2.2.1 Flood Risk Assessment

Calder Park

Tungsten Development. Surface Water Assessment.

Final Report

November 2020

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Revision History

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13 November 2020	Draft Report	Mark Barwood
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Contract

This report describes work commissioned by Mark Barwood, on behalf of Alan Wood and Partners, by email dated the 4 November 2020. Alan Wood and Partners' representative for the contract was Mark Barwood. Mark Bentley of JBA Consulting carried out this work.

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Purpose

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Acknowledgements

The help with arranging the provision of data by Mark Barwood is gratefully acknowledged.

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Abbreviations

AEP	Annual Exceedance Probability
DEP	Donaldson Edwards Partnership
FEH	Flood Estimation Handbook
FRA	Flood Risk Assessment
HEC-RAS	Hydrologic Engineering Center – River Analysis System
JBA	Jeremy Benn Associates
mAOD	Metres Above Ordnance Datum
SSU	Scientific Support Unit
SuDS	Sustainable Drainage System
WYP	West Yorkshire Police

1 Introduction

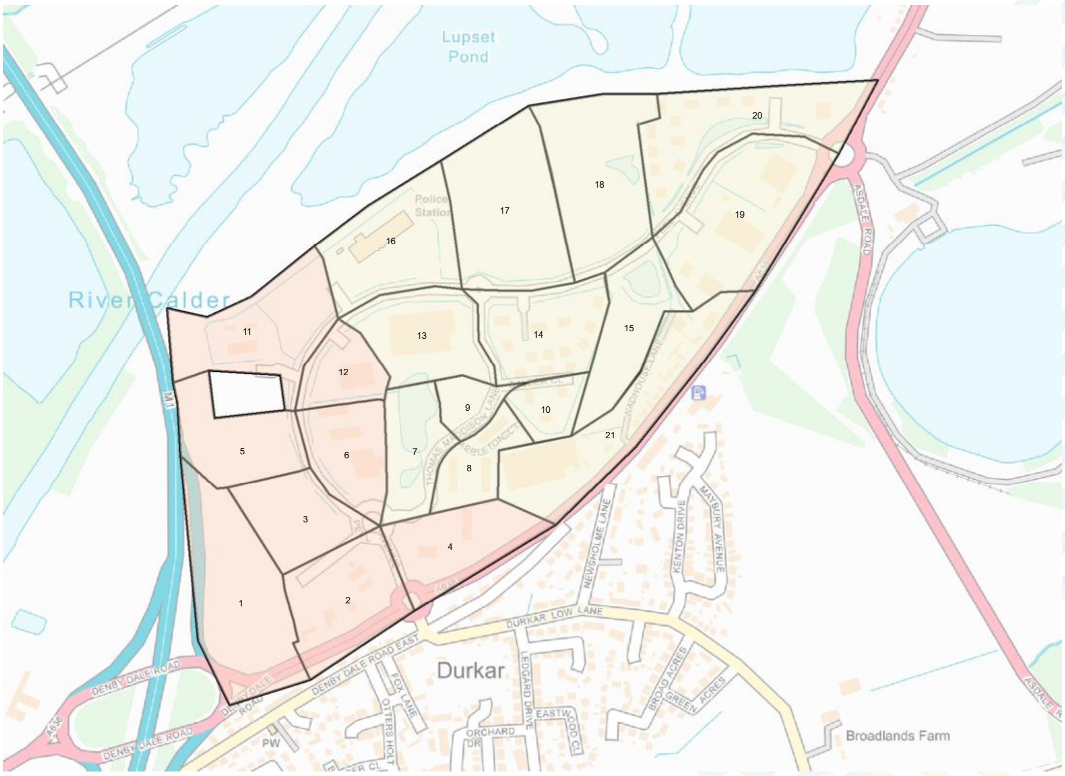
1.1 Flood risk assessment

The Calder Park development site is adjacent to Junction 39 of the M1. The site is bounded to the north and east by a flood embankment of the River Calder, to the west by the M1 motorway and to the south by Denby Dale Road. Between 2002 and 2018, JBA undertook work assessing the flood risk to the site and assisting with the design of the surface water drainage system. The surface water drainage system was designed to use a network of dry open channels (swales) and storage ponds to attenuate the runoff from the development plots.

There are proposals to develop a remaining plot of land at Calder Park. The proposed plot of land lies on the northern edge of Calder Park, just to the east of the Scientific Support Unit (SSU) building of West Yorkshire Police (WYP). The proposed platform level for the site will be set at 26.5mAOD. Appendix A contains drawings showing the proposed development. The total area of the plot of land for the new development is 60,700m^2 of which 51,950m^2 will comprise impermeable area. Therefore, 85% of the proposed will be impermeable which is greater than the value of 70% that was assumed for the design of the drainage system. This study has assessed the effect of increased runoff due to the greater impermeable area at the proposed site on water levels in the surface water drainage system.

JBA have re-used data previously collected on behalf of Peel Holdings, the developer of Calder Park. This includes topographical survey data showing ground levels and a computer model built for the business park surface water drainage system.

Figure 1-1 shows the Calder Park site. The proposed development will be in zones 17 and 18.



Not to scale

Figure 1-1: Calder Park location plan

1.2 Report structure

The report is presented in three sections:

1. **Introduction** – this section sets the study in context
2. **Surface Water Runoff** – flood risk from runoff within the Calder Park development
3. **Conclusions**

2 Surface water drainage

2.1 Surface water drainage system

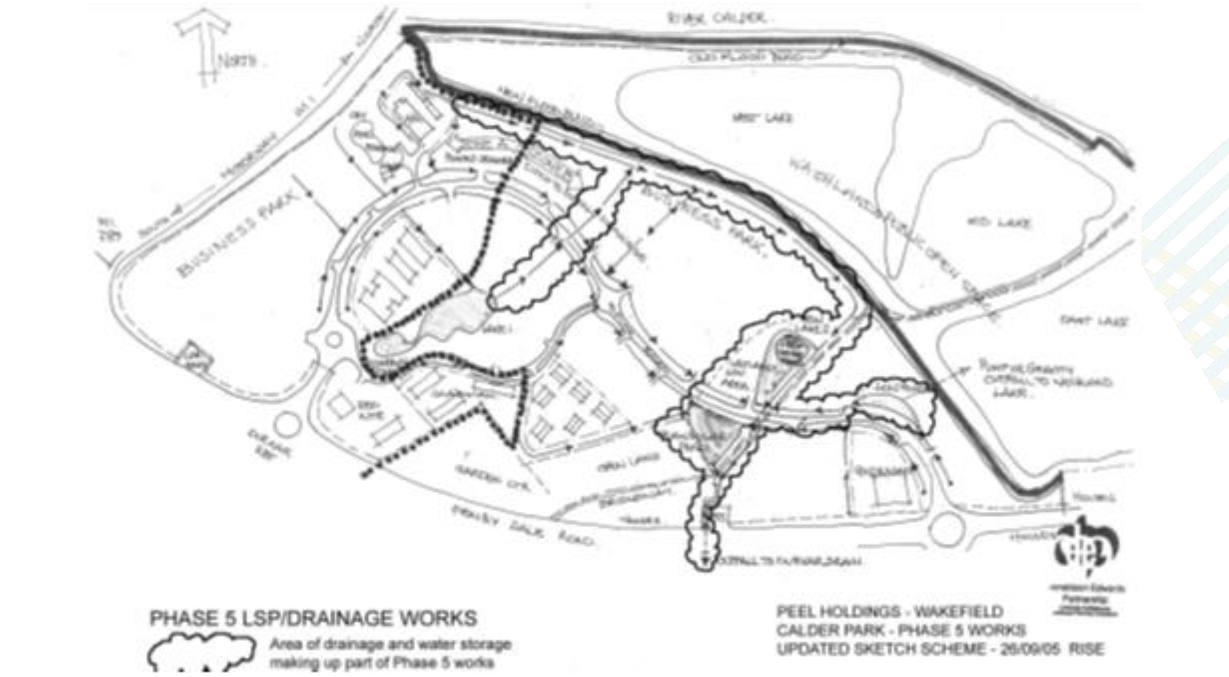
The drainage scheme for the Calder Park site utilises Sustainable Drainage System (SuDS) principles and is an integral part of the site design. The buildings, car parks and access roads are situated on raised platforms. A series of wide ditches (see Figure 2-1) run alongside the roads and between the building plots. There are also a few lakes that form part of the drainage scheme as well as improving the aesthetics of the site.

Figure 2-1: Typical swale at Calder Park

The site has been split into two surface water management zones, A and B (see Figure-2-2). Zone A utilises more traditional storm water drainage where runoff from roofs and roads is piped to a holding pond. This pond is then pumped directly to the River Calder at a rate of up to 1m^3/s. The system was designed such that there was sufficient storage in the pond to cope with runoff from a 100-year storm (including a 20% increase in rainfall intensity to allow for climate change).

Since the pond was designed the climate change allowance has been updated so that a 40% increase in rainfall intensity should be applied. Runoff calculations to the storage pond during a 1 in 100-year storm with an allowance of 40% increase in rainfall intensity show the pond level would rise to 26.0mAOD. That is 1m below the bank top (27.0mAOD). Therefore, the pond is large enough to deal with the revised allowances for climate change.

Runoff from all permeable areas and from subsequent plot developments (Zone B) goes directly to the ditches and is held in the land drainage system.



Note: Zone A is west of the dotted line and Zone B is to the east.

Figure 2-2: Calder Park drainage zones

The only outflow from the Calder Park site is to Durkar Drain. Water flows in a culvert (600mm diameter) under the Denby Dale Road. The ditch system at Calder Park connects to this culvert. The effective invert level of the culvert under the Denby Dale Road has been surveyed at 23.49mAOD. The lowest levels at Calder Park (in the north-east corner of the site) are lower than this. Therefore, it is not possible to drain the whole of Calder Park to Durkar Drain. A pumping station operates during high water levels to transfer water to the adjacent washland at a rate of 50l/s.

Surface water drainage for Calder Park has been calculated in three parts:

- Calculation of the volumes and rates of runoff during design events
- Hydraulic modelling of the ditch system to derive design water levels
- Calculations for the capacity of the culvert under Denby Dale Road.

2.2 Rainfall runoff calculations

Design rainfall depths for the site were calculated from the Flood Estimation Handbook (FEH). Two durations of storm were considered: 60-minutes and 7-hours for a 1%AEP (100-year return period) flood event.

Further consideration was given to the effects of climate change. The latest government guidance states that a 40% increase in rainfall should be used to account for climate change up to the year 2115 in the case of essential infrastructure .

A further storm profile comprising two consecutive 7-hour storms for a 3.3%AEP (30-year return period) was considered. This combination was used to assess the capacity of the surface water drainage system to recover from one storm before a further event occurs.

Table 2-1 summarises the design storms.

Table 2-1: Volume of storm rainfall

Storm Duration	3.3%AEP (30-year return period)			1%AEP (100-year return period)		
	Rainfall depth (mm)	Volume over site (m^3)	Peak storm intensity (mm/hr)	Rainfall depth (mm)	Volume over site (m^3)	Peak storm intensity (mm/hr)
60-minute				40	22,287	168
7-hour	90	50,298	18	62	34,234	24
with 40% allowance for climate change						
60-minute	Climate change scenario only applies to the 1% AEP (100-year flood event)			56	31,202	235
7-hour				87	47,928	34

For the purposes of calculating surface water runoff at Calder Park, the site was divided into 21 sub-catchments (see Figure 1-1). The sub-catchments represent different development plots across the Calder Park site. The calculations for each sub-catchment and the split between permeable and impermeable areas were adjusted based on plans supplied by WSP. For this study the percentage impermeable area of zones 17 and 18 was increased from 70% to 85%.

The division of the site enables the calculation of runoff volumes to be split between sites draining to the storage pond for pumping to the Calder and sites draining to the open ditches and eventual discharge to Durkar Drain. Storm water runoff volumes were calculated assuming that impermeable areas had a 100% runoff rate and 30% for permeable areas. Table 2-2 summarises volume of runoff for each design storm.

Table 2-2: Volume (m^3) of surface water runoff

Design scenario	Storm duration	Runoff destination	
		Pumped discharge	Durkar Drain
3.3%AEP	Two consecutive 7-hour storms	13,537	24,688
1%AEP (100-year return period)	1-hour	5,975	10,887
	7-hour	9,177	16,722
1% AEP with climate change	1-hour	8,365	15,242
	7-hour	12,848	23,411

Further runoff calculations were undertaken to assess the effect of saturated ground conditions which assumed the runoff from permeable areas was also 100%. Table 2.3 summarises the volume of runoff for this situation.

Table 2-3: Volume (m^3) of surface water runoff

Design scenario	Storm duration	Runoff destination	
		Pumped discharge	Durkar Drain
3.3%AEP	Two consecutive 7-hour storms	13,537	36,962
1%AEP (100-year return period)	1-hour	5,975	16,313
	7-hour	9,177	25,057
1% AEP with climate change	1-hour	8,365	22,838
	7-hour	12,848	35,080

There is a further discharge to the drainage system from the SSU building of WYP. The building is heated/cooled using an open loop groundwater heat extraction system. Depending on the energy demand this may result in peak flows of 18l/s to the ditch system. Over a 60-minute storm this would result in, at most, an extra $65m^3$ of flow. Over a seven-hour storm the maximum runoff would be $454m^3$ assuming the system operates at peak capacity throughout the storm.

Although the discharge from the SSU is small it is continuous and so there is the potential for the flow to reduce the capacity of the drainage ditches prior to a storm commencing. Therefore, the simulations of the drainage system have assumed the ditches are partially full due to continuous flow from the groundwater heat extraction system.

2.3 Hydraulic modelling

2.3.1 HEC-RAS model

Modelling of the drainage ditches was undertaken using HEC-RAS. HEC-RAS is an unsteady state one-dimensional river modelling package developed by the Hydrologic Engineering Center of the US Army Corps of Engineers. The software allows the user to calculate the variation of water surface within a channel network during a storm. The system can handle a looped network of channels, a branched system or just a single river reach. The model also allows the simulation of the effect of flood storage on routing flood flows. The model runs quickly and so was able to represent the draining down of the drainage system following an event.

The HEC-RAS model was originally set up with ditch profiles based on information supplied by Peel Holdings. The ditches are generally constructed by making a 1-in-3 slope down from the plot level to the proposed ditch invert level, cutting a flat bed 1.5m wide at this level and then making a 1 in 3 slope up to meet the opposite bank height. The actual ditch widths therefore depend on the adjacent ground heights and the ditch invert level.

Figure 2-3 shows the model for the completed ditch system.

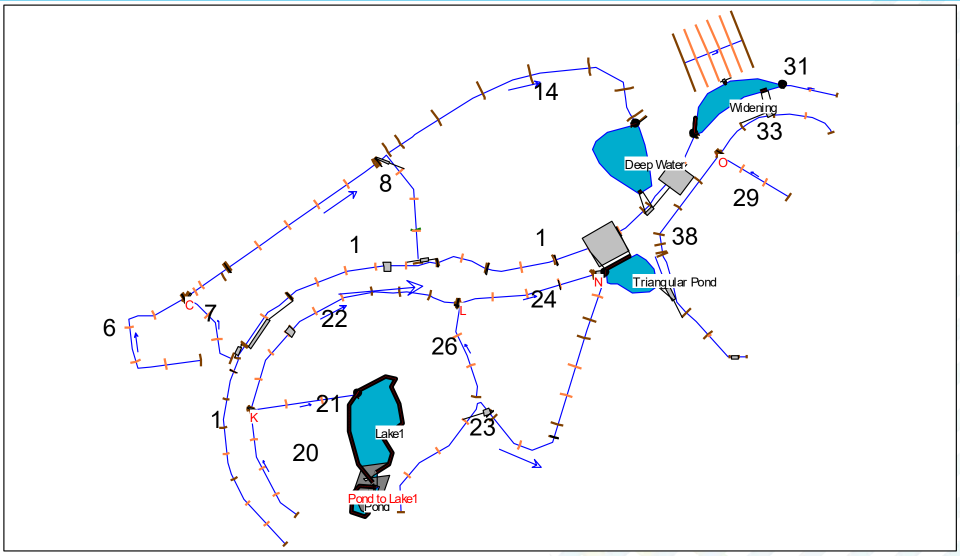


Figure 2-3: HEC-RAS schematic of completed drainage system

A combination of lateral weirs (sometimes incorporating culverts) and open channel junctions were used for connections between ditches and other ditches or lakes. To control water levels, retain the design standing water level and aid model stability (to prevent reaches becoming dry), a number of weirs were defined at intervals along the ditches. The weirs in the model were only defined where required to retain standing water levels.

Manning's *n* values (representing the roughness of the drains) were set to 0.03 at all locations. Since the drains are essentially a storage system (with flow controlled by weirs) the model results were expected to be insensitive to Manning's n values. However, sensitivity tests for this were undertaken (see Section 2.4.2). The model was run with a timestep of 6 seconds.

2.3.2 Standing water levels

At the start of a model simulation it is necessary to define the initial water levels in the ditches. In the case of the drainage system at Calder Park this is especially important because it will define the excess flood storage capacity in the system. The level of water in the ditches will depend on four factors:

- Drainage system design
- Groundwater levels
- Discharges from the SSU groundwater heat extraction system
- Recent storms, which have not fully drained away

To allow for high groundwater levels (see Chapter 4), the ditches have been designed to have a standing water depth of 0.5m with 1.5m depth in the lakes. Only ditches to the east of the Calder Park site would have deeper water (0.65m).

