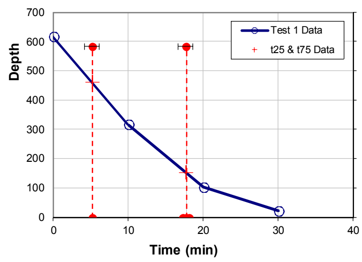
TEST 2

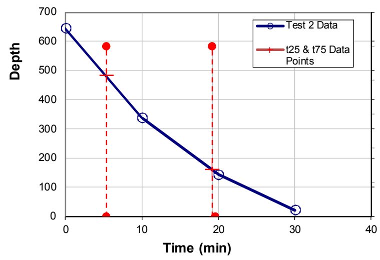
Time	Water level	Water Depth
0	355	645
10	662	338
20	856	144
30	978	22

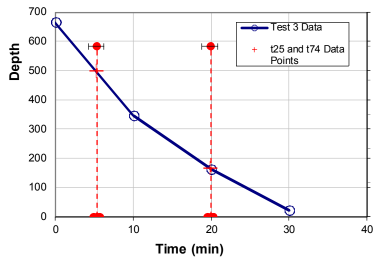
TEST 3

Time	Water level	Water Depth
0	335	665
10	654	346
20	837	163
30	978	22

3. DATA ANALYSIS







Depth at t=	617			
Depth 75%	462.75			
Depth 25%	154.25			
5.14155	462.75			
17.641	154.25			
tp75-25	12.49945	minutes	749.967	seconds
f1 =	1.85E-04 m/s			

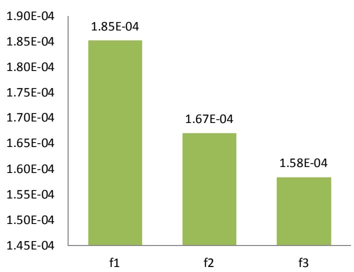
Depth at t=0	645			
Depth 75%	483.75			
Depth 25%	161.25			
5.2525	483.75			
19.111	161.25			
tp75-25	13.8585	minutes	831.51	seconds
f2 =	1.67E-04 m/s			

Depth at t=0	665			
Depth 75%	498.75			
Depth 25%	166.25			
5.2115	498.75			
19.8225	166.25			
tp75-25	14.611	minutes	876.66	seconds
f3=	1.58E-04 m/s			

4. SUMMARY

Infiltration Rate

f1	1.85E-04
f2	1.67E-04
f3	1.58E-04



Appendix C

ICP SUDS Mean Annual Flood

Input

Return Period (years)	2	Soil	0.400
Area (ha)	0.675	Urban	0.000
SAAR (mm)	654	Region Number	Region 6

Results	**l/s**
QBAR Rural	2.1
QBAR Urban	2.1
Q2 years	1.9
Q1 year	1.8
Q30 years	4.8
Q100 years	6.8

STORM SEWER DESIGN by the Modified Rational Method

Design Criteria for Storm

Pipe Sizes STANDARD Manhole Sizes STANDARD

FSR Rainfall Model - England and Wales

Return Period (years)	2	PIMP (%)	100
M5-60 (mm)	20.000	Add Flow / Climate Change (%)	0
Ratio R	0.409	Minimum Backdrop Height (m)	0.200
Maximum Rainfall (mm/hr)	50	Maximum Backdrop Height (m)	1.500
Maximum Time of Concentration (mins)	30	Min Design Depth for Optimisation (m)	0.500
Foul Sewage (l/s/ha)	0.000	Min Vel for Auto Design only (m/s)	1.00
Volumetric Runoff Coeff.	0.750	Min Slope for Optimisation (1:X)	500