Initial conditions for the model simulations have been set assuming the groundwater heat extraction system is continuously discharging. This means that storage in the ditches at the start of the simulation has been partially used up before the storm begins.

Model simulations were also undertaken to assess the effect of two consecutive storms occurring. These tested the effect of high standing water levels that had not been able to drain out of the system before the onset of another storm. In this situation the standing water levels at the start of the storm are defined by the model following the simulation of runoff from the first storm.

2.4 Model results

The completed HEC-RAS model was run for the following three storms (all using realistic Runoff calculations – see Table 2-1), allowing outflow to Durkar Drain:

- 100-year, 60-minute Summer Storm
- 100-year, 7-hour Winter Storm
- Two consecutive 30-year, 7-hour Winter Storms

The tail-water level of the Denby Dale Road culvert was set at 25.2mAOD for the period of all the simulations. The flood level of 25.2mAOD was based on advice from Wakefield Council and represents the observed flood level on Denby Dale Road during the flood of the 25 June 2007. This is equivalent to a surcharged condition for the culvert and represents the case of impeded discharge from Durkar Drain to the River Calder (see section 2.5). Under these conditions there is backflow from Durkar Drain into the Calder Park site.

2.4.1 Water levels

The model simulations showed that generally across the site the two consecutive storms had the most impact on water levels in the drainage system. This is because high water levels at Denby Dale Road (25.2mAOD) prevent runoff escaping to Durkar Drain.

Therefore, water levels in the drainage system are sensitive to the runoff volume as opposed to the rainfall intensity.

This conclusion is different to that found during the original FRA in 2008. That study assumed a lower flood level at Denby Dale Road of 25.0mAOD and allowed water to leave Calder Park between consecutive storms.

Table 2-4 shows the highest water level in each ditch from each of the three simulations based on the final ditch layout with development completed. The ditch reach labels can be found on Figure 2-3. Simulated flood levels remained in bank throughout the ditch system.

Table 2-4: Maximum water levels in ditches - Completed drainage system

Upper site		Lower site	
Ditch	Maximum water level (mAOD)	Ditch	Maximum water level (mAOD)
1	26.28	23	25.76
6	25.28	24	25.62
7	25.29	26	25.62
8	25.33	29	25.22
14	25.31	31	25.22
20	26.18	33	25.22
21	26.18	38	25.22
22	26.18		

The effect of increased runoff from the site of the proposed development due to a greater impermeable area has a marginal impact on water levels. The only ditches where an impact was seen were ditches 29 and 38 where the water level increased by just 0.01m.

The proposed plot for the Tungsten Development will be set to 26.5mAOD. The maximum water level in the pond Deep Water and the ditches surrounding the development does not rise above 25.33mAOD. Therefore, there is over 1m freeboard.

2.4.2 Increased runoff (saturated ground conditions)

A sensitivity test with increased runoff to account for saturated ground conditions was undertaken. Runoff from the permeable areas of the site was assumed to be 100% (see section 2.2) and the SSU open loop groundwater heat extraction system was flowing at its maximum rate of 18l/s. This increased the total volume of runoff from the site and raised water levels in the ditches.

Table 2-5 shows the maximum water levels in the ditches for the completed system.

Table 2-5: Maximum water levels in ditches – 100% runoff

Upper site		Lower site	
Ditch	Maximum water level (mAOD)	Ditch	Maximum water level (mAOD)
1	26.60	23	25.83
6	25.47	24	25.83
7	25.58	26	25.83
8	25.47	29	25.24
14	25.36	31	25.24
20	26.23	33	25.24
21	26.23	38	25.24
22	26.23		

Generally, increasing the runoff has raised water levels across the site by the order of a few centimetres. The only significant impact on water level were in Ditches 1, 6, 7, 8, 24 and 26 where levels were raised by 0.32m (ditch 1), 0.29m (ditch 7), 0.14m (ditch 8), 0.21m (ditches 24 and 26). Even so, with saturated runoff, peak water levels were contained within bank.

Even with saturated runoff, the freeboard for the Tungsten Development in the pond Deep Water and the ditches surrounding the development is maintained at over 1m.

2.4.3 Climate change

A sensitivity test for the effects of climate change was also undertaken by applying a 40% increase in the volume of runoff. Table 2-6 shows the peak water levels in each ditch for the climate change scenario.

Table 2-6: Maximum water levels in ditches – Climate change

Upper site		Lower site	
Ditch	Maximum water level (mAOD)	Ditch	Maximum water level (mAOD)
1	26.36	23	25.81
6	25.33	24	25.78
7	25.35	26	25.78
8	25.41	29	25.24
14	25.39	31	25.24
20	26.23	33	25.24
21	26.23	38	25.24
22	26.23		

The impact of climate change was generally less than that due to saturated ground conditions and water levels were retained within the ditch system. For the climate change scenario, the freeboard for the Tungsten Development in the pond Deep Water and the ditches surrounding the development is maintained at over 1m.

2.4.4 Channel Roughness

The sensitivity of the model results to increased channel roughness was assessed by doubling the Manning's *n* coefficient from 0.03 to 0.06 across the site. There was very little sensitivity to Manning's *n* in the lower part of the site which is storage-dominated (only ditch 23 in this area showing an increase in water levels of 0.07m). A more widespread sensitivity was seen in the upper site where flux is more important, but most increases were less than 0.03m.

2.5 Durkar Drain

The outflow from Calder Park to Durkar Drain passes in culvert under the Denby Dale Road, then through a short section of open channel near a foul water pumping station and then into another culvert.

As part of the Wakefield Flood Defence Strategy developed by the EA, the outflow of Durkar Drain to the River Calder is flapped. It is expected that the flap will close for events greater than the 1-in-2-year flood on the River Calder. A further flap valve at the area known as 'Pugneys Entrance' closes during a 1-in-40-year flood event on the River Calder. This results in the Durkar Drain being 'tidelocked'. Durkar Drain is an ordinary watercourse and so is the responsibility of Wakefield Council.

Wakefield Council has purchased mobile pumps that can be deployed across their council area. Therefore, emergency pumps could be deployed to pump excess water from Durkar Drain to the River Calder during flood events. However, consultations with Wakefield Council indicate that this is not certain.

Therefore, the design runs for to test the surface water drainage system at Calder Park have made a conservative assumption that emergency pumps are not deployed to Durkar Drain. In this case water levels in Durkar Drain will backup and so a downstream level of 25.2mAOD has been applied. Under this scenario there will be backflow to the Calder Park site. In normal circumstances this situation should be avoided because of the risk of polluted water stagnating within the system of swales at Calder Park.

If emergency pumps are deployed, they should lower levels in Durkar Drain provided it is kept clear downstream of Denby Dale Road. With these lower levels (24.5mAOD) there would be no backflow on to the Calder Park site. In this case the drainage system would only be required to store runoff from Calder Park.

2.6 Culvert capacity

The culvert under the Denby Dale Road is a 600m diameter pipe. The invert level of the culvert is approximately 23.27mAOD. This information contrasts with a spot level survey (Donaldson Edwards Partnership (DEP), June 2005) which indicated that the invert at the culvert entrance is 23.49mAOD. This apparent discrepancy is probably caused by the large amount of siltation at the culvert. It is likely that the DEP level is the effective invert level due to siltation of the bed (i.e. 0.22m of silt blocking the bottom of the culvert). At the time WSP stated it was not possible to determine the invert of the pipe through the silt. The WSP information (from heights relative to footpath levels) indicated a downstream invert level of 23.22mAOD. The road is approximately 30m wide, which would give the culvert a slope of 1 in 500.

Surface runoff from Calder Park will not be restricted. Modelling (with a clear culvert) indicates that unrestricted flow leaving the site will exceed the agricultural runoff rates. This should not cause problems for Durkar Drain because in general rainfall on the site will enter Durkar Drain before the River Calder rises, 'tidelocking' the drain.

When the Durkar Drain is 'tidelocked', emergency pumps may be deployed by Wakefield Council to pump some of the flow that could be expected in Durkar Drain. If the culvert were totally blocked water levels could rise to 25.2mAOD, the boundary condition used to test the performance of the surface water drainage system at Calder Park (see 2.4).

3 Conclusions

3.1 Surface water drainage

The drainage scheme for Calder Park uses SuDS principles as an integral part of the site design. A series of wide ditches run alongside the roads and between the development platforms. There are also a number of lakes that form part of the drainage scheme as well as providing an aesthetic attraction.

Generally peak levels in the drainage system across the site were caused by consecutive storm events. This is because the model tests used high water levels at Denby Dale Road (25.2mAOD) that prevent runoff escaping to Durkar Drain. Therefore, water levels in the drainage system are sensitive to the runoff volume as opposed to the rainfall intensity.

Model simulations of the surface water drainage system have been undertaken for the proposed completed drainage system with a full developed business park. The simulations tested the impact of increased runoff from the proposed development on water levels in the surface water drainage system.

The effect of increased runoff from the site of the proposed development due to a greater impermeable area was shown to have a marginal impact on water levels. The maximum increase in water level of 0.01m was confined to just two ditches (number 29 and 38). However, simulated flood levels remained in bank throughout the ditch system.

With respect to the proposed Tungsten Development, flood levels in the Deep Water pond and surrounding ditches/ swales are of sufficient volume to prevent flooding of the Property. Under a 1 in 100-year event with an extra 40% allowance for climate change these is a freeboard of over 1m in the drainage system surrounding the site. Therefore, the surface water network can accommodate the free discharge of runoff from the Property based on an impermeable area of 51,950m^2.

3.2 Durkar Drain

The discharge of surface water from Calder Park is to Durkar Drain via a culvert under Denby Dale Road. As part of the Wakefield Flood Defence Strategy developed by the EA, the outflow of Durkar Drain to the River Calder flows through two flapped structures. As a results, during a 1-in-40-year flood event on the River Calder, Durkar Drain is 'tidelocked'. Durkar Drain is an ordinary watercourse and so is the responsibility of Wakefield Council.

Wakefield Council has purchased mobile pumps that can be deployed across their council area. Therefore, emergency pumps could be deployed to pump excess water from Durkar Drain to the River Calder during flood events. However, consultations with Wakefield Council indicate that this is not certain.

Therefore, design runs for this FRA have made a conservative assumption that emergency pumps are not deployed to Durkar Drain. In this case water levels in Durkar Drain will backup and so a downstream level of 25.2mAOD has been applied. Under this scenario there will be backflow to the Calder Park site. In normal circumstances this situation should be avoided because of the risk of polluted water stagnating within the system of swales at Calder Park.

If emergency pumps are deployed, they should lower levels in Durkar Drain provided it is kept clear downstream of Denby Dale Road. With these lower levels (24.5mAOD) there would be no backflow on to the Calder Park site. In this case the drainage system would only be required to store runoff from Calder Park.

Appendix A Proposed development at Calder Park

DO NOT SCALE!
ALL DIMENSIONS SHOULD BE CHECKED ON SITE BEFORE WORK COMMENCES

BOUNDARY SUBJECT TO TITLE PLAN CHECK

SUBJECT TO REVIEW OF LEVELS AND FALLS

Rev.	Date	Description	Drawn
C	29.09.20	Office area amended to include SF. Areas updated to include gatehouse / control tower. Parking numbers added. Note for attenuation tank added.	DW
B	05.08.20	Sprinkler tanks moved	DW
A	04.08.20	Boundary amended inline with information from solicitors - drawing Calder Park - Ordnance Survey Mastermap. Site plan amended accordingly and detailing updated.	DW

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project
Calder Park, Wakefield

drawing title
Proposed Site Plan

date August 2020
status Feasibility
scale 1:500 @ A0
drawn DW checked
job no. 2479 dwg no. F404 rev. C

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REPORT on the

DRAINAGE STRATEGY

for the

PROPOSED DEVELOPMENT

at

CALDER PARK COMMERCIAL DEVELOPMENT

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Reference: 63150	-	-

CONTENTS

EXECUTIVE SUMMARY

This drainage strategy report has been written in support of a planning application to develop of 15.215ha of land for a large commercial unit. The development is located adjacent to Peel Avenue, Calder Park Commercial Estate, Wakefield within 1km of junction 39 of the M1 motorway.

Foul effluent from the development will be discharged ultimately to an existing foul sewer located ithin Peel Avenue which has been installed by the vendor to support the development. All foul sewerage on site and within Peel Avenue is private and so a pre development enquiry has not been submitted toi Yorkshire Water.

Surface water from the development will be freely discharging into large deep swales surrounding three boundaries on the site, with a balancing pond located on the eastend boundary. A previous Surface Water Assessment report prepared by JBA Consulting indiactes that the peripheral surface water drainage infrastucture has been designed to cater for all the surface water run-off from the site and attenuation and SuDS mitigation measures are within the entire surface water drainage network.

The on-site and off-site network has been designed for a 1 in 100 year storm event plus 40% allowance for climate change and a factor of safety of 2 included in the calculations. Approval will be required from the regulatory bodies before the detail design is finalised.

The use of soakaways and other infiltration techniques has been considered but not deemed viable since the site is underlain by several meters of made ground which is predominantly of a impermeable cohesive nature and so in accordance with the SuDS hierarchy the surface water is being discharged into the surrounding swales.

Water quality will be improved through the use of permeable paving for the car parking areas. However, there is little opportunity to provide further public amenity or biodiversity to the site. Although there are some areas which will be landscaoed and shown on the architects appended plans.

This report demonstrates that a suitable Sustainable Urban Drainage System (SuDS) can be implemented for this scheme which complies with the NPPF. Both water quantity and water quality issues have been addressed and demonstrate that the environmental issues will not be compromised as a result of this scheme.

1. INTRODUCTION

1.1. This drainage strategy report has been commissioned by Carbide Properties Limited who are proposing to construct a 304,550sq ft (2.83ha) warehouse unit. The new unit will occupy most of the land with the remaining area being concrete yard and permeable car parking. There will be offices and car parking for 302 cars, along with 22 lorry delivery docks and a driver rest area allocated within the new design.

1.2. Around all boundaries there is a landscaping margin which is up to 19m wide along the Peel Avenue frontage.

1.3. The objective of the report is to demonstrate that proper consideration has been given to SuDS principles for the surface water drainage design. This report summarises the proposals in the context of current legislation.

1.4. In terms of flood risk, this has been addressed in a separate flood risk report titled Calder Park – Tungsten Development. Surface Water Assessment - prepared by JBA Consulting Dated November 2020. This report is an update of the original Flood Risk Report dated January 2018, at the time of writing the original report there was no site layout and so an assumption was made with regard to the impermeable area. This original report was updated to account for the proposed site layout and appended to this report. Although the report principally deals with the flood risk to the site, the issue of surface water outfalls from the site is covered extensively, this is because the run-off from this site is just a small part of a much larger private surface water network which covers the whole of Calder Park.

1.5. The site is located adjacent to the northern side of Peel Avenue, Wakefield WF4 3FL and is currently undergoing enabling works including significant earthworks to raise the levels. Peel Avenue is a private road and is connected at both ends to Denby Dale Road.

1.6. The proposal includes the construction of two separate accesses, one for car parking and one for commercial vehicles but both will enter from Peel Avenue. Large delivery and service vehicles will directly enter the service yard from the highway to the front of the building, this area will be designed as impermeable with a gate house control point. The car park will be located at the eastern side of the main warehouse and will consist of impermeable circulatory route with parking bays to be designed to a permeable pavement specification.

1.7. The layout is outlined in drawings produced by HTC Architects and this is appended to this report. Discussions with the client and the highways engineer have revealed that there will be no adoptable roads within the industrial estate. It is understood that on-site sewerage will remain private and therefore will NOT be adopted by Yorkshire Water. SuDS features are to be maintained by the owner via a management company and not the local water authority.

1.8. The floor level for the building has been set at 27.00m AOD which is in line with the original flood risk assessment report as this is at a similar level to the 1 in 1000-year level for the washland adjacent to the River Calder (27.04mAOD). Given that the building will be nominally 150mm higher than the surrounding ground level this is considered adequate and in line with the JBA Consulting proposals.

1.9. The general conditions contained in the appendices are applied to this report and it should be read and construed accordingly.

2. SITE LOCATION AND INFORMATION

2.1. The site is located within Calder Park Commercial Estate on Peel Avenue, Wakefield, WF4 3FL which is a private road off A636, Denby Dale Road. The site is located approximately 1km east of junction 39 of the M1 and has a NGR of 431822, 417996. The site has an area of 15.215ha and is presently open waste land undergoing enabling works.

2.2. The site is generally flat although at the present time it is undergoing earthworks to raise the levels and create a plateau of 26.500m AOD. The site is bounded on the north by the River Calder catchment and a flood embankment (managed by the Environment Agency), to the east there is an attenuation pond (Deepwater) and to the west there is a new police station and forensic services building. On the northern, southern and western boundaries there is a large swale which in some places is 3m wide at the bottom, this is designed to act as attenuation and also convey the surface water through the surface water network. The final outfall for the whole of the Calder Park development is at Durkar drain to the south of the site.

2.3. The immediate surrounding land uses are:

Boundary	Adjacent Land Use	Proximal Land Use
North	Open countryside	Open countryside
East	Business unit	Commercial business unit
South	Private highway	Commercial business unit
West	Business units	Commercial business unit

3. PROPOSED DRAINAGE SCHEME

3.1. Foul water – Existing Conditions

3.1.1. For this type of development, it is usual to submit a Pre-Development enquiry to Yorkshire Water which would detail where the foul and surface water sewers are located and at what invert level however, this has not been undertaken since all the foul and stormwater sewers remain private for this development.

3.1.2. The site has been sold as a serviced site and consequently foul water connections are being provided, a copy of the engineers drawing is appended to this report to show the location and depths of these private sewers.

3.2. Foul Water Drainage Proposals

3.2.1. There are generally three methods for dealing with foul effluent that is produced from a commercial development: -

A) Public/private sewers – The use of private sewers is always the preferred option since they are relatively maintenance free and require no additional area to discharge the foul water.

B) Septic tank – This is quite simply a tank, usually buried below ground, which collects the foul effluent and is periodically emptied, occasionally it also has a herringbone drainage system fitted for removing the 'clean' water that collects in the upper layers of the tank, these are seldom used in a commercial context, as they require high levels of maintenance and can produce foul odours if not maintained correctly. A septic tank would be unsuitable for a development of this type for a multiple number of reasons including lack of available space within a densely occupied area. The Environment Agency are also preventing the installation of new septic tanks and encouraging the installation of small treatment stations where other proposals are not suitable or not available.

C) A small treatment works – These are generally used where there is no public sewer within the vicinity, but only if the General Binding Rules for Small Sewage Discharges are met, or if a permit is obtained from the Environment Agency. Small treatment works discharge either to the ground or to surface water. Treatment works are seldom used where foul sewers are available for connections.

3.2.2. As stated above, the preferred option for this type of development is to discharge into a public/private foul sewer. Alan Wood and Partners have provided sewer details showing the approximate point of connection and confirmation that proposed outfall can discharge into the private sewers.

3.2.3. An indirect section 106 application will be required for the indirect connection into the public sewer with the appropriate application fee but this should not be a significant issue to obtain during the design stage.

3.2.4. The architectural site plans indicate that the new unit will be larger than the existing unit and so there may well be an increase in staff number. For the purposes of initial design, reference has been made to British Water Code of Practice Loads and Flows – 4, it is assumed that the warehouse will have canteen facilities and so the flow/day for a commercial development is 100l/day. The layout indicates that there is 302 car parking and so the predicted maximum daily flow of 30 200litres or 0.4l/sec based upon a 24-hour day, at full warehouse capacity. It should be noted that some of the parking spaces will be used for visitors and therefore the above figure could be seen as a conservative value.

3.2.5. It should be noted that the effluent from any staff canteen will be fitted with a proprietary grease trap to ensure that fat and grease are not discharged into the main foul sewer. The grease trap will also be maintained as part of the unit's maintenance and health and hygiene regime.

4. SURFACE WATER DRAINAGE SYSTEM

4.1. Existing Conditions

4.1.1. The site has not been developed previously and prior to construction of Calder Park it was open countryside and no doubt the run off from the fields passed into a series of ditches, however as Calder Park has been developed the surface water drainage system, as described in the flood risk assessment, for the whole development has been installed.

4.1.2. The site is surrounded on three sides with swales and these are up to 3m wide at the bottom on the northern elevation, the swales have been designed to accommodate run-off from this site and it has been designed to not flood for a 1 in 100 year event plus 40% for climate change. The swales act as carrier drains and provide a treatment train for SuDS purposes.

4.1.3. The existing network is a combination of lateral weirs (sometimes incorporating culverts) and open channel junctions to connect swales and ditches to lakes.

4.2. Surface Water Drainage Proposals

4.2.1. Government directives in the form of the Building Regulations Part H3 indicate that the following hierarchy should be adopted as a means disposing of the surface water run-off: -

- Soakaways
- Discharge into a watercourse at an appropriate rate.
- Discharge into a surface water sewer at an agreed rate.

4.2.2. In accordance with The SuDS Manual, the design of the surface water system should consider water quantity, water quality, amenity and biodiversity and this report considers each of these four categories.

4.2.3. **Water Quantity** – the whole development has been designed as a single network with attenuation and flow control throughout the system, the HEC-RAS model shown in the report assumes free flowing discharge into the swales and this flows around the network and eventually outfalls into Durkar Drain. Surface water modelling indicated in clause 2.4.3 of the FRA (1 in 100 year plus 40% for climate change) indicates that the maximum water level in Deep Water will not exceed 25.330m AOD which is over 1m lower than the finished plateau level. Therefore an unattenuated outflow from the site will not constitute a problem to downstream flooding.

4.2.4. The system has been designed for an impermeable are of 51 950m^2 which represents all hard surfacing and building across the site, however there will be 303 car parking spaces and associated access roads which will be designed as porous paving and although this will be designed as a type 3 system not all the run-off will outfall the site, this area equates to 7094m^2. The outflow from this area will be attenuated as the run-off passes through the Corse granular aggregate below.

4.2.5. The onsite surface water network will remain private, but approval from the regulator will still be required.

4.2.6. Given that the runoff from the development as a whole will be restricted to the agreed run-off from the site, water quantity issues have been addressed.

4.2.7. **Water Quality** – With reference to The SuDS Manual, CIRIA C753, consideration is also given to water quality. The opportunities to provide an improvement to the water quality are limited as the site will be developed in keeping with the current surroundings but nevertheless some SuDS techniques are discussed below. With reference to the architect's site layout drawings and table 4.3 of the SuDS Manual the pollution hazard level is low for a majority of the site, however the service areas around the buildings are considered to be a medium pollution hazard potential.

4.2.8. Green Roofs - both extensive and intensive green roofs have been considered; given the infrastructure that will be present on the roof and the increase in structural support and maintenance required it would be unlikely that a green roof would be cost effective for this type of development. It is also extremely unlikely that the planning officers would permit this in view of the fact that none of the surrounding buildings have these types of roofs. On this basis, green roofs are not considered appropriate.

4.2.9. Blue Roofs – Blue roofs are flat roofs used to hold water and release it over a period of time; these are not viable for the site. A Blue roof is not in keeping with the local architecture and would require significant additional structural support.

4.2.10. Infiltration Systems – These include infiltration blankets, soakaways, trenches and infiltration basins. Filter drains improve water quality by reducing pollutant levels in runoff by filtering out fine sediments, metals, hydrocarbons and other pollutants. They also encourage adsorption and biodegradation processes. The use of filter drains on this site has been considered and although no permeability testing has been undertaken, filter drains are not considered appropriate since the underlying soil is predominantly cohesive and the amount of standing water on site indicates that infiltration systems are unlikely to work on this site. the limited infiltration that is available will be maximized by discharging all the run-off into the surrounding swales and detention basin.

4.2.11. Pervious Pavements - It is proposed to construct all 300 car parking spaces as permeable paving so that the surface water is cleaned as it passes through coarse graded aggregate sub-base. Due to the underlying geology, a Type C system will be used, with filter and carrier pipes to distribute the attenuated run-off throughout the permeable paving structure. The pipes will be laid in trenches integral with the sub-base. The main access routes into the car parking and to the offices will be in impermeable bituminous construction but the roads will be cambers so that all the run-off passes through the porous paving car parking.

4.2.12. However, it is not possible to use permeable paving for the service yards, as the weight of Heavy Goods Vehicles would damage the permeable surfaces and it would degrade very quickly. There is only a low to medium potential hazard for pollutants from this site and the use of porous paving is considered entirely appropriate for a development of this type. The exact type of porous paving is not known at present and will have to be approved by the planning officer; however, the use of concrete block permeable paving is the preferred option. With reference to Table 26.3 of the SuDS Manual permeable paving would provide suitable SuDS mitigation indices for a development of this type and therefore when combined with the development wide attenuation, the paving construction would provide an adequate storm water treatment train for the site by filtering out the silts and hydrocarbons.

4.2.13. The pervious pavements will be maintained by the end user in accordance with the section 6 of this report. The maintenance schedule will be included in the Operation and Maintenance manual for the buildings for future reference.

4.2.14. **Amenity** - defined as ''a useful or pleasant facility or service'' in the SuDS Manual. The development has the opportunity to provide an amenity within the landscaped areas and with a suitably landscaped attenuation pond the development is considered suitable.

4.2.15. **Biodiversity** – there are soft landscaped areas along all four boundaries to this site with the widest margin being up to 19m wide on the front elevation and this will provide some biodiversity within the development. On the eastern side of the site there is an attenuation pond which will be sympathetically landscaped and planted to provide a suitable area for wildlife.

4.2.16. A plan showing the on site preliminary drainage strategy for the development is appended to this report in Appendix III.

4.3. Design Consideration of Exceedance

4.3.1. Whilst the offsite network has been designed to demonstrate that there will be no flooding for a 1 in 100 year event plus 40% for climate change, it is important to consider exceedance, in the event of an exceptional rainfall event, that is with a return period of over 100 years, or in the event of a system failure. Should this occur, the water would back up into the impermeable yard area and the spare capacity within the permeable paving attenuation. No allowance will be made for the storage in the permeable paving sub-base in the normal case. Should the sub-base become completely saturated, water would pond above ground within the car park area. The proposed level in the porous paved areas will also be retained by a 125mm half battered kerb.

4.3.2. The external levels will generally be set 150mm lower than the ground floor slab although level thresholds will be provided, this will further reduce the propensity for any run-off to enter into the buildings in the event of exceedance occurring.

4.3.3. The measures described above will be sufficient to provide enough time for employees and guests to exit the building in the event of an abnormal flood occurring.

5. MAINTENANCE OF THE SURFACE WATER DRAINAGE SYSTEM

5.1. It is recognised that the surface water drainage system has been designed with future maintenance in mind. Pervious paving construction design and catch-pits will trap the vast majority of silt to prevent entry into the attenuation tank.

5.2. A maintenance and management plan has been proposed and is appended to this report.

5.3. The measures are suggestions and the frequency of maintenance should be adjusted if necessary, to suit levels of silt build up and defects in the system. The schedule appended to this report will be included in the Operation and Maintenance manual for future reference.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1. Foul water – the foul effluent from the development can be discharged into the existing foul sewers located along Peel Avenue. The foul effluent will discharge into an existing private sewer as confirmed by the vendor. The connection to a private sewer will be subject to a S106 agreement with Yorkshire Water.

6.2. Surface water – The surface water drainage system from the development will discharge into swales and an attenuation pond surrounding the site, further treatment trains will be provided by discharging the surface water from the car parking areas into the permeable paving prior to discharging into the balancing pond.

6.3. This report demonstrates that the opportunities for suitable Sustainable Urban Drainage Systems are limited for this site but sufficient techniques can be implemented to comply with the NPPF. Both water quantity and water quality issues have been addressed and enhanced, the report also demonstrates that environmental issues will not be compromised as a result of this scheme.

6.4. A management company will maintain the surface water drainage system, and a maintenance and management plan has been appended to this report.

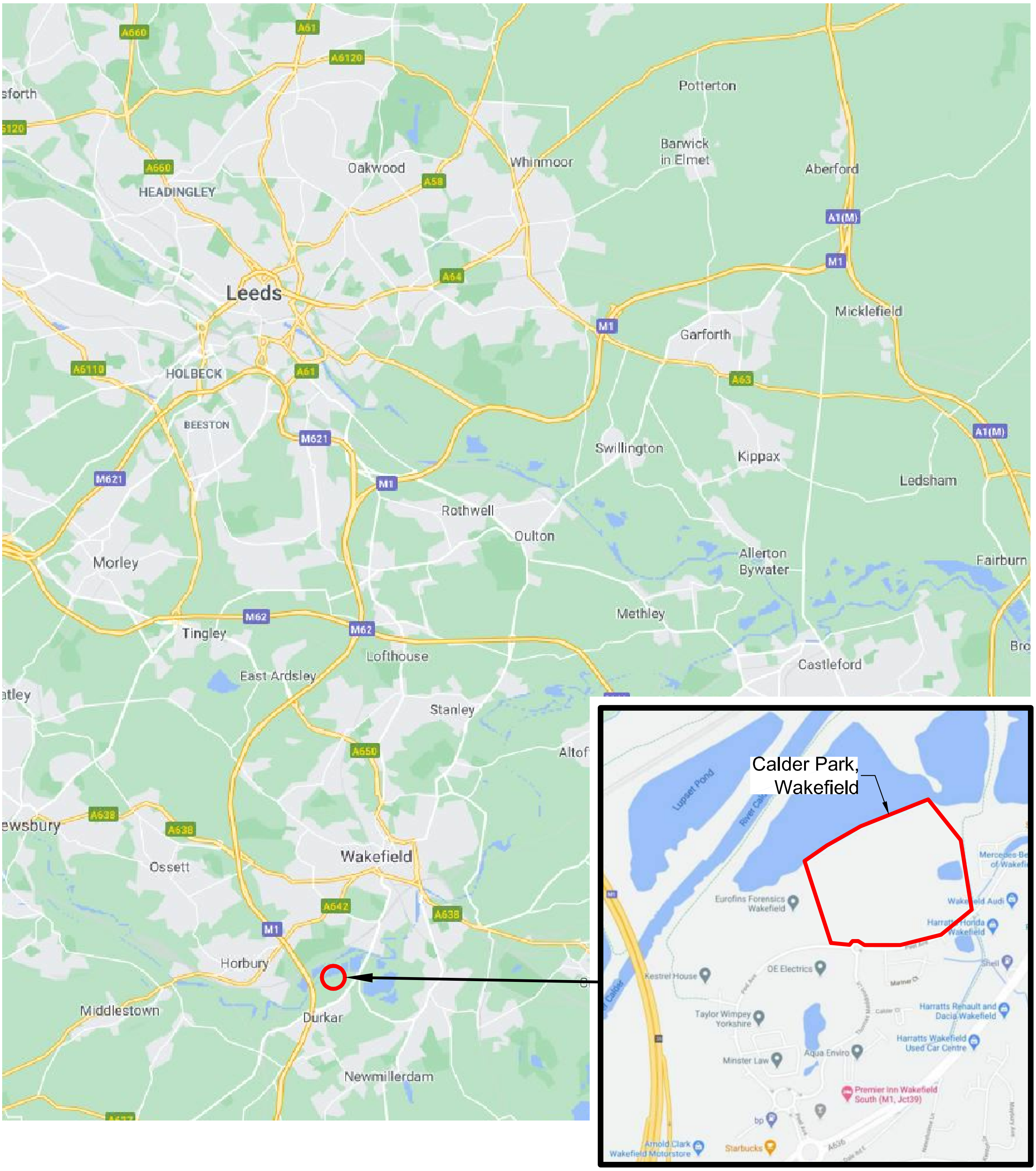
Paul Daniel,
Drainage Engineer,
PRP UK Ltd

B. M. Smith

Barry Smith
Director
PRP UK Ltd

APPENDIX I

SITE LOCATION PLAN



Project:

Calder Park, Wakefield

Title:

Site Location Plan

PRP:
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Status:

PRELIMINARY

Engineer:	KH	Date:	Nov 2020
Drawn:	GF	Scales @ A4:	NTS
Checked:	BMS		
Project No:	63128	Drg No: SK01	Rev: P1

APPENDIX II

ARCHITECT LAYOUT PLAN

DO NOT SCALE!
ALL DIMENSIONS SHOULD BE CHECKED ON SITE BEFORE WORK COMMENCES

BOUNDARY SUBJECT TO TITLE PLAN CHECK

SUBJECT TO REVIEW OF LEVELS AND FALLS

Rev.	Date	Description	Drawn
D	05.11.20	Amended inline with client instruction	DW
C	29.09.20	Office area amended to include SF. Areas updated to include gatehouse / control tower. Parking numbers added. Note for attenuation tank added.	DW
B	05.08.20	Sprinkler tanks moved	DW
A	04.08.20	Boundary amended inline with information from solicitors - drawing Calder Park - Ordnance Survey Mastermap. Site plan amended accordingly and detailing updated.	DW

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client
Tungsten

project
Calder Park, Wakefield

drawing title
Proposed Site Plan

date August 2020
status Feasibility
scale 1:500 @ A0
drawn DW checked
job no. 2479 dwg no. F404 rev. D

APPENDIX III

PROPOSED PRELIMINARY DRAINAGE LAYOUTS

Notes:

1. DO NOT SCALE FROM THIS DRAWING.
2. All dimensions are in millimetres Unless Noted Otherwise (u.n.o.)
3. Drawing is to be read in conjunction with all relevant architect's drawings. Any inconsistencies should be reported to PRP immediately.
4. All levels and dimensions are to be checked on site before any work commences.
5. For more information see PRP drawings:
   63104 - 100series - Drainage and External Works
   63104 - 200series - Foundations
   63104 - 300series - Superstructure
6. The Health and Safety at Work act is to be complied with at all times. Attention is drawn to the wearing of hard hats, safety boots, reflective clothing, and the use of any other required safety equipment.