Designed with Level Inverts

Time Area Diagram for Storm

Time (mins)	Area (ha)	Time (mins)	Area (ha)
0-4	0.042	4-8	0.025

Total Area Contributing (ha) = 0.067

Total Pipe Volume (m³) = 2.121

Network Design Table for Storm

PN	Length (m)	Fall (m)	Slope (1:X)	I.Area (ha)	T.E. (mins)	Base Flow (l/s)	k (mm)	HYD SECT	DIA (mm)	Section Type	Auto Design
1.000	10.000	0.171	58.5	0.007	6.00	0.0	0.600	o	300	Pipe/Conduit	
1.001	10.000	0.171	58.5	0.000	0.00	0.0	0.600	o	300	Pipe/Conduit	
1.002	10.000	0.171	58.5	0.000	0.00	0.0	0.600	o	300	Pipe/Conduit	

Network Results Table

PN	Rain (mm/hr)	T.C. (mins)	US/IL (m)	Σ I.Area (ha)	Σ Base Flow (l/s)	Foul (l/s)	Add Flow (l/s)	Vel (m/s)	Cap (l/s)	Flow (l/s)
1.000	50.00	6.08	97.000	0.007	0.0	0.0	0.0	2.06	145.6	0.9
1.001	50.00	6.16	96.829	0.007	0.0	0.0	0.0	2.06	145.6	0.9
1.002	50.00	6.24	96.658	0.007	0.0	0.0	0.0	2.06	145.6	0.9

Free Flowing Outfall Details for Storm

Outfall Pipe Number	Outfall Name	C. Level (m)	I. Level (m)	Min I. Level (m)	D,L (mm)	W (mm)
1.002		97.540	96.487	0.000	0	0

Simulation Criteria for Storm

Volumetric Runoff Coeff	0.750	Additional Flow - % of Total Flow	0.000
Areal Reduction Factor	1.000	MADD Factor * 10m³/ha Storage	2.000
Hot Start (mins)	0	Inlet Coeffiecient	0.800
Hot Start Level (mm)	0	Flow per Person per Day (l/per/day)	0.000
Manhole Headloss Coeff (Global)	0.500	Run Time (mins)	60
Foul Sewage per hectare (l/s)	0.000	Output Interval (mins)	1

Number of Input Hydrographs	0	Number of Storage Structures	2
Number of Online Controls	1	Number of Time/Area Diagrams	0
Number of Offline Controls	0	Number of Real Time Controls	0

Synthetic Rainfall Details

Rainfall Model	FSR	Profile Type	Summer
Return Period (years)	2	Cv (Summer)	0.750
Region	England and Wales	Cv (Winter)	0.840
M5-60 (mm)	20.000	Storm Duration (mins)	30
Ratio R	0.409		

Online Controls for Storm

Hydro-Brake® Optimum Manhole: Point 3, DS/PN: 1.002, Volume (m³): 0.8

Unit Reference	MD-SHE-0052-1000-0600-1000
Design Head (m)	0.600
Design Flow (l/s)	1.0
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	52
Invert Level (m)	96.658
Minimum Outlet Pipe Diameter (mm)	75
Suggested Manhole Diameter (mm)	1200

Control Points	**Head (m)**	**Flow (l/s)**
Design Point (Calculated)	0.600	1.0
Flush-Flo™	0.186	1.0
Kick-Flo®	0.389	0.8
Mean Flow over Head Range	-	0.9

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	**Flow (l/s)**	**Depth (m)**	**Flow (l/s)**	**Depth (m)**	**Flow (l/s)**	**Depth (m)**	**Flow (l/s)**
0.100	0.9	1.200	1.4	3.000	2.1	7.000	3.1
0.200	1.0	1.400	1.5	3.500	2.2	7.500	3.2
0.300	1.0	1.600	1.5	4.000	2.4	8.000	3.3
0.400	0.8	1.800	1.6	4.500	2.5	8.500	3.4
0.500	0.9	2.000	1.7	5.000	2.6	9.000	3.5
0.600	1.0	2.200	1.8	5.500	2.7	9.500	3.6
0.800	1.1	2.400	1.9	6.000	2.8		
1.000	1.3	2.600	1.9	6.500	3.0		