Drainage:

1. The position, line, level and diameter of all existing drainage apparatus should be confirmed on site prior to the commencement of the works. Any discrepancies should be reported to PRP immediately.
2. The connection of foul and surface water drainage to the existing public sewer system shall be subject to the approval of the water authority
3. For positions of all rainwater pipes & foul outlets refer to Architect's drawings.
4. All drainage works shall be carried out in accordance with WRc "Sewers for Adoption - 7th edition"
5. All joints between precast manhole components shall have a minimum uncompressed thickness of 10mm of proprietary bitumen or resin mastic sealant.
6. Chambers & manholes with outgoing pipes of greater than Ø600mm shall be fitted with removable stainless steel (grade 316) safety chains or polypropylene rope.
7. Storm & foul branch connections are to be laid at gradients of between 1:10 & 1:80
8. All in-situ concrete shall be minimum grade GEN3.
9. Precast concrete cover & reducing slabs to be heavy duty reinforced concrete to BS 5911.
10. Rising mains shall be black MDPE SDR11 as WI 4-32-03 & joints & fittings to be in accordance with WI 4-32-04. Other approved pipe materials to be in accordance with their relevant BS.
11. Manhole covers & frames shall be manufactured in cast iron or ductile iron & shall comply with requirements of BS EN 124 & shall be kite marked or equivalent.
12. Where there is no intermediate manhole between the start of a surface water pipe run and the soakaway the gradient of the run shall be not less than 1 : 60.
13. All completed work shall be suitably protected from damage by construction work. Damaged drainage will not be accepted. It is recommended that no heavy loading or underground work is permitted above or near unprotected drainage, and that dumpers, trucks, fork lifts or other heavy vehicles are not driven along or near pipe runs.
14. Inspection chambers, soakaways and flow control units are to be installed strictly in accordance with manufacturer guidance and instructions

Drainage Legend:

- Existing Surface Water
- Existing Foul
- S New Surface Water
- F New Foul
- Perforated Pipe
- RE Rodding Eye
- Road Gully
- Foul Gully
- ACO / Channel Drain
- Type C Permeable Paving
- Existing Levels
- +100.000 New Levels
- 1/X Slope
- Flood Exceedance Route

Rev	Date	Description	By	Chk
P1	02/12/2020	Issued For Planning	SK	/BMS

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Client:
Tungsten Properties Ltd

Architect:
HTC Architects

Project:
Calder Park, Wakefield

Title:
Drainage Strategy

Status:
PRELIMINARY

Engineer:	SK	Date:	NOVEMBER 2020
Drawn:	SK	Scales @ A0:	1:500
Checked:	BMS		
Drawing No:	101	Rev:	P1

APPENDIX IV

MICRODRAINAGE CALCULATIONS

Summary of Results for 100 year Return Period (+40%)

Half Drain Time exceeds 7 days.

Outflow is too low. Design is unsatisfactory.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Volume (m³)	Status
15 min Summer	26.305	0.305	0.0	16.9	O K
30 min Summer	26.365	0.365	0.0	24.1	O K
60 min Summer	26.421	0.421	0.0	32.0	O K
120 min Summer	26.473	0.473	0.0	40.5	O K
180 min Summer	26.502	0.502	0.0	45.6	Flood Risk
240 min Summer	26.521	0.521	0.0	49.2	Flood Risk
360 min Summer	26.548	0.548	0.0	54.3	Flood Risk
480 min Summer	26.567	0.567	0.0	58.3	Flood Risk
600 min Summer	26.582	0.582	0.0	61.4	Flood Risk
720 min Summer	26.594	0.594	0.0	64.0	Flood Risk
960 min Summer	26.614	0.614	0.0	68.2	Flood Risk
1440 min Summer	26.640	0.640	0.0	74.3	Flood Risk
2160 min Summer	26.666	0.666	0.0	80.4	Flood Risk
2880 min Summer	26.683	0.683	0.0	84.6	Flood Risk
4320 min Summer	26.705	0.705	0.0	90.1	Flood Risk
5760 min Summer	26.718	0.718	0.0	93.5	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Time-Peak (mins)
15 min Summer	125.869	0.0	27
30 min Summer	84.215	0.0	42
60 min Summer	53.779	0.0	72
120 min Summer	33.180	0.0	132
180 min Summer	24.659	0.0	192
240 min Summer	19.841	0.0	252
360 min Summer	14.545	0.0	372
480 min Summer	11.669	0.0	492
600 min Summer	9.828	0.0	612
720 min Summer	8.537	0.0	732
960 min Summer	6.830	0.0	972
1440 min Summer	4.978	0.0	1452
2160 min Summer	3.621	0.0	2172
2880 min Summer	2.885	0.0	2892
4320 min Summer	2.092	0.0	4332
5760 min Summer	1.663	0.0	5776

Summary of Results for 100 year Return Period (+40%)

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Volume (m³)	Status
7200 min Summer	26.727	0.727	0.0	95.8	Flood Risk
8640 min Summer	26.733	0.733	0.0	97.2	Flood Risk
10080 min Summer	26.736	0.736	0.0	98.3	Flood Risk
15 min Winter	26.328	0.328	0.0	19.5	O K
30 min Winter	26.390	0.390	0.0	27.6	O K
60 min Winter	26.449	0.449	0.0	36.4	O K
120 min Winter	26.504	0.504	0.0	46.0	Flood Risk
180 min Winter	26.534	0.534	0.0	51.7	Flood Risk
240 min Winter	26.554	0.554	0.0	55.7	Flood Risk
360 min Winter	26.583	0.583	0.0	61.5	Flood Risk
480 min Winter	26.603	0.603	0.0	65.9	Flood Risk
600 min Winter	26.619	0.619	0.0	69.5	Flood Risk
720 min Winter	26.632	0.632	0.0	72.4	Flood Risk
960 min Winter	26.653	0.653	0.0	77.2	Flood Risk
1440 min Winter	26.681	0.681	0.0	84.1	Flood Risk
2160 min Winter	26.709	0.709	0.0	91.1	Flood Risk
2880 min Winter	26.728	0.728	0.0	95.9	Flood Risk
4320 min Winter	26.752	0.752	0.0	102.5	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Time-Peak (mins)
7200 min Summer	1.391	0.0	7208
8640 min Summer	1.201	0.0	8648
10080 min Summer	1.063	0.0	10088
15 min Winter	125.869	0.0	27
30 min Winter	84.215	0.0	42
60 min Winter	53.779	0.0	72
120 min Winter	33.180	0.0	132
180 min Winter	24.659	0.0	192
240 min Winter	19.841	0.0	250
360 min Winter	14.545	0.0	370
480 min Winter	11.669	0.0	488
600 min Winter	9.828	0.0	608
720 min Winter	8.537	0.0	726
960 min Winter	6.830	0.0	964
1440 min Winter	4.978	0.0	1442
2160 min Winter	3.621	0.0	2152
2880 min Winter	2.885	0.0	2864
4320 min Winter	2.092	0.0	4284

Summary of Results for 100 year Return Period (+40%)

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Volume (m³)	Status
5760 min Winter	26.768	0.768	0.0	106.8	Flood Risk
7200 min Winter	26.778	0.778	0.0	109.6	Flood Risk
8640 min Winter	26.786	0.786	0.0	111.7	Flood Risk
10080 min Winter	26.792	0.792	0.0	113.2	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Time-Peak (mins)
5760 min Winter	1.663	0.0	5712
7200 min Winter	1.391	0.0	7136
8640 min Winter	1.201	0.0	8560
10080 min Winter	1.063	0.0	9896

Rainfall Details

Rainfall Model	FSR	Winter Storms	Yes
Return Period (years)	100	Cv (Summer)	0.750
Region	England and Wales	Cv (Winter)	0.840
M5-60 (mm)	19.000	Shortest Storm (mins)	15
Ratio R	0.360	Longest Storm (mins)	10080
Summer Storms	Yes	Climate Change %	+40

Time Area Diagram

Total Area (ha) 0.091

Time From:	(mins) To:	Area (ha)	Time From:	(mins) To:	Area (ha)	Time From:	(mins) To:	Area (ha)
0	4	0.031	4	8	0.030	8	12	0.030

Model Details

Storage is Online Cover Level (m) 26.800

Porous Car Park Structure

Infiltration Coefficient Base (m/hr)	0.00000	Width (m)	15.1
Membrane Percolation (mm/hr)	1000	Length (m)	60.0
Max Percolation (l/s)	251.7	Slope (1:X)	80.0
Safety Factor	2.0	Depression Storage (mm)	5
Porosity	0.30	Evaporation (mm/day)	3
Invert Level (m)	26.000	Membrane Depth (m)	1

Summary of Results for 100 year Return Period (+40%)

Half Drain Time exceeds 7 days.

Outflow is too low. Design is unsatisfactory.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Volume (m³)	Status
15 min Summer	26.292	0.292	0.0	46.0	O K
30 min Summer	26.353	0.353	0.0	67.2	O K
60 min Summer	26.409	0.409	0.0	90.3	O K
120 min Summer	26.462	0.462	0.0	115.1	O K
180 min Summer	26.491	0.491	0.0	129.9	O K
240 min Summer	26.510	0.510	0.0	140.3	Flood Risk
360 min Summer	26.536	0.536	0.0	155.3	Flood Risk
480 min Summer	26.555	0.555	0.0	166.6	Flood Risk
600 min Summer	26.570	0.570	0.0	175.6	Flood Risk
720 min Summer	26.582	0.582	0.0	183.1	Flood Risk
960 min Summer	26.601	0.601	0.0	195.1	Flood Risk
1440 min Summer	26.627	0.627	0.0	212.2	Flood Risk
2160 min Summer	26.651	0.651	0.0	228.9	Flood Risk
2880 min Summer	26.667	0.667	0.0	240.2	Flood Risk
4320 min Summer	26.686	0.686	0.0	254.3	Flood Risk
5760 min Summer	26.697	0.697	0.0	262.3	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Time-Peak (mins)
15 min Summer	125.869	0.0	27
30 min Summer	84.215	0.0	42
60 min Summer	53.779	0.0	72
120 min Summer	33.180	0.0	132
180 min Summer	24.659	0.0	192
240 min Summer	19.841	0.0	252
360 min Summer	14.545	0.0	372
480 min Summer	11.669	0.0	492
600 min Summer	9.828	0.0	612
720 min Summer	8.537	0.0	732
960 min Summer	6.830	0.0	972
1440 min Summer	4.978	0.0	1452
2160 min Summer	3.621	0.0	2172
2880 min Summer	2.885	0.0	2892
4320 min Summer	2.092	0.0	4332
5760 min Summer	1.663	0.0	5768

Summary of Results for 100 year Return Period (+40%)

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Volume (m³)	Status
7200 min Summer	26.703	0.703	0.0	266.7	Flood Risk
8640 min Summer	26.706	0.706	0.0	268.8	Flood Risk
10080 min Summer	26.707	0.707	0.0	269.8	Flood Risk
15 min Winter	26.315	0.315	0.0	53.5	O K
30 min Winter	26.378	0.378	0.0	77.3	O K
60 min Winter	26.437	0.437	0.0	103.3	O K
120 min Winter	26.493	0.493	0.0	131.1	O K
180 min Winter	26.523	0.523	0.0	147.7	Flood Risk
240 min Winter	26.543	0.543	0.0	159.4	Flood Risk
360 min Winter	26.571	0.571	0.0	176.3	Flood Risk
480 min Winter	26.592	0.592	0.0	189.1	Flood Risk
600 min Winter	26.608	0.608	0.0	199.3	Flood Risk
720 min Winter	26.620	0.620	0.0	207.8	Flood Risk
960 min Winter	26.640	0.640	0.0	221.5	Flood Risk
1440 min Winter	26.668	0.668	0.0	241.0	Flood Risk
2160 min Winter	26.694	0.694	0.0	260.4	Flood Risk
2880 min Winter	26.712	0.712	0.0	273.7	Flood Risk
4320 min Winter	26.734	0.734	0.0	290.9	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Time-Peak (mins)
7200 min Summer	1.391	0.0	7208
8640 min Summer	1.201	0.0	8648
10080 min Summer	1.063	0.0	10088
15 min Winter	125.869	0.0	27
30 min Winter	84.215	0.0	42
60 min Winter	53.779	0.0	72
120 min Winter	33.180	0.0	132
180 min Winter	24.659	0.0	190
240 min Winter	19.841	0.0	250
360 min Winter	14.545	0.0	370
480 min Winter	11.669	0.0	488
600 min Winter	9.828	0.0	606
720 min Winter	8.537	0.0	726
960 min Winter	6.830	0.0	964
1440 min Winter	4.978	0.0	1438
2160 min Winter	3.621	0.0	2152
2880 min Winter	2.885	0.0	2860
4320 min Winter	2.092	0.0	4284

Summary of Results for 100 year Return Period (+40%)

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Volume (m³)	Status
5760 min Winter	26.747	0.747	0.0	301.2	Flood Risk
7200 min Winter	26.755	0.755	0.0	307.6	Flood Risk
8640 min Winter	26.760	0.760	0.0	311.6	Flood Risk
10080 min Winter	26.763	0.763	0.0	314.2	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Time-Peak (mins)
5760 min Winter	1.663	0.0	5712
7200 min Winter	1.391	0.0	7072
8640 min Winter	1.201	0.0	8480
10080 min Winter	1.063	0.0	9888

Rainfall Details

Rainfall Model	FSR	Winter Storms	Yes
Return Period (years)	100	Cv (Summer)	0.750
Region	England and Wales	Cv (Winter)	0.840
M5-60 (mm)	19.000	Shortest Storm (mins)	15
Ratio R	0.360	Longest Storm (mins)	10080
Summer Storms	Yes	Climate Change %	+40

Time Area Diagram

Total Area (ha) 0.267

Time From:	**(mins) To:**	**Area (ha)**	**Time From:**	**(mins) To:**	**Area (ha)**	**Time From:**	**(mins) To:**	**Area (ha)**
0	4	0.089	4	8	0.089	8	12	0.089

Model Details

Storage is Online Cover Level (m) 26.800

Porous Car Park Structure

Infiltration Coefficient Base (m/hr)	0.00000	Width (m)	45.0
Membrane Percolation (mm/hr)	1000	Length (m)	75.0
Max Percolation (l/s)	937.5	Slope (1:X)	80.0
Safety Factor	2.0	Depression Storage (mm)	5
Porosity	0.30	Evaporation (mm/day)	3
Invert Level (m)	26.000	Membrane Depth (m)	1

Summary of Results for 100 year Return Period (+40%)

Half Drain Time exceeds 7 days.

Outflow is too low. Design is unsatisfactory.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Volume (m³)	Status
15 min Summer	26.295	0.295	0.0	6.3	O K
30 min Summer	26.353	0.353	0.0	9.0	O K
60 min Summer	26.407	0.407	0.0	11.9	O K
120 min Summer	26.458	0.458	0.0	15.1	O K
180 min Summer	26.486	0.486	0.0	17.0	O K
240 min Summer	26.504	0.504	0.0	18.3	Flood Risk
360 min Summer	26.530	0.530	0.0	20.2	Flood Risk
480 min Summer	26.549	0.549	0.0	21.7	Flood Risk
600 min Summer	26.564	0.564	0.0	22.9	Flood Risk
720 min Summer	26.576	0.576	0.0	23.9	Flood Risk
960 min Summer	26.594	0.594	0.0	25.4	Flood Risk
1440 min Summer	26.620	0.620	0.0	27.7	Flood Risk
2160 min Summer	26.645	0.645	0.0	29.9	Flood Risk
2880 min Summer	26.661	0.661	0.0	31.5	Flood Risk
4320 min Summer	26.682	0.682	0.0	33.5	Flood Risk
5760 min Summer	26.695	0.695	0.0	34.8	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Time-Peak (mins)
15 min Summer	125.869	0.0	27
30 min Summer	84.215	0.0	42
60 min Summer	53.779	0.0	72
120 min Summer	33.180	0.0	132
180 min Summer	24.659	0.0	192
240 min Summer	19.841	0.0	252
360 min Summer	14.545	0.0	372
480 min Summer	11.669	0.0	492
600 min Summer	9.828	0.0	612
720 min Summer	8.537	0.0	732
960 min Summer	6.830	0.0	972
1440 min Summer	4.978	0.0	1452
2160 min Summer	3.621	0.0	2172
2880 min Summer	2.885	0.0	2892
4320 min Summer	2.092	0.0	4332
5760 min Summer	1.663	0.0	5776

Summary of Results for 100 year Return Period (+40%)

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Volume (m³)	Status
7200 min Summer	26.703	0.703	0.0	35.6	Flood Risk
8640 min Summer	26.708	0.708	0.0	36.1	Flood Risk
10080 min Summer	26.712	0.712	0.0	36.5	Flood Risk
15 min Winter	26.317	0.317	0.0	7.2	O K
30 min Winter	26.377	0.377	0.0	10.3	O K
60 min Winter	26.434	0.434	0.0	13.6	O K
120 min Winter	26.488	0.488	0.0	17.1	O K
180 min Winter	26.517	0.517	0.0	19.3	Flood Risk
240 min Winter	26.537	0.537	0.0	20.7	Flood Risk
360 min Winter	26.564	0.564	0.0	22.9	Flood Risk
480 min Winter	26.584	0.584	0.0	24.6	Flood Risk
600 min Winter	26.600	0.600	0.0	25.9	Flood Risk
720 min Winter	26.612	0.612	0.0	27.0	Flood Risk
960 min Winter	26.632	0.632	0.0	28.8	Flood Risk
1440 min Winter	26.660	0.660	0.0	31.3	Flood Risk
2160 min Winter	26.686	0.686	0.0	33.9	Flood Risk
2880 min Winter	26.705	0.705	0.0	35.7	Flood Risk
4320 min Winter	26.728	0.728	0.0	38.2	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Time-Peak (mins)
7200 min Summer	1.391	0.0	7208
8640 min Summer	1.201	0.0	8648
10080 min Summer	1.063	0.0	10088
15 min Winter	125.869	0.0	27
30 min Winter	84.215	0.0	42
60 min Winter	53.779	0.0	72
120 min Winter	33.180	0.0	132
180 min Winter	24.659	0.0	192
240 min Winter	19.841	0.0	250
360 min Winter	14.545	0.0	370
480 min Winter	11.669	0.0	488
600 min Winter	9.828	0.0	608
720 min Winter	8.537	0.0	726
960 min Winter	6.830	0.0	964
1440 min Winter	4.978	0.0	1442
2160 min Winter	3.621	0.0	2152
2880 min Winter	2.885	0.0	2864
4320 min Winter	2.092	0.0	4284

Summary of Results for 100 year Return Period (+40%)

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Volume (m^3)	Status
5760 min Winter	26.743	0.743	0.0	39.7	Flood Risk
7200 min Winter	26.753	0.753	0.0	40.8	Flood Risk
8640 min Winter	26.760	0.760	0.0	41.5	Flood Risk
10080 min Winter	26.765	0.765	0.0	42.1	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m^3)	Time-Peak (mins)
5760 min Winter	1.663	0.0	5712
7200 min Winter	1.391	0.0	7136
8640 min Winter	1.201	0.0	8560
10080 min Winter	1.063	0.0	9896

Rainfall Details

Rainfall Model	FSR	Winter Storms	Yes
Return Period (years)	100	Cv (Summer)	0.750
Region	England and Wales	Cv (Winter)	0.840
M5-60 (mm)	19.000	Shortest Storm (mins)	15
Ratio R	0.360	Longest Storm (mins)	10080
Summer Storms	Yes	Climate Change %	+40

Time Area Diagram

Total Area (ha) 0.034

Time From:	**(mins) To:**	**Area (ha)**	**Time From:**	**(mins) To:**	**Area (ha)**	**Time From:**	**(mins) To:**	**Area (ha)**
0	4	0.012	4	8	0.011	8	12	0.011

Model Details

Storage is Online Cover Level (m) 26.800

Porous Car Park Structure

Infiltration Coefficient Base (m/hr)	0.00000	Width (m)	6.0
Membrane Percolation (mm/hr)	1000	Length (m)	58.0
Max Percolation (l/s)	96.7	Slope (1:X)	80.0
Safety Factor	2.0	Depression Storage (mm)	5
Porosity	0.30	Evaporation (mm/day)	3
Invert Level (m)	26.000	Membrane Depth (m)	1

APPENDIX V

SURFACE WATER ASSESSMNET INCLUDING FLOOD RISK ASSESSMENT

Calder Park

Tungsten Development. Surface Water Assessment.

Final Report

November 2020

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Revision History

Revision Ref / Date	Amendments	Issued to
13 November 2020	Draft Report	Mark Barwood
17 November 2020	Final Report	Mark Barwood

Contract

This report describes work commissioned by Mark Barwood, on behalf of Alan Wood and Partners, by email dated the 4 November 2020. Alan Wood and Partners' representative for the contract was Mark Barwood. Mark Bentley of JBA Consulting carried out this work.

Prepared by .... Mark Bentley BSc CEng CEnv FCIWEM C.WEM

Technical Director

Reviewed by .... Gavin Hodson BSc FdSc

Team Leader

Purpose

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JBA Consulting has no liability regarding the use of this report except to Alan Wood and Partners.

Acknowledgements

The help with arranging the provision of data by Mark Barwood is gratefully acknowledged.

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JBA is aiming to reduce its per capita carbon emissions.

Contents

Appendices

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Abbreviations

AEP	Annual Exceedance Probability
DEP	Donaldson Edwards Partnership
FEH	Flood Estimation Handbook
FRA	Flood Risk Assessment
HEC-RAS	Hydrologic Engineering Center – River Analysis System
JBA	Jeremy Benn Associates
mAOD	Metres Above Ordnance Datum
SSU	Scientific Support Unit
SuDS	Sustainable Drainage System
WYP	West Yorkshire Police

1 Introduction

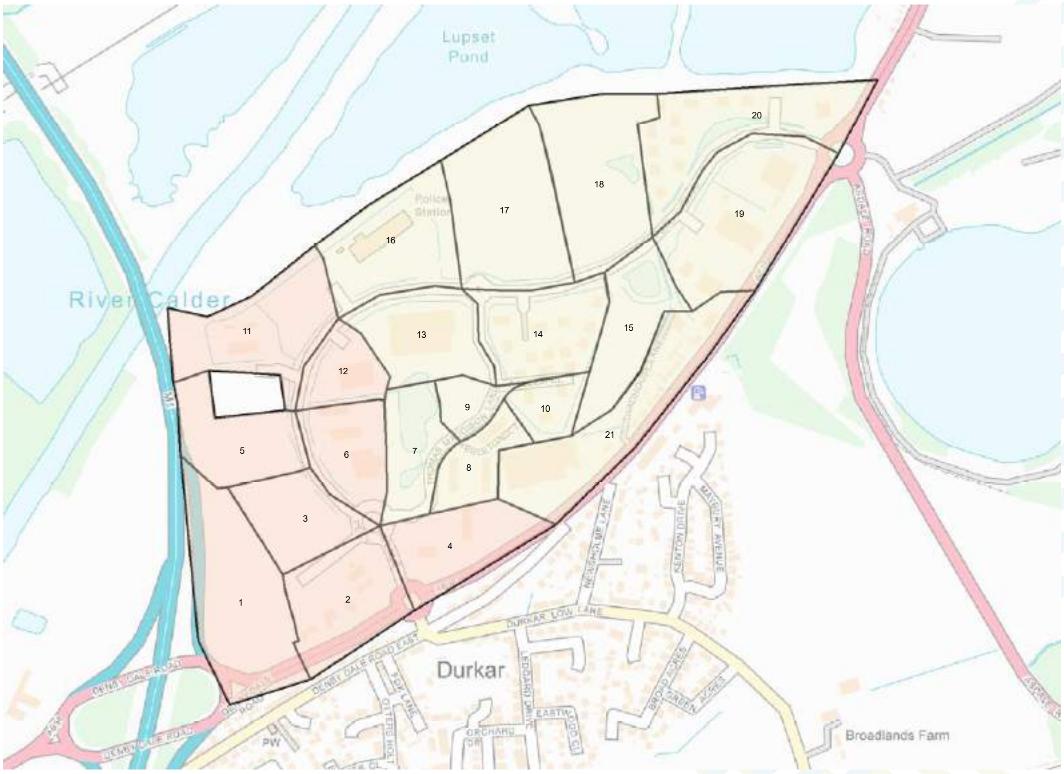
1.1 Flood risk assessment

The Calder Park development site is adjacent to Junction 39 of the M1. The site is bounded to the north and east by a flood embankment of the River Calder, to the west by the M1 motorway and to the south by Denby Dale Road. Between 2002 and 2018, JBA undertook work assessing the flood risk to the site and assisting with the design of the surface water drainage system. The surface water drainage system was designed to use a network of dry open channels (swales) and storage ponds to attenuate the runoff from the development plots.

There are proposals to develop a remaining plot of land at Calder Park. The proposed plot of land lies on the northern edge of Calder Park, just to the east of the Scientific Support Unit (SSU) building of West Yorkshire Police (WYP). The proposed platform level for the site will be set at 26.5mAOD. Appendix A contains drawings showing the proposed development. The total area of the plot of land for the new development is 60,700m^2 of which 51,950m^2 will comprise impermeable area. Therefore, 85% of the proposed will be impermeable which is greater than the value of 70% that was assumed for the design of the drainage system. This study has assessed the effect of increased runoff due to the greater impermeable area at the proposed site on water levels in the surface water drainage system.

JBA have re-used data previously collected on behalf of Peel Holdings, the developer of Calder Park. This includes topographical survey data showing ground levels and a computer model built for the business park surface water drainage system.

Figure 1-1 shows the Calder Park site. The proposed development will be in zones 17 and 18.



Not to scale

Figure 1-1: Calder Park location plan

1.2 Report structure

The report is presented in three sections:

1. **Introduction** – this section sets the study in context
2. **Surface Water Runoff** – flood risk from runoff within the Calder Park development
3. **Conclusions**

2 Surface water drainage

2.1 Surface water drainage system

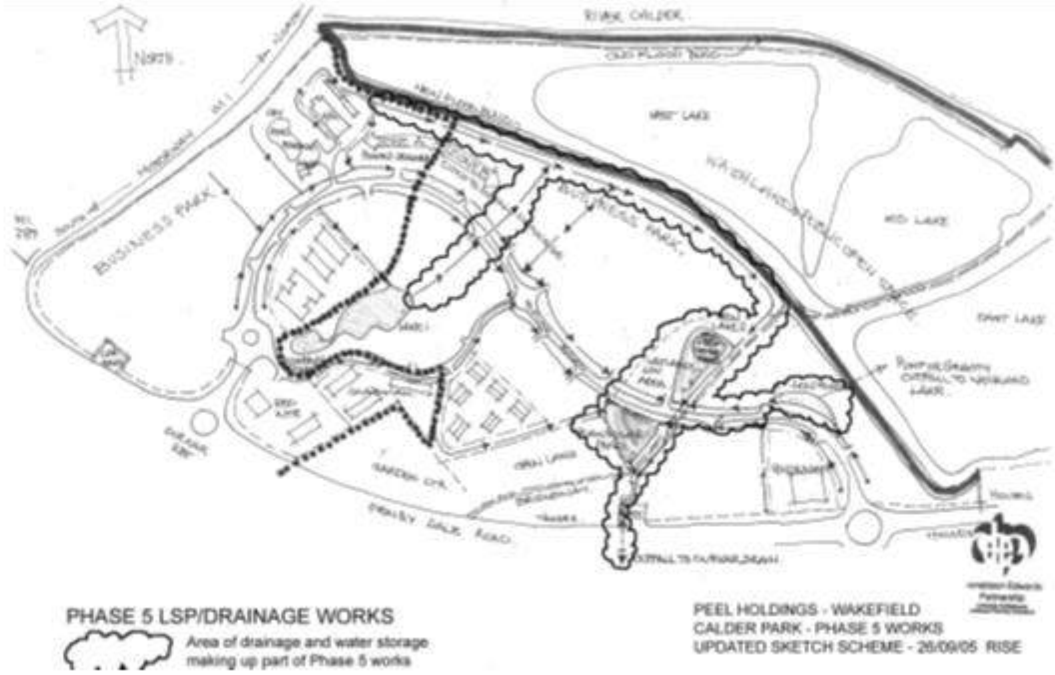
The drainage scheme for the Calder Park site utilises Sustainable Drainage System (SuDS) principles and is an integral part of the site design. The buildings, car parks and access roads are situated on raised platforms. A series of wide ditches (see Figure 2-1) run alongside the roads and between the building plots. There are also a few lakes that form part of the drainage scheme as well as improving the aesthetics of the site.

Figure 2-1: Typical swale at Calder Park

The site has been split into two surface water management zones, A and B (see Figure-2-2). Zone A utilises more traditional storm water drainage where runoff from roofs and roads is piped to a holding pond. This pond is then pumped directly to the River Calder at a rate of up to 1m^3/s. The system was designed such that there was sufficient storage in the pond to cope with runoff from a 100-year storm (including a 20% increase in rainfall intensity to allow for climate change).

Since the pond was designed the climate change allowance has been updated so that a 40% increase in rainfall intensity should be applied. Runoff calculations to the storage pond during a 1 in 100-year storm with an allowance of 40% increase in rainfall intensity show the pond level would rise to 26.0mAOD. That is 1m below the bank top (27.0mAOD). Therefore, the pond is large enough to deal with the revised allowances for climate change.

Runoff from all permeable areas and from subsequent plot developments (Zone B) goes directly to the ditches and is held in the land drainage system.



Note: Zone A is west of the dotted line and Zone B is to the east.

Figure 2-2: Calder Park drainage zones

The only outflow from the Calder Park site is to Durkar Drain. Water flows in a culvert (600mm diameter) under the Denby Dale Road. The ditch system at Calder Park connects to this culvert. The effective invert level of the culvert under the Denby Dale Road has been surveyed at 23.49mAOD. The lowest levels at Calder Park (in the north-east corner of the site) are lower than this. Therefore, it is not possible to drain the whole of Calder Park to Durkar Drain. A pumping station operates during high water levels to transfer water to the adjacent washland at a rate of 50l/s.

Surface water drainage for Calder Park has been calculated in three parts:

- Calculation of the volumes and rates of runoff during design events
- Hydraulic modelling of the ditch system to derive design water levels
- Calculations for the capacity of the culvert under Denby Dale Road.

2.2 Rainfall runoff calculations

Design rainfall depths for the site were calculated from the Flood Estimation Handbook (FEH). Two durations of storm were considered: 60-minutes and 7-hours for a 1%AEP (100-year return period) flood event.

Further consideration was given to the effects of climate change. The latest government guidance states that a 40% increase in rainfall should be used to account for climate change up to the year 2115 in the case of essential infrastructure .

A further storm profile comprising two consecutive 7-hour storms for a 3.3%AEP (30-year return period) was considered. This combination was used to assess the capacity of the surface water drainage system to recover from one storm before a further event occurs.

Table 2-1 summarises the design storms.

Table 2-1: Volume of storm rainfall

Storm Duration	3.3%AEP (30-year return period)			1%AEP (100-year return period)		
	Rainfall depth (mm)	Volume over site (m^3)	Peak storm intensity (mm/hr)	Rainfall depth (mm)	Volume over site (m^3)	Peak storm intensity (mm/hr)
60-minute				40	22,287	168
7-hour	90	50,298	18	62	34,234	24
with 40% allowance for climate change						
60-minute	Climate change scenario only applies to the 1% AEP (100-year flood event)			56	31,202	235
7-hour				87	47,928	34

For the purposes of calculating surface water runoff at Calder Park, the site was divided into 21 sub-catchments (see Figure 1-1). The sub-catchments represent different development plots across the Calder Park site. The calculations for each sub-catchment and the split between permeable and impermeable areas were adjusted based on plans supplied by WSP. For this study the percentage impermeable area of zones 17 and 18 was increased from 70% to 85%.

The division of the site enables the calculation of runoff volumes to be split between sites draining to the storage pond for pumping to the Calder and sites draining to the open ditches and eventual discharge to Durkar Drain. Storm water runoff volumes were calculated assuming that impermeable areas had a 100% runoff rate and 30% for permeable areas. Table 2-2 summarises volume of runoff for each design storm.

Table 2-2: Volume (m^3) of surface water runoff

Design scenario	Storm duration	Runoff destination	
		Pumped discharge	Durkar Drain
3.3%AEP	Two consecutive 7-hour storms	13,537	24,688
1%AEP (100-year return period)	1-hour	5,975	10,887
	7-hour	9,177	16,722
1% AEP with climate change	1-hour	8,365	15,242
	7-hour	12,848	23,411

Further runoff calculations were undertaken to assess the effect of saturated ground conditions which assumed the runoff from permeable areas was also 100%. Table 2.3 summarises the volume of runoff for this situation.

Table 2-3: Volume (m^3) of surface water runoff

Design scenario	Storm duration	Runoff destination	
		Pumped discharge	Durkar Drain
3.3%AEP	Two consecutive 7-hour storms	13,537	36,962
1%AEP (100-year return period)	1-hour	5,975	16,313
	7-hour	9,177	25,057
1% AEP with climate change	1-hour	8,365	22,838
	7-hour	12,848	35,080

There is a further discharge to the drainage system from the SSU building of WYP. The building is heated/cooled using an open loop groundwater heat extraction system. Depending on the energy demand this may result in peak flows of 18l/s to the ditch system. Over a 60-minute storm this would result in, at most, an extra 65m^3 of flow. Over a seven-hour storm the maximum runoff would be 454m^3 assuming the system operates at peak capacity throughout the storm.

Although the discharge from the SSU is small it is continuous and so there is the potential for the flow to reduce the capacity of the drainage ditches prior to a storm commencing. Therefore, the simulations of the drainage system have assumed the ditches are partially full due to continuous flow from the groundwater heat extraction system.

2.3 Hydraulic modelling

2.3.1 HEC-RAS model

Modelling of the drainage ditches was undertaken using HEC-RAS. HEC-RAS is an unsteady state one-dimensional river modelling package developed by the Hydrologic Engineering Center of the US Army Corps of Engineers. The software allows the user to calculate the variation of water surface within a channel network during a storm. The system can handle a looped network of channels, a branched system or just a single river reach. The model also allows the simulation of the effect of flood storage on routing flood flows. The model runs quickly and so was able to represent the draining down of the drainage system following an event.