Storage Structures for Storm

Trench Soakaway Manhole: Infiltration Drains, DS/PN: 1.000

Infiltration Coefficient Base (m/hr)	0.57050	Trench Width (m)	0.1
Infiltration Coefficient Side (m/hr)	0.57050	Trench Length (m)	1188.0
Safety Factor	2.0	Slope (1:X)	0.0
Porosity	0.30	Cap Volume Depth (m)	0.300
Invert Level (m)	97.000	Cap Infiltration Depth (m)	0.000

Infiltration Basin Manhole: Point 2, DS/PN: 1.001

Invert Level (m)	96.829	Safety Factor	2.0
Infiltration Coefficient Base (m/hr)	0.57050	Porosity	1.00
Infiltration Coefficient Side (m/hr)	0.57050		

Depth (m)	**Area (m²)**	**Depth (m)**	**Area (m²)**
0.000	12.3	1.000	12.3

1 year Return Period Summary of Critical Results by Maximum Level (Rank 1) for Storm

Simulation Criteria

Areal Reduction Factor	1.000	Additional Flow - % of Total Flow	0.000
Hot Start (mins)	0	MADD Factor * 10m³/ha Storage	2.000
Hot Start Level (mm)	0	Inlet Coeffiecient	0.800
Manhole Headloss Coeff (Global)	0.500	Flow per Person per Day (l/per/day)	0.000
Foul Sewage per hectare (l/s)	0.000		

Number of Input Hydrographs	0	Number of Storage Structures	2
Number of Online Controls	1	Number of Time/Area Diagrams	0
Number of Offline Controls	0	Number of Real Time Controls	0

Synthetic Rainfall Details

Rainfall Model	FSR	Ratio R	0.409
Region	England and Wales	Cv (Summer)	0.750
M5-60 (mm)	20.000	Cv (Winter)	0.840

Margin for Flood Risk Warning (mm)	10.0	DVD Status	OFF
Analysis Timestep	Fine	Inertia Status	OFF
DTS Status	ON		

Profile(s)	Summer and Winter
Duration(s) (mins)	15, 30, 60, 120, 240, 360, 480, 960, 1440
Return Period(s) (years)	1, 30, 100
Climate Change (%)	0, 0, 30

PN	US/MH Name	Storm	Return Period	Climate Change	First (X) Surcharge	First (Y) Flood	First (Z) Overflow	Overflow Act.
1.000	Infiltration Drains	15 Winter	1	+0%				
1.001	Point 2	15 Winter	1	+0%				
1.002	Point 3	15 Winter	1	+0%				

PN	US/MH Name	Water Level (m)	Surcharged Depth (m)	Flooded Volume (m³)	Flow / Cap.	Overflow (l/s)	Pipe Flow (l/s)	Status	Level Exceeded
1.000	Infiltration Drains	97.002	-0.298	0.000	0.00		0.1	OK	
1.001	Point 2	96.830	-0.299	0.000	0.00		0.1	OK	
1.002	Point 3	96.673	-0.285	0.000	0.00		0.1	OK	

30 year Return Period Summary of Critical Results by Maximum Level (Rank 1) for Storm

Simulation Criteria

Areal Reduction Factor 1.000 Additional Flow - % of Total Flow 0.000
Hot Start (mins) 0 MADD Factor * 10m³/ha Storage 2.000
Hot Start Level (mm) 0 Inlet Coeffiecient 0.800
Manhole Headloss Coeff (Global) 0.500 Flow per Person per Day (l/per/day) 0.000
Foul Sewage per hectare (l/s) 0.000

Number of Input Hydrographs 0 Number of Storage Structures 2
Number of Online Controls 1 Number of Time/Area Diagrams 0
Number of Offline Controls 0 Number of Real Time Controls 0

Synthetic Rainfall Details

Rainfall Model FSR Ratio R 0.409
Region England and Wales Cv (Summer) 0.750
M5-60 (mm) 20.000 Cv (Winter) 0.840