The HEC-RAS model was originally set up with ditch profiles based on information supplied by Peel Holdings. The ditches are generally constructed by making a 1-in-3 slope down from the plot level to the proposed ditch invert level, cutting a flat bed 1.5m wide at this level and then making a 1 in 3 slope up to meet the opposite bank height. The actual ditch widths therefore depend on the adjacent ground heights and the ditch invert level.

Figure 2-3 shows the model for the completed ditch system.

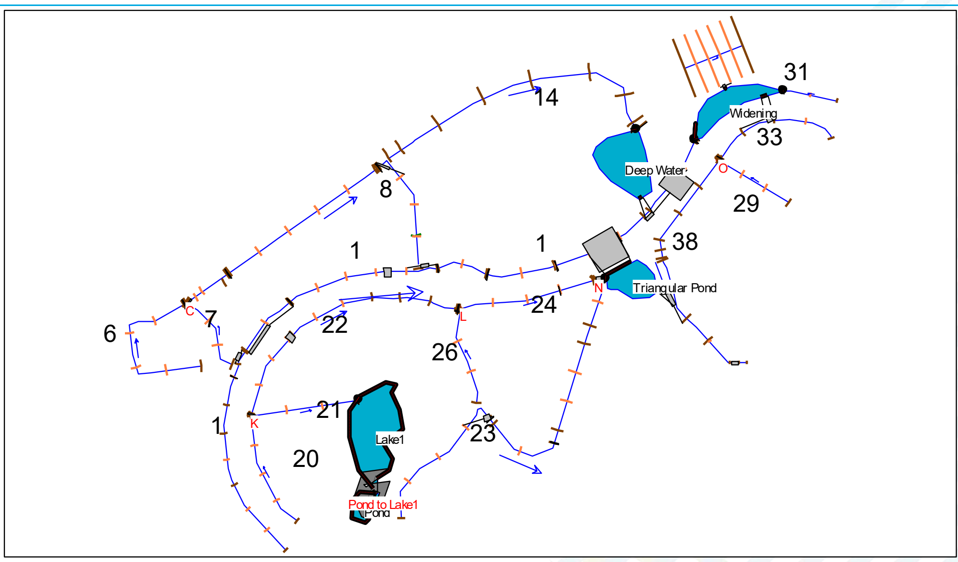


Figure 2-3: HEC-RAS schematic of completed drainage system

A combination of lateral weirs (sometimes incorporating culverts) and open channel junctions were used for connections between ditches and other ditches or lakes. To control water levels, retain the design standing water level and aid model stability (to prevent reaches becoming dry), a number of weirs were defined at intervals along the ditches. The weirs in the model were only defined where required to retain standing water levels.

Manning's *n* values (representing the roughness of the drains) were set to 0.03 at all locations. Since the drains are essentially a storage system (with flow controlled by weirs) the model results were expected to be insensitive to Manning's n values. However, sensitivity tests for this were undertaken (see Section 2.4.2). The model was run with a timestep of 6 seconds.

2.3.2 Standing water levels

At the start of a model simulation it is necessary to define the initial water levels in the ditches. In the case of the drainage system at Calder Park this is especially important because it will define the excess flood storage capacity in the system. The level of water in the ditches will depend on four factors:

- Drainage system design
- Groundwater levels
- Discharges from the SSU groundwater heat extraction system
- Recent storms, which have not fully drained away

To allow for high groundwater levels (see Chapter 4), the ditches have been designed to have a standing water depth of 0.5m with 1.5m depth in the lakes. Only ditches to the east of the Calder Park site would have deeper water (0.65m).

Initial conditions for the model simulations have been set assuming the groundwater heat extraction system is continuously discharging. This means that storage in the ditches at the start of the simulation has been partially used up before the storm begins.

Model simulations were also undertaken to assess the effect of two consecutive storms occurring. These tested the effect of high standing water levels that had not been able to drain out of the system before the onset of another storm. In this situation the standing water levels at the start of the storm are defined by the model following the simulation of runoff from the first storm.

2.4 Model results

The completed HEC-RAS model was run for the following three storms (all using realistic Runoff calculations – see Table 2-1), allowing outflow to Durkar Drain:

- 100-year, 60-minute Summer Storm
- 100-year, 7-hour Winter Storm
- Two consecutive 30-year, 7-hour Winter Storms

The tail-water level of the Denby Dale Road culvert was set at 25.2mAOD for the period of all the simulations. The flood level of 25.2mAOD was based on advice from Wakefield Council and represents the observed flood level on Denby Dale Road during the flood of the 25 June 2007. This is equivalent to a surcharged condition for the culvert and represents the case of impeded discharge from Durkar Drain to the River Calder (see section 2.5). Under these conditions there is backflow from Durkar Drain into the Calder Park site.

2.4.1 Water levels

The model simulations showed that generally across the site the two consecutive storms had the most impact on water levels in the drainage system. This is because high water levels at Denby Dale Road (25.2mAOD) prevent runoff escaping to Durkar Drain.

Therefore, water levels in the drainage system are sensitive to the runoff volume as opposed to the rainfall intensity.

This conclusion is different to that found during the original FRA in 2008. That study assumed a lower flood level at Denby Dale Road of 25.0mAOD and allowed water to leave Calder Park between consecutive storms.

Table 2-4 shows the highest water level in each ditch from each of the three simulations based on the final ditch layout with development completed. The ditch reach labels can be found on Figure 2-3. Simulated flood levels remained in bank throughout the ditch system.

Table 2-4: Maximum water levels in ditches - Completed drainage system

Upper site		Lower site	
Ditch	Maximum water level (mAOD)	Ditch	Maximum water level (mAOD)
1	26.28	23	25.76
6	25.28	24	25.62
7	25.29	26	25.62
8	25.33	29	25.22
14	25.31	31	25.22
20	26.18	33	25.22
21	26.18	38	25.22
22	26.18		

The effect of increased runoff from the site of the proposed development due to a greater impermeable area has a marginal impact on water levels. The only ditches where an impact was seen were ditches 29 and 38 where the water level increased by just 0.01m.

The proposed plot for the Tungsten Development will be set to 26.5mAOD. The maximum water level in the pond Deep Water and the ditches surrounding the development does not rise above 25.33mAOD. Therefore, there is over 1m freeboard.

2.4.2 Increased runoff (saturated ground conditions)

A sensitivity test with increased runoff to account for saturated ground conditions was undertaken. Runoff from the permeable areas of the site was assumed to be 100% (see section 2.2) and the SSU open loop groundwater heat extraction system was flowing at its maximum rate of 18l/s. This increased the total volume of runoff from the site and raised water levels in the ditches.

Table 2-5 shows the maximum water levels in the ditches for the completed system.

Table 2-5: Maximum water levels in ditches – 100% runoff

Upper site		Lower site	
Ditch	Maximum water level (mAOD)	Ditch	Maximum water level (mAOD)
1	26.60	23	25.83
6	25.47	24	25.83
7	25.58	26	25.83
8	25.47	29	25.24
14	25.36	31	25.24
20	26.23	33	25.24
21	26.23	38	25.24
22	26.23		

Generally, increasing the runoff has raised water levels across the site by the order of a few centimetres. The only significant impact on water level were in Ditches 1, 6, 7, 8, 24 and 26 where levels were raised by 0.32m (ditch 1), 0.29m (ditch 7), 0.14m (ditch 8), 0.21m (ditches 24 and 26). Even so, with saturated runoff, peak water levels were contained within bank.

Even with saturated runoff, the freeboard for the Tungsten Development in the pond Deep Water and the ditches surrounding the development is maintained at over 1m.

2.4.3 Climate change

A sensitivity test for the effects of climate change was also undertaken by applying a 40% increase in the volume of runoff. Table 2-6 shows the peak water levels in each ditch for the climate change scenario.

Table 2-6: Maximum water levels in ditches – Climate change

Upper site		Lower site	
Ditch	Maximum water level (mAOD)	Ditch	Maximum water level (mAOD)
1	26.36	23	25.81
6	25.33	24	25.78
7	25.35	26	25.78
8	25.41	29	25.24
14	25.39	31	25.24
20	26.23	33	25.24
21	26.23	38	25.24
22	26.23		

The impact of climate change was generally less than that due to saturated ground conditions and water levels were retained within the ditch system. For the climate change scenario, the freeboard for the Tungsten Development in the pond Deep Water and the ditches surrounding the development is maintained at over 1m.

2.4.4 Channel Roughness

The sensitivity of the model results to increased channel roughness was assessed by doubling the Manning's *n* coefficient from 0.03 to 0.06 across the site. There was very little sensitivity to Manning's *n* in the lower part of the site which is storage-dominated (only ditch 23 in this area showing an increase in water levels of 0.07m). A more widespread sensitivity was seen in the upper site where flux is more important, but most increases were less than 0.03m.

2.5 Durkar Drain

The outflow from Calder Park to Durkar Drain passes in culvert under the Denby Dale Road, then through a short section of open channel near a foul water pumping station and then into another culvert.

As part of the Wakefield Flood Defence Strategy developed by the EA, the outflow of Durkar Drain to the River Calder is flapped. It is expected that the flap will close for events greater than the 1-in-2-year flood on the River Calder. A further flap valve at the area known as 'Pugneys Entrance' closes during a 1-in-40-year flood event on the River Calder. This results in the Durkar Drain being 'tidelocked'. Durkar Drain is an ordinary watercourse and so is the responsibility of Wakefield Council.

Wakefield Council has purchased mobile pumps that can be deployed across their council area. Therefore, emergency pumps could be deployed to pump excess water from Durkar Drain to the River Calder during flood events. However, consultations with Wakefield Council indicate that this is not certain.

Therefore, the design runs for to test the surface water drainage system at Calder Park have made a conservative assumption that emergency pumps are not deployed to Durkar Drain. In this case water levels in Durkar Drain will backup and so a downstream level of 25.2mAOD has been applied. Under this scenario there will be backflow to the Calder Park site. In normal circumstances this situation should be avoided because of the risk of polluted water stagnating within the system of swales at Calder Park.

If emergency pumps are deployed, they should lower levels in Durkar Drain provided it is kept clear downstream of Denby Dale Road. With these lower levels (24.5mAOD) there would be no backflow on to the Calder Park site. In this case the drainage system would only be required to store runoff from Calder Park.

2.6 Culvert capacity

The culvert under the Denby Dale Road is a 600m diameter pipe. The invert level of the culvert is approximately 23.27mAOD. This information contrasts with a spot level survey (Donaldson Edwards Partnership (DEP), June 2005) which indicated that the invert at the culvert entrance is 23.49mAOD. This apparent discrepancy is probably caused by the large amount of siltation at the culvert. It is likely that the DEP level is the effective invert level due to siltation of the bed (i.e. 0.22m of silt blocking the bottom of the culvert). At the time WSP stated it was not possible to determine the invert of the pipe through the silt. The WSP information (from heights relative to footpath levels) indicated a downstream invert level of 23.22mAOD. The road is approximately 30m wide, which would give the culvert a slope of 1 in 500.

Surface runoff from Calder Park will not be restricted. Modelling (with a clear culvert) indicates that unrestricted flow leaving the site will exceed the agricultural runoff rates. This should not cause problems for Durkar Drain because in general rainfall on the site will enter Durkar Drain before the River Calder rises, 'tidelocking' the drain.

When the Durkar Drain is 'tidelocked', emergency pumps may be deployed by Wakefield Council to pump some of the flow that could be expected in Durkar Drain. If the culvert were totally blocked water levels could rise to 25.2mAOD, the boundary condition used to test the performance of the surface water drainage system at Calder Park (see 2.4).

3 Conclusions

3.1 Surface water drainage

The drainage scheme for Calder Park uses SuDS principles as an integral part of the site design. A series of wide ditches run alongside the roads and between the development platforms. There are also a number of lakes that form part of the drainage scheme as well as providing an aesthetic attraction.

Generally peak levels in the drainage system across the site were caused by consecutive storm events. This is because the model tests used high water levels at Denby Dale Road (25.2mAOD) that prevent runoff escaping to Durkar Drain. Therefore, water levels in the drainage system are sensitive to the runoff volume as opposed to the rainfall intensity.

Model simulations of the surface water drainage system have been undertaken for the proposed completed drainage system with a full developed business park. The simulations tested the impact of increased runoff from the proposed development on water levels in the surface water drainage system.

The effect of increased runoff from the site of the proposed development due to a greater impermeable area was shown to have a marginal impact on water levels. The maximum increase in water level of 0.01m was confined to just two ditches (number 29 and 38). However, simulated flood levels remained in bank throughout the ditch system.

With respect to the proposed Tungsten Development, flood levels in the Deep Water pond and surrounding ditches/ swales are of sufficient volume to prevent flooding of the Property. Under a 1 in 100-year event with an extra 40% allowance for climate change these is a freeboard of over 1m in the drainage system surrounding the site. Therefore, the surface water network can accommodate the free discharge of runoff from the Property based on an impermeable area of 51,950m^2.

3.2 Durkar Drain

The discharge of surface water from Calder Park is to Durkar Drain via a culvert under Denby Dale Road. As part of the Wakefield Flood Defence Strategy developed by the EA, the outflow of Durkar Drain to the River Calder flows through two flapped structures. As a results, during a 1-in-40-year flood event on the River Calder, Durkar Drain is 'tidelocked'. Durkar Drain is an ordinary watercourse and so is the responsibility of Wakefield Council.

Wakefield Council has purchased mobile pumps that can be deployed across their council area. Therefore, emergency pumps could be deployed to pump excess water from Durkar Drain to the River Calder during flood events. However, consultations with Wakefield Council indicate that this is not certain.

Therefore, design runs for this FRA have made a conservative assumption that emergency pumps are not deployed to Durkar Drain. In this case water levels in Durkar Drain will backup and so a downstream level of 25.2mAOD has been applied. Under this scenario there will be backflow to the Calder Park site. In normal circumstances this situation should be avoided because of the risk of polluted water stagnating within the system of swales at Calder Park.

If emergency pumps are deployed, they should lower levels in Durkar Drain provided it is kept clear downstream of Denby Dale Road. With these lower levels (24.5mAOD) there would be no backflow on to the Calder Park site. In this case the drainage system would only be required to store runoff from Calder Park.

Appendix
A Proposed development at Calder Park

DO NOT SCALE!
ALL DIMENSIONS SHOULD BE CHECKED ON SITE BEFORE WORK COMMENCES

BOUNDARY SUBJECT TO TITLE PLAN CHECK

SUBJECT TO REVIEW OF LEVELS AND FALLS

Rev.	Date	Description	Drawn
C	29.09.20	Office area amended to include SF. Areas updated to include gatehouse / control tower. Parking numbers added. Note for attenuation tank added.	DW
B	05.08.20	Sprinkler tanks moved	DW
A	04.08.20	Boundary amended inline with information from solicitors - drawing Calder Park - Ordnance Survey Mastermap. Site plan amended accordingly and detailing updated.	DW

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client
Tungsten

project
Calder Park, Wakefield

drawing title
Proposed Site Plan

date August 2020
status Feasibility
scale 1:500 @ A0
drawn DW checked
job no. 2479 dwg no. F404 rev. C