Margin for Flood Risk Warning (mm) 10.0 DVD Status OFF
Analysis Timestep Fine Inertia Status OFF
DTS Status ON

Profile(s) Summer and Winter
Duration(s) (mins) 15, 30, 60, 120, 240, 360, 480, 960, 1440
Return Period(s) (years) 1, 30, 100
Climate Change (%) 0, 0, 30

PN	US/MH Name	Storm	Return Period	Climate Change	First (X) Surcharge	First (Y) Flood	First (Z) Overflow	Overflow Act.
1.000	Infiltration Drains	15 Winter	30	+0%				
1.001	Point 2	15 Winter	30	+0%				
1.002	Point 3	15 Winter	30	+0%				

PN	US/MH Name	Water Level (m)	Surcharged Depth (m)	Flooded Volume (m³)	Flow / Cap.	Overflow (l/s)	Pipe Flow (l/s)	Status	Level Exceeded
1.000	Infiltration Drains	97.005	-0.295	0.000	0.00		0.3	OK	
1.001	Point 2	96.832	-0.297	0.000	0.00		0.2	OK	
1.002	Point 3	96.683	-0.275	0.000	0.00		0.2	OK	

100 year Return Period Summary of Critical Results by Maximum Level (Rank 1) for Storm

Simulation Criteria

Areal Reduction Factor 1.000 Additional Flow - % of Total Flow 0.000
Hot Start (mins) 0 MADD Factor * 10m³/ha Storage 2.000
Hot Start Level (mm) 0 Inlet Coeffiecient 0.800
Manhole Headloss Coeff (Global) 0.500 Flow per Person per Day (l/per/day) 0.000
Foul Sewage per hectare (l/s) 0.000

Number of Input Hydrographs 0 Number of Storage Structures 2
Number of Online Controls 1 Number of Time/Area Diagrams 0
Number of Offline Controls 0 Number of Real Time Controls 0

Synthetic Rainfall Details

Rainfall Model FSR Ratio R 0.409
Region England and Wales Cv (Summer) 0.750
M5-60 (mm) 20.000 Cv (Winter) 0.840

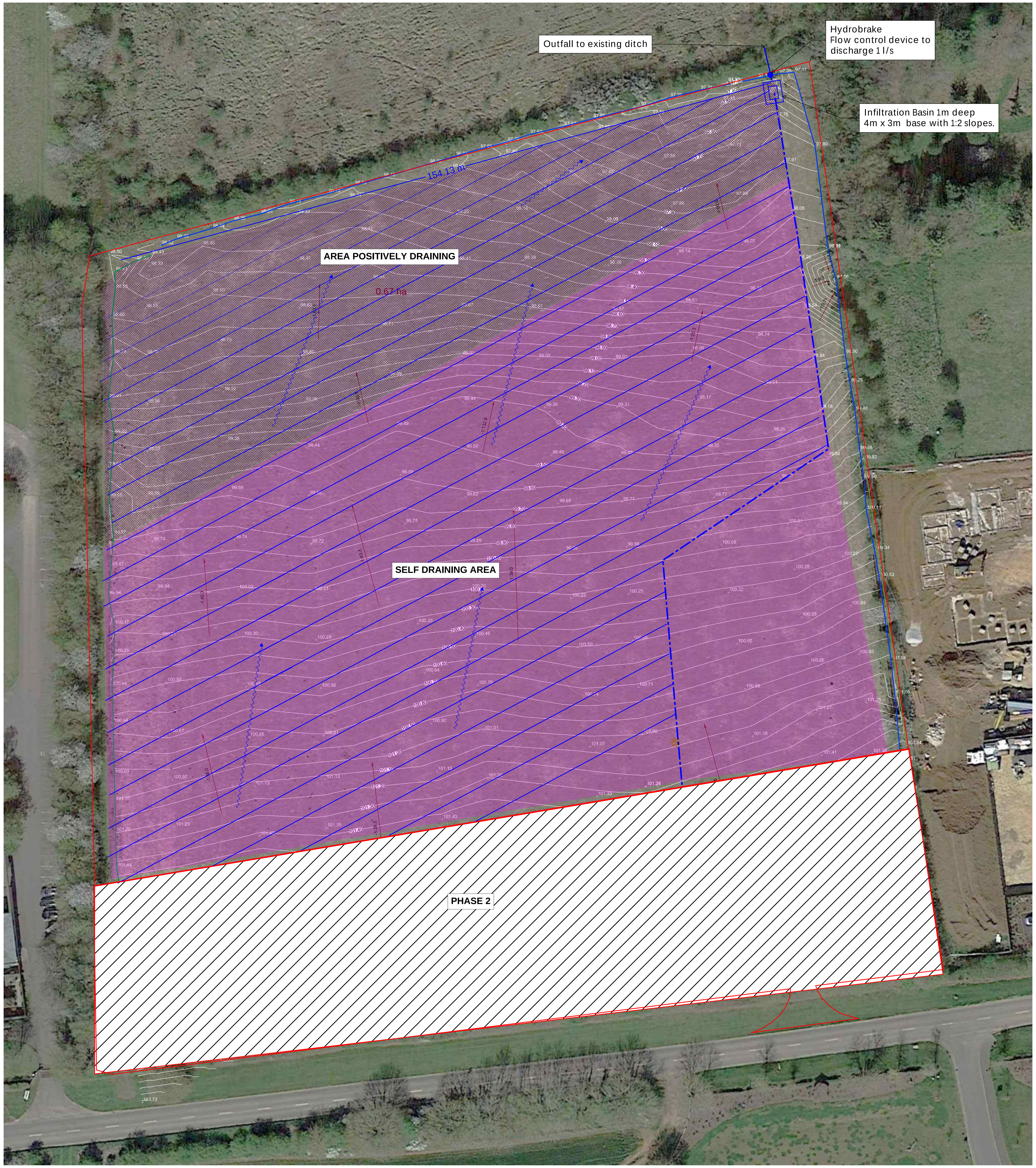
Margin for Flood Risk Warning (mm) 10.0 DVD Status OFF
Analysis Timestep Fine Inertia Status OFF
DTS Status ON

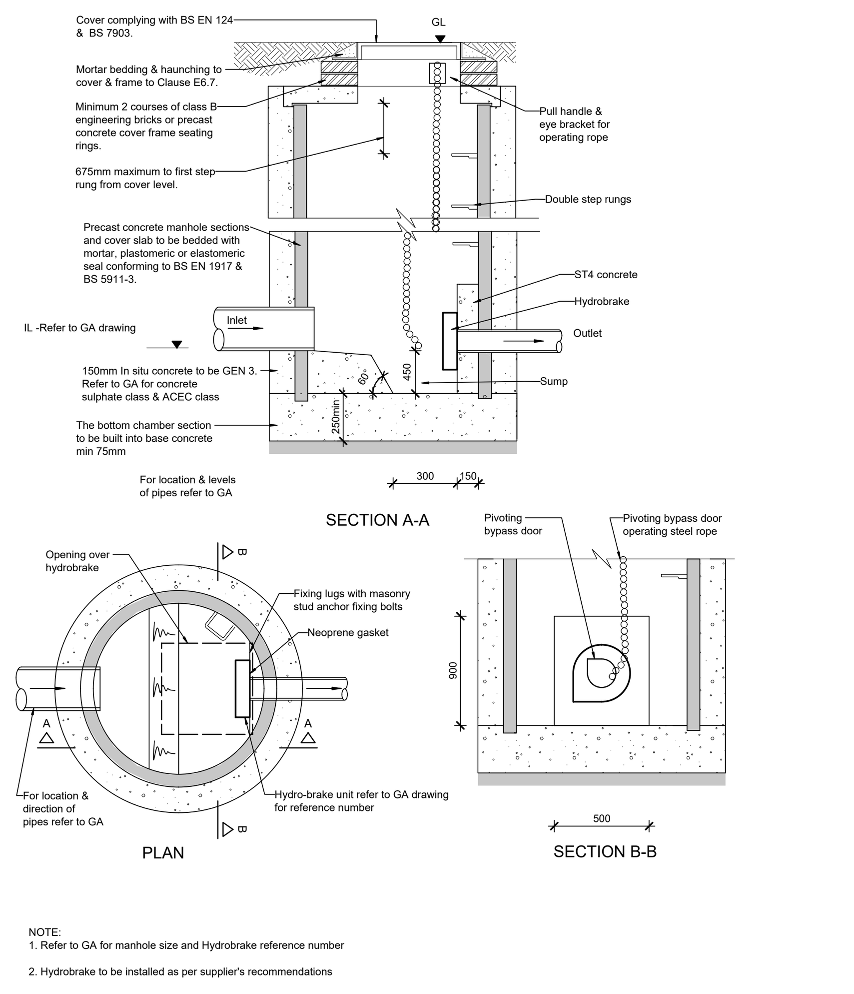
Profile(s) Summer and Winter
Duration(s) (mins) 15, 30, 60, 120, 240, 360, 480, 960, 1440
Return Period(s) (years) 1, 30, 100
Climate Change (%) 0, 0, 30