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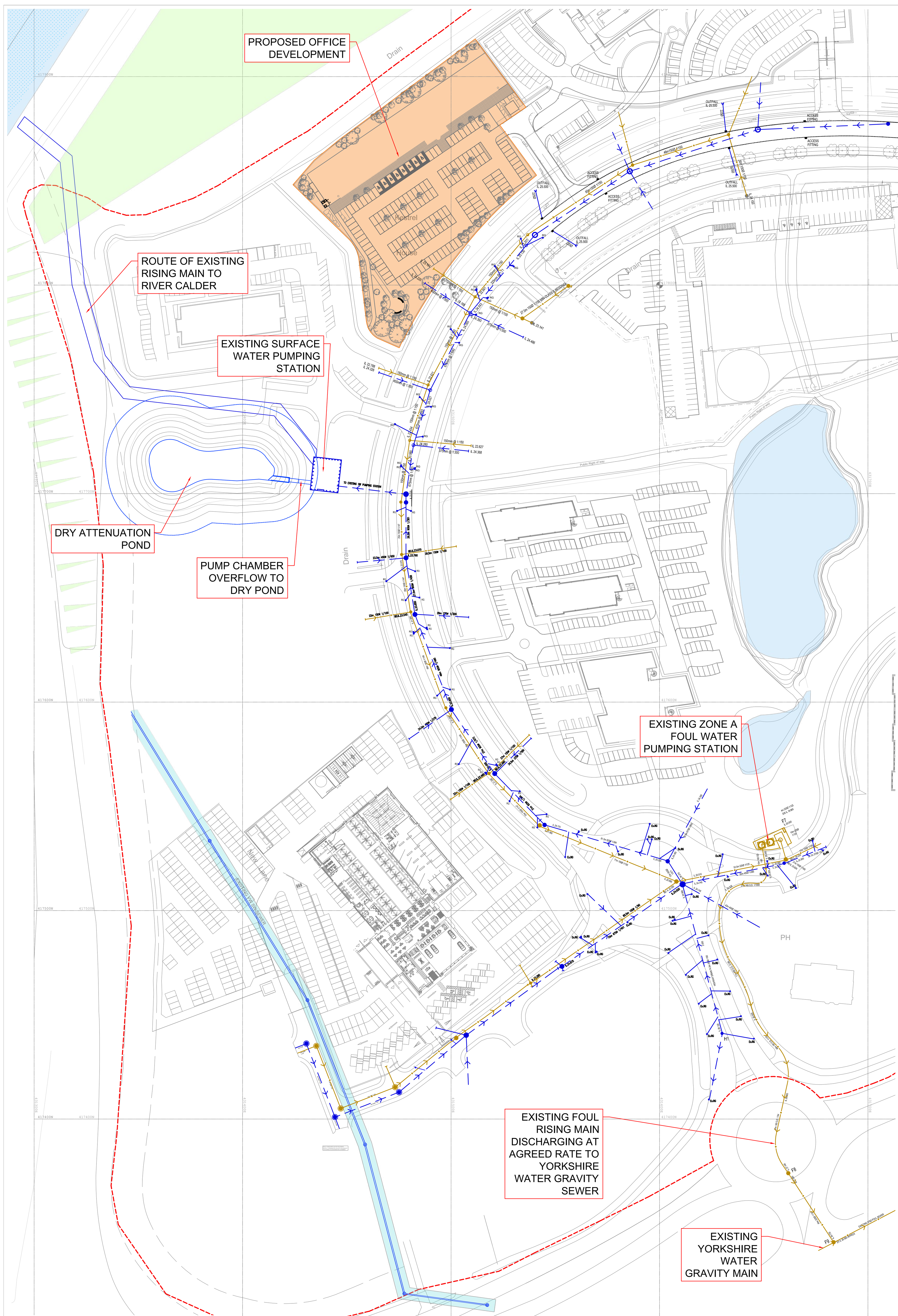
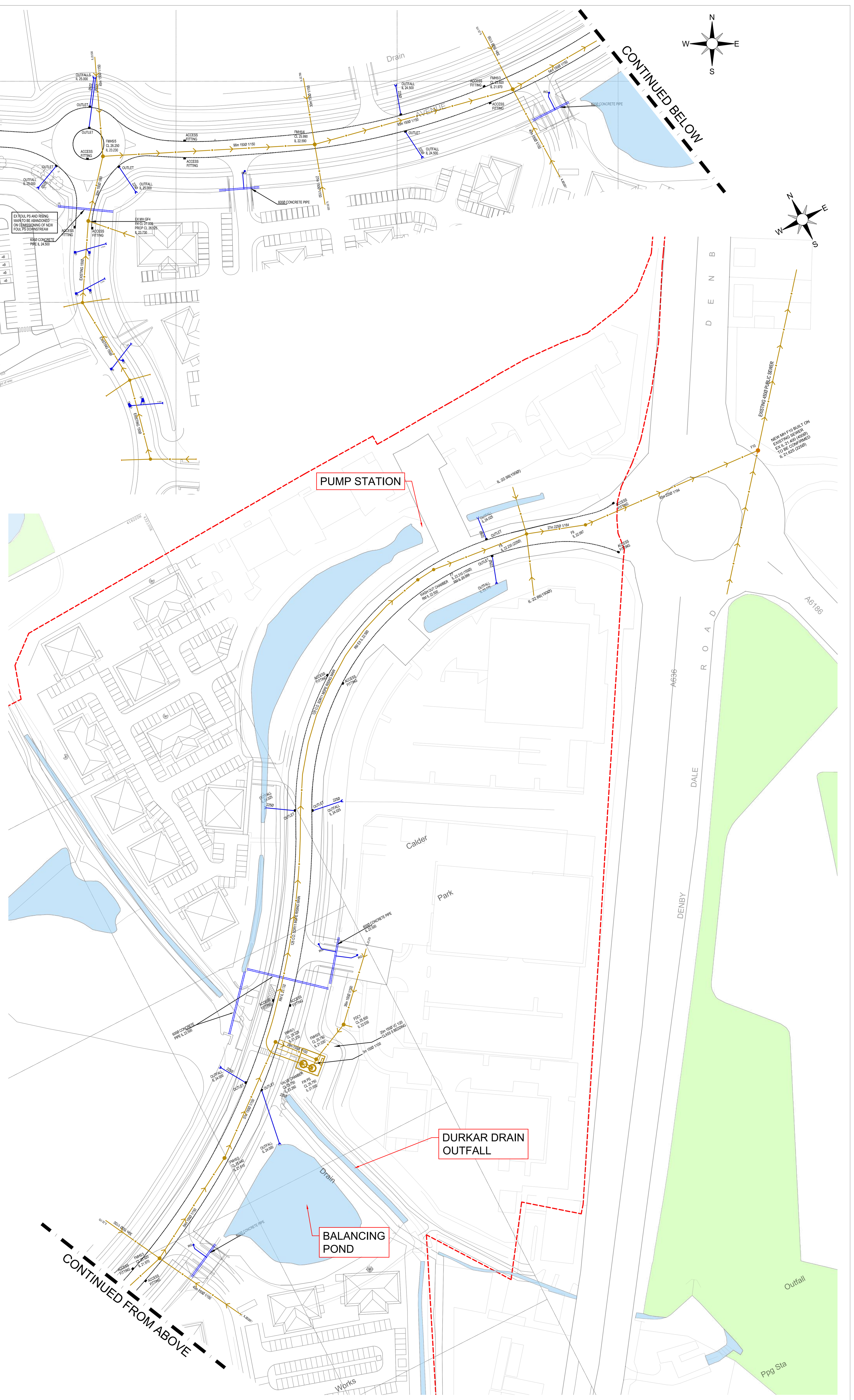
APPENDIX VI

EXISTING FOUL CONNECTION DRAWING



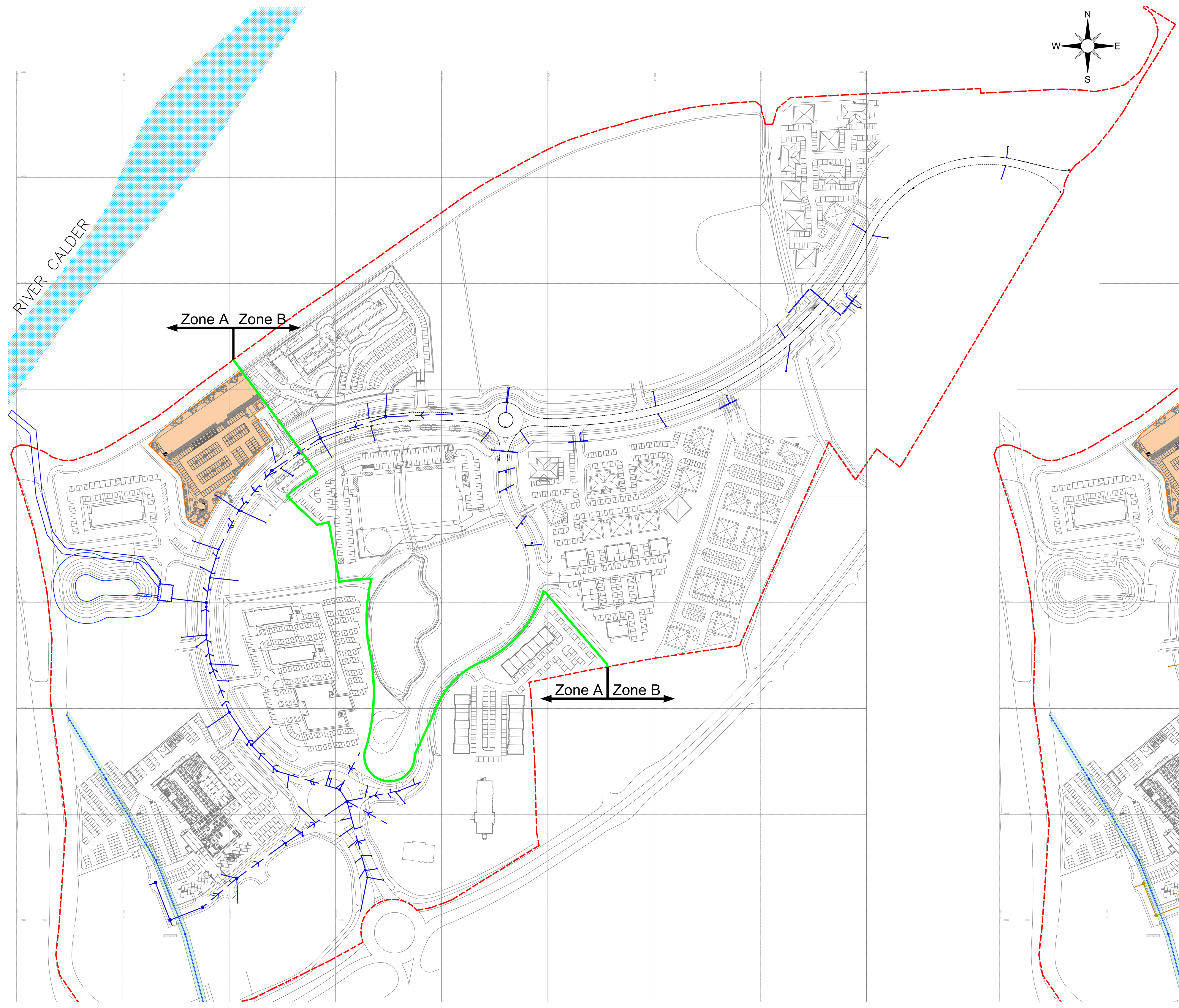
Drainage Layout - West

Scale 1:1000



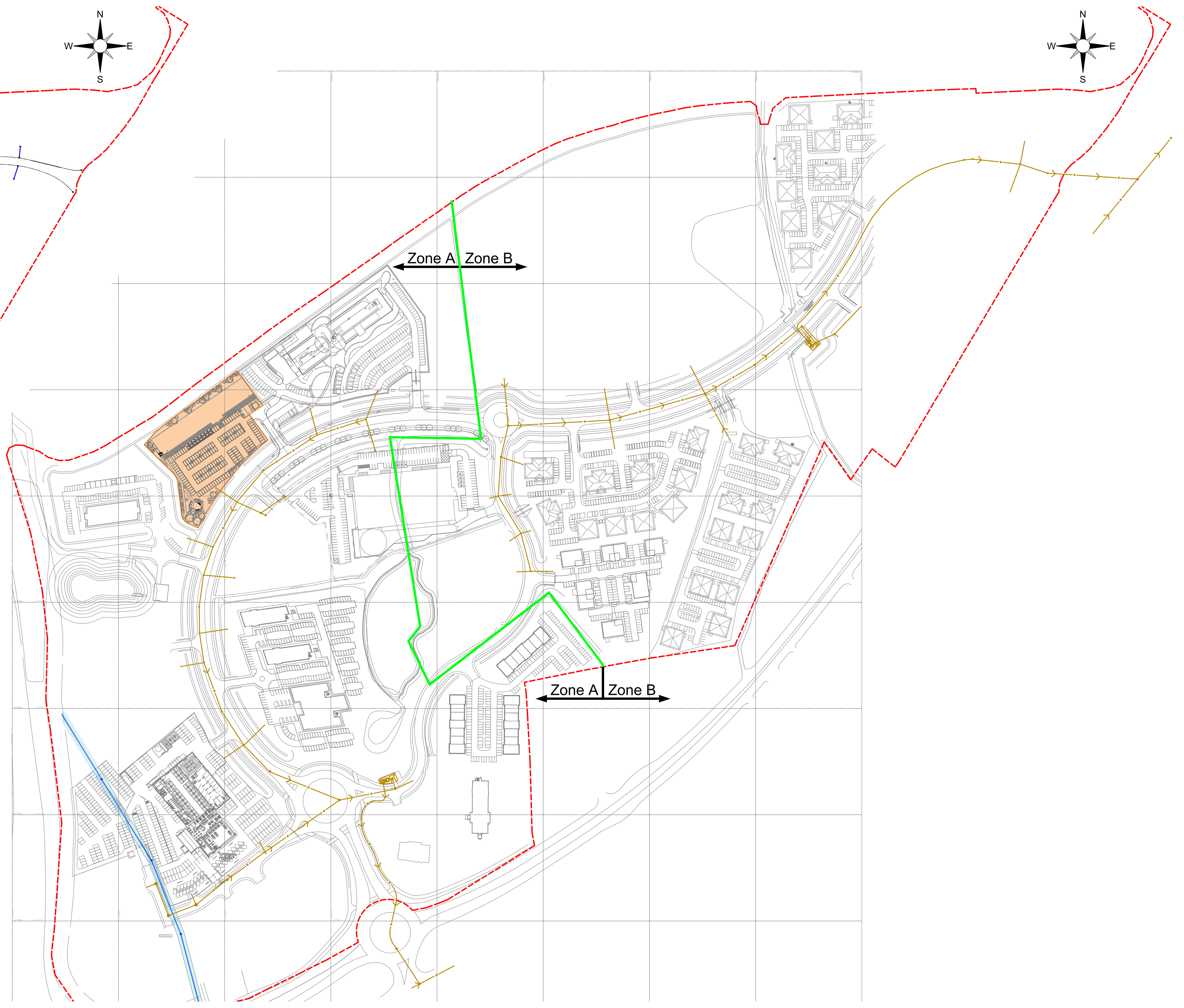
Drainage Layout Continued - East

Scale 1:1000



Indicative surface water boundary between Zone A & Zone B

Scale 1:2500



Indicative foul water boundary between Zone A & Zone B

Scale 1:2500

NOTES:

1. THESE NOTES ARE INTENDED TO AUGMENT DRAWINGS AND SPECIFICATIONS. WHERE CONFLICT OF REQUIREMENTS EXIST THE ORDER OF PRECEDENCE SHALL BE AS SHOWN IN THE SPECIFICATION. OTHERWISE THE STRICTEST PROVISION SHALL GOVERN.
2. THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ALL OTHER RELEVANT ENGINEERS AND ARCHITECTS DRAWINGS.
3. DRAWINGS NOT TO BE SCALED. ALL DIMENSIONS TO BE CHECKED ON SITE BY THE CONTRACTOR. ANY DISCREPANCIES TO BE NOTIFIED TO THE ENGINEER AND FURTHER INSTRUCTIONS OBTAINED BEFORE WORK IS COMMENCED.
4. THE STRUCTURE IS DESIGNED TO BE SELF-SUPPORTING AND STABLE AFTER THE BUILDING IS FULLY COMPLETED. IT IS THE CONTRACTORS SOLE RESPONSIBILITY TO DETERMINE THE ERECTION PROCEDURE AND SEQUENCE AND ENSURE THAT THE BUILDING AND ITS COMPONENTS ARE SAFE DURING ERECTION. THIS INCLUDES THE ADDITION OF WHATEVER TEMPORARY BRACING, GUYS OR TIE-DOWNS WHICH MAY BE NECESSARY, SUCH MATERIAL REMAINING THE THE PROPERTY OF THE CONTRACTOR ON COMPLETION, AND FOR ENSURING THAT THE WORKS AND ANY ADJACENT PROPERTIES ARE SAFE IN THE TEMPORARY CONDITION.

100mm at A0

Legend

— ← —	Existing SW
—•—←	Existing FW
━━	Zone A / Zone B Drainage Boundary

Rev	Description	Date	By	Chk	App
P02	EASTERN AREA ADDED	13.12.19	NB	MB	MB
P01	FIRST ISSUE	13.11.19	NB	MB	MB

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Project: Calder Park - Zone 1 Earth Works / Landrover Enabling Works, Wakefield

Client: Peel Investments (North) Ltd

Drawing: Overall Calder Park Drainage GA Layout (Surface & Foul Water)

Role: Civil

Drawing Status: PRELIMINARY

Job. no. 42499 | Scale@ A0: As Noted | Rev. P02

Project	Originator	Volume	Level	Type	Role	Number
42499	AWP	XX	XX	DR	C	2150

42499 - AWP - XX - XX - DR - C - 2150

APPENDIX VII

MAINTENANCE AND MANAGEMENT PLAN

MAINTENANCE AND ACTION SCHEDULE
FOR
SURFACE WATER DRAINAGE

at

Peel Avenue, Calder Park, Wakefield

Project No:	**63150**	Prepared by:	PRP Catherine House
For:	Peel Avenue, Calder Park		Old Harboorough Road Brixworth
Date:	November 2020		Northampton NN6 9BX

1. Manholes and inspection chambers should be regularly inspected and debris/silt removed, if this is not removed then it is likely to become hard packed requiring considerable effort to remove it.

2. The following are guidelines for when inspections and treatment should be carried out based on a typical private surface water drainage system and foul water drainage system. The rate of silt and debris accumulation should be monitored and the frequency of inspection may need to be adjusted based on this.

   2.1. Monthly:

   - Hard surfacing should be swept regularly to prevent silt being washed into the surface. This will minimise necessary maintenance of the permeable surface. (every two months during Spring and Summer)
   - Approved herbicide to be applied to permeable surface to prevent foliage growing and blocking voids. (every two months during Spring and Summer)
   - Lift all final outflow manhole cover and inspect to make sure that the outfall and inlet are clear. (every two months during Spring and Summer)
   - Remove litter (including leaf litter) and debris from surface of permeable paving and access chambers.

   2.2. Annually:

   - Inspect all gutters and gullies for sediment and debris and remove as necessary to prevent it from entering into the attenuation tank.
   - Any roots that have entered the system should be removed.
   - Inspect manholes and remove any silt or debris from base and ensure that they are clean.
   - Any residual weeds to be removed manually.