PN	US/MH Name	Storm	Return Period	Climate Change	First (X) Surcharge	First (Y) Flood	First (Z) Overflow	Overflow Act.
1.000	Infiltration Drains	15 Winter	100	+30%				
1.001	Point 2	15 Winter	100	+30%				
1.002	Point 3	15 Winter	100	+30%				

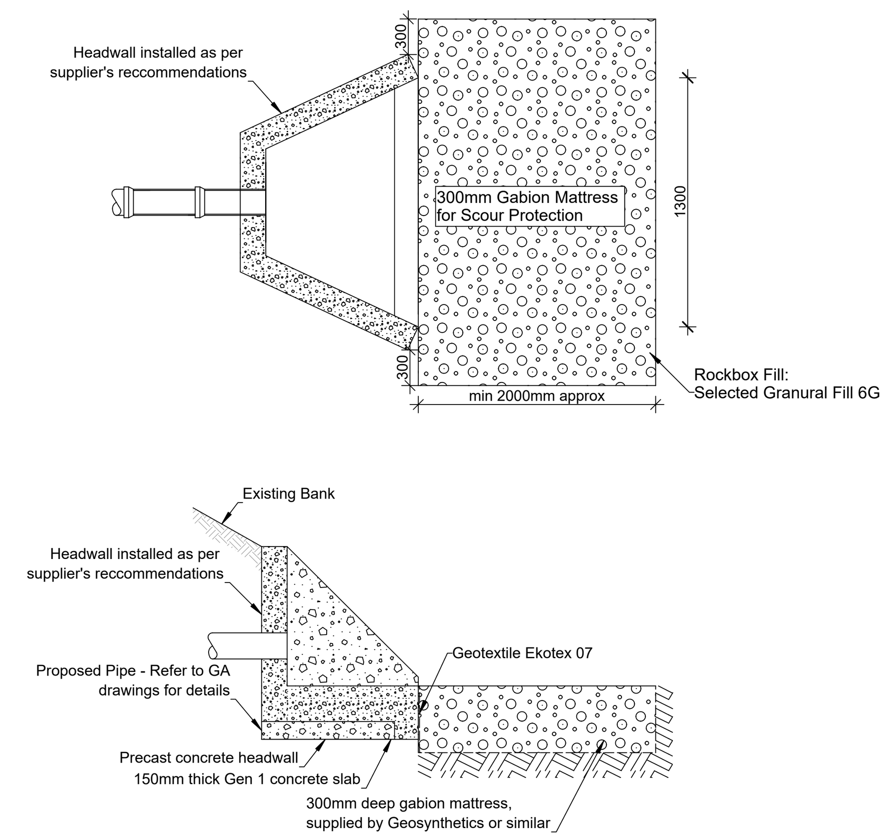
PN	US/MH Name	Water Level (m)	Surcharged Depth (m)	Flooded Volume (m³)	Flow / Cap.	Overflow (l/s)	Pipe Flow (l/s)	Status	Level Exceeded
1.000	Infiltration Drains	97.008	-0.292	0.000	0.01		0.6	OK	
1.001	Point 2	96.834	-0.295	0.000	0.00		0.4	OK	
1.002	Point 3	96.693	-0.265	0.000	0.00		0.4	OK	

Appendix D



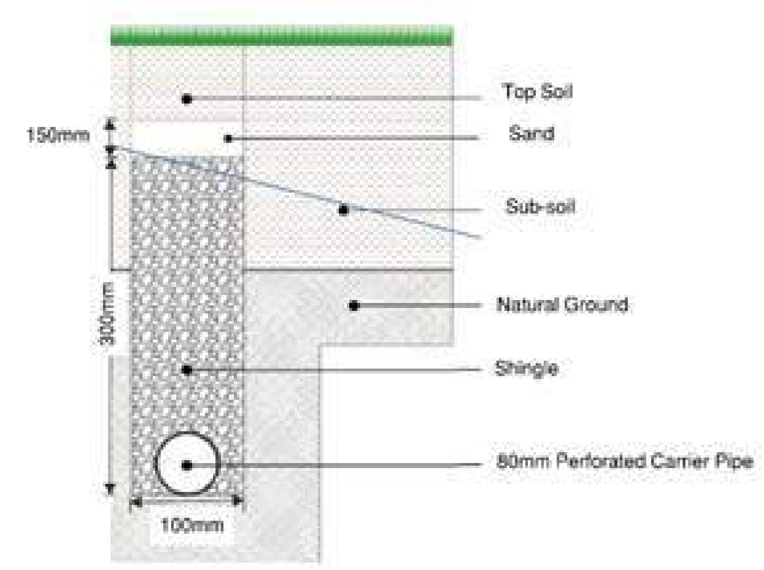


Flow Control Device Chamber- Hydrobrake



Headwall	Accommodate Pipe Sizes	Back wall height (external)	Front wall height (external)	Width at back wall (internal)	Width at front wall (internal)	Headwall Length	Wall Thickness	Floor thickness	Approx. Weight
	mm	mm	mm	mm	mm	mm	mm	mm	kg
FPMcCann Precast Headwall HW Small 100	300,225,160	810	300	520	1300	1320	100	162	1090

Precast Concrete Headwall



Perforated Pipe Details

KEY

- Perforated Pipe
- Carrier Pipe
- Exceedance Flow
- Self Draining Area (due to infiltration rate)
- Positively drained area

SCALE 1:500

Drawing Scale Bar

Drawing scale	Line length	Drawing scale	Line length
1:5	= 0.25 metres	1:200	= 10.0 metres
1:10	= 0.5 metres	1:250	= 12.5 metres
1:20	= 1.0 metres	1:500	= 25.0 metres
1:25	= 1.25 metres	1:1000	= 50.0 metres
1:50	= 2.5 metres	1:1250	= 62.5 metres
1:100	= 5.0 metres	1:2500	= 125 metres

Measure length of line above for checking of scale

GENERAL NOTES

This drawing should be read in conjunction with relevant Structural and Architectural Information:

All work to be undertaken in accordance with relevant health and safety legislation, adopting safe methods of working and in accordance with manufacturers' product literature and H&S data sheets.

Contractor to check all levels, dimensions and heights on site before commencing.

Refer to structural engineers details for all structural, underground drainage requirements.

Rev	Details	Date	By	Ch'd

Drawing Status:

PRELIMINARY

RIDA
FLOOD RISK ASSESSMENT & DRAINAGE STRATEGIES

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Client:

Project:
Land North Of Milton Road, Adderbury

Drawing:
Proposed Drainage Strategy

Print Size:	Project No:	Drawing No:	Revision:
A1	0202	003	P1