   2.3. As required:

   - Jet all pipework: perforated and solid wall.
   - Power wash permeable surface and relay with new permeable stone tank beneath, if required.

Maintenance Schedule	Required Action	Typical Frequency
Regular Maintenance	Inspect all gutters and gullies for sediment and debris and remove as necessary to prevent it from entering into the system	Annually
	Any roots that have entered the system should be removed	Annually
	Roads and all car parking spaces should be swept regularly to prevent silt being washed into the surface. This will minimise maintenance	Monthly during the Autumn and Winter but to be assessed on the site
Occasional Maintenance	Inspect manholes and other chamber and remove any silt or debris from base and ensure that they are clean	Annually
Remedial Actions	Reconstruct permeable paving and/or replace if performance deteriorates of failure occurs	As required
Monitoring	Inspect silt traps and note rate of sediment accumulation	Monthly in the first year and then annually
	Check outfalls to ensure all is working correctly	Annually

Catchpits, manholes and inspection chambers should be regularly inspected and debris/silt removed, if this is not removed then it is likely to become hard packed requiring considerable effort to remove it. Replacement of the coarse granular aggregate and the membrane will be necessary if the system becomes blocked with silt. Effective monitoring will give information on changes in infiltration and provide a warning of potential failure in the long term.

APPENDIX VIII

GENERAL CONDITIONS

1. This report has been prepared and written specifically for the Client named in the introduction and is exclusively for their benefit. No reliance may be placed in the contents of this report by any third party except with the express agreement of the original Client and the written agreement of PRP. Such written agreement may require the payment of an additional fee.

2. The ‘Recommendations’ section of this report only provide an overview of the guidance and should not be specifically relied upon in their own right but should be considered in relation to the whole report and the development described in this report.

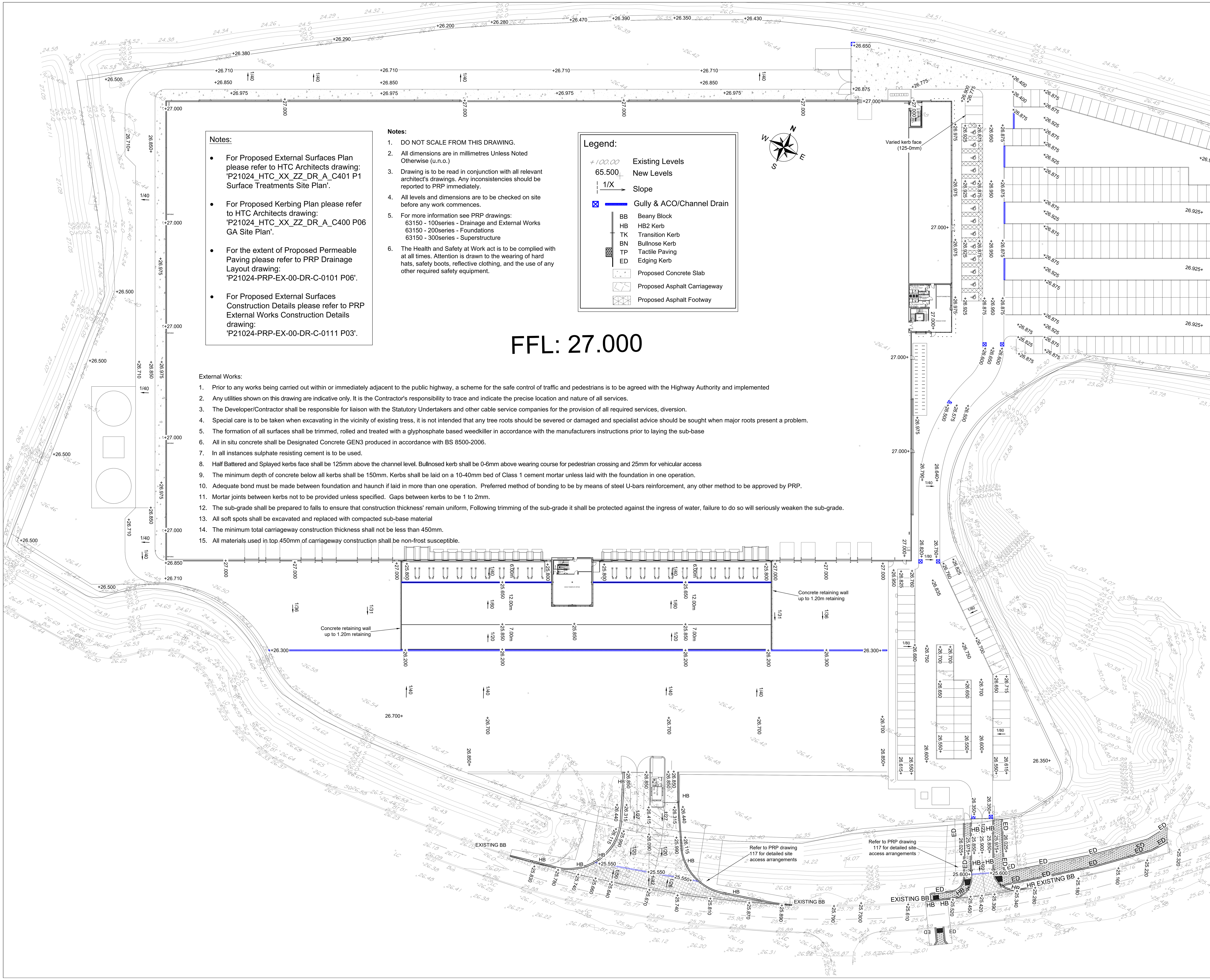
3. This report has been prepared and written in the context of the proposals for the development of the site as stated by the Client and will not be valid in a differing context. Furthermore, new information, improved practices, or legislation may necessitate alterations to the report in whole or in part after its submission. Therefore, with any change in circumstances or after the expiry of one year from the date of this report, it should be referred to us for re-assessment.

4. Any assessments made in this report are based on the ground conditions and as revealed by the test pits and boreholes undertaken by others and where appropriate other relevant data which may have been obtained for the site. The sources of such information are detailed in this report and while PRP use only such sources as are believed to be reliable, PRP will not be liable for the authenticity or reliability of information obtained from others.

5. Where data has been provided or is made available to PRP and this has been used in the report, it has been assumed that the information is correct. No responsibility can be accepted by PRP for inaccuracies within the data supplied.

6. The copyright in this report and other related plans and documents prepared by PRP is owned by them and no such report, plan or document may be reproduced, published or adapted without their written consent. Complete copies of the report may however be made and distributed by the Client as an expedient in dealing with matters related to its commission.

7. This report has been prepared solely for the Client’s purposes in obtaining planning permission related to the proposed development indicated in the report.



Notes:

- For Proposed External Surfaces Plan please refer to HTC Architects drawing: 'P21024_HTC_XX_ZZ_DR_A_C401 P1 Surface Treatments Site Plan'.
- For Proposed Kerbing Plan please refer to HTC Architects drawing: 'P21024_HTC_XX_ZZ_DR_A_C400 P06 GA Site Plan'.
- For the extent of Proposed Permeable Paving please refer to PRP Drainage Layout drawing: 'P21024-PRP-EX-00-DR-C-0101 P06'.
- For Proposed External Surfaces Construction Details please refer to PRP External Works Construction Details drawing: 'P21024-PRP-EX-00-DR-C-0111 P03'.

Notes:

1. DO NOT SCALE FROM THIS DRAWING.
2. All dimensions are in millimetres Unless Noted Otherwise (u.n.o.)
3. Drawing is to be read in conjunction with all relevant architect's drawings. Any inconsistencies should be reported to PRP immediately.
4. All levels and dimensions are to be checked on site before any work commences.
5. For more information see PRP drawings:
   63150 - 100series - Drainage and External Works
   63150 - 200series - Foundations
   63150 - 300series - Superstructure
6. The Health and Safety at Work act is to be complied with at all times. Attention is drawn to the wearing of hard hats, safety boots, reflective clothing, and the use of any other required safety equipment.

Legend:

+100.00	Existing Levels
65.500	New Levels
1/X	Slope
	Gully & ACO/Channel Drain
BB	Beany Block
HB	HB2 Kerb
TK	Transition Kerb
BN	Bullnose Kerb
TP	Tactile Paving
ED	Edging Kerb
	Proposed Concrete Slab
	Proposed Asphalt Carriageway
	Proposed Asphalt Footway

FFL: 27.000

External Works:

1. Prior to any works being carried out within or immediately adjacent to the public highway, a scheme for the safe control of traffic and pedestrians is to be agreed with the Highway Authority and implemented
2. Any utilities shown on this drawing are indicative only. It is the Contractor's responsibility to trace and indicate the precise location and nature of all services.
3. The Developer/Contractor shall be responsible for liaison with the Statutory Undertakers and other cable service companies for the provision of all required services, diversion.
4. Special care is to be taken when excavating in the vicinity of existing tress, it is not intended that any tree roots should be severed or damaged and specialist advice should be sought when major roots present a problem.
5. The formation of all surfaces shall be trimmed, rolled and treated with a glyphosphate based weedkiller in accordance with the manufacturers instructions prior to laying the sub-base
6. All in situ concrete shall be Designated Concrete GEN3 produced in accordance with BS 8500-2006.
7. In all instances sulphate resisting cement is to be used.
8. Half Battered and Splayed kerbs face shall be 125mm above the channel level. Bullnosed kerb shall be 0-6mm above wearing course for pedestrian crossing and 25mm for vehicular access
9. The minimum depth of concrete below all kerbs shall be 150mm. Kerbs shall be laid on a 10-40mm bed of Class 1 cement mortar unless laid with the foundation in one operation.
10. Adequate bond must be made between foundation and haunch if laid in more than one operation. Preferred method of bonding to be by means of steel U-bars reinforcement, any other method to be approved by PRP.
11. Mortar joints between kerbs not to be provided unless specified. Gaps between kerbs to be 1 to 2mm.
12. The sub-grade shall be prepared to falls to ensure that construction thickness' remain uniform, Following trimming of the sub-grade it shall be protected against the ingress of water, failure to do so will seriously weaken the sub-grade.
13. All soft spots shall be excavated and replaced with compacted sub-base material
14. The minimum total carriageway construction thickness shall not be less than 450mm.
15. All materials used in top 450mm of carriageway construction shall be non-frost susceptible.

Rev	Date	Description	By / Chk
C01	26/11/2021	Amendments to site accesses	ST / JM
P07	15/11/2021	Porous car park simplified, levels added	ST / JM
P06	25/10/2021	Latest Architect's Site Plan added. Proposed Levels updated. Levels added around accesses.	SK / JM
P05	03/09/2021	Site plan updated to Architects latest layout. Proposed levels updated accordingly.	SK / JM
P04	12/07/2021	External works levels updated.	SK /BMS
P03	17/06/2021	dwg 0103 changed to 0110, BIM nos added	KMV/BMS
P2	12/02/2021	Bund levels added as land architect dets..	BMS/BMS
P1	15/12/2020	Issued for Comment.	SK /BMS

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Client: TUNGSTEN PROPERTIES LTD

Architect: HTC Architects

Project: Calder Park
Wakefield
WK4 3FL

Title: External Works

Status: **CONSTRUCTION**

Engineer:	SK	Date:	Nov 2020
Drawn:	SK	Scales @ A1:	1:500
Checked:	BMS		
PRP Ref No:	63150	Software:	2D AutoCAD 2010

Drg No: P21024-PRP-EX-00-DR-C-0110

Rev: C01

Calder Park

Tungsten Development. Surface Water Assessment.

Final Report

November 2020

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Revision History

Revision Ref/Date	Amendments	Issued to
13 November 2020	Draft Report	Mark Barwood
17 November 2020	Final Report	Mark Barwood

Contract

This report describes work commissioned by Mark Barwood, on behalf of Alan Wood and Partners, by email dated the 4 November 2020. Alan Wood and Partners' representative for the contract was Mark Barwood. Mark Bentley of JBA Consulting carried out this work.

Prepared by .... Mark Bentley BSc CEng CEnv FCIWEM C.WEM

Technical Director

Reviewed by .... Gavin Hodson BSc FdSc

Team Leader

Purpose

This document has been prepared as a Draft Report for Alan Wood and Partners. JBA Consulting accepts no responsibility or liability for any use that is made of this document other than by the Client for the purposes for which it was originally commissioned and prepared.

JBA Consulting has no liability regarding the use of this report except to Alan Wood and Partners.

Acknowledgements

The help with arranging the provision of data by Mark Barwood is gratefully acknowledged.

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JBA is aiming to reduce its per capita carbon emissions.

Contents

Appendices

List of Figures

List of Tables

Abbreviations

AEP	Annual Exceedance Probability
DEP	Donaldson Edwards Partnership
FEH	Flood Estimation Handbook
FRA	Flood Risk Assessment
HEC-RAS	Hydrologic Engineering Center – River Analysis System
JBA	Jeremy Benn Associates
mAOD	Metres Above Ordnance Datum
SSU	Scientific Support Unit
SuDS	Sustainable Drainage System
WYP	West Yorkshire Police

1 Introduction

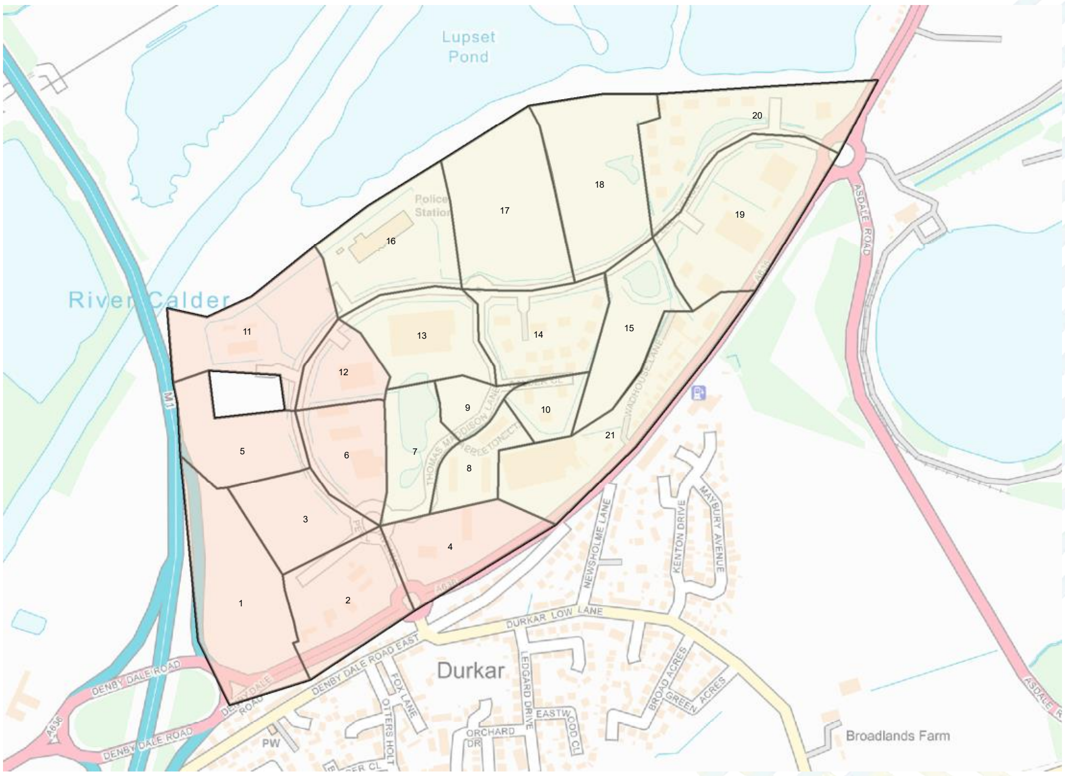
1.1 Flood risk assessment

The Calder Park development site is adjacent to Junction 39 of the M1. The site is bounded to the north and east by a flood embankment of the River Calder, to the west by the M1 motorway and to the south by Denby Dale Road. Between 2002 and 2018, JBA undertook work assessing the flood risk to the site and assisting with the design of the surface water drainage system. The surface water drainage system was designed to use a network of dry open channels (swales) and storage ponds to attenuate the runoff from the development plots.

There are proposals to develop a remaining plot of land at Calder Park. The proposed plot of land lies on the northern edge of Calder Park, just to the east of the Scientific Support Unit (SSU) building of West Yorkshire Police (WYP). The proposed platform level for the site will be set at 26.5mAOD. Appendix A contains drawings showing the proposed development. The total area of the plot of land for the new development is 60,700m^2 of which 51,950m^2 will comprise impermeable area. Therefore, 85% of the proposed will be impermeable which is greater than the value of 70% that was assumed for the design of the drainage system. This study has assessed the effect of increased runoff due to the greater impermeable area at the proposed site on water levels in the surface water drainage system.

JBA have re-used data previously collected on behalf of Peel Holdings, the developer of Calder Park. This includes topographical survey data showing ground levels and a computer model built for the business park surface water drainage system.

Figure 1-1 shows the Calder Park site. The proposed development will be in zones 17 and 18.



Not to scale

Figure 1-1: Calder Park location plan

1.2 Report structure

The report is presented in three sections:

1 Introduction – this section sets the study in context

2 Surface Water Runoff – flood risk from runoff within the Calder Park development

3 Conclusions

2 Surface water drainage

2.1 Surface water drainage system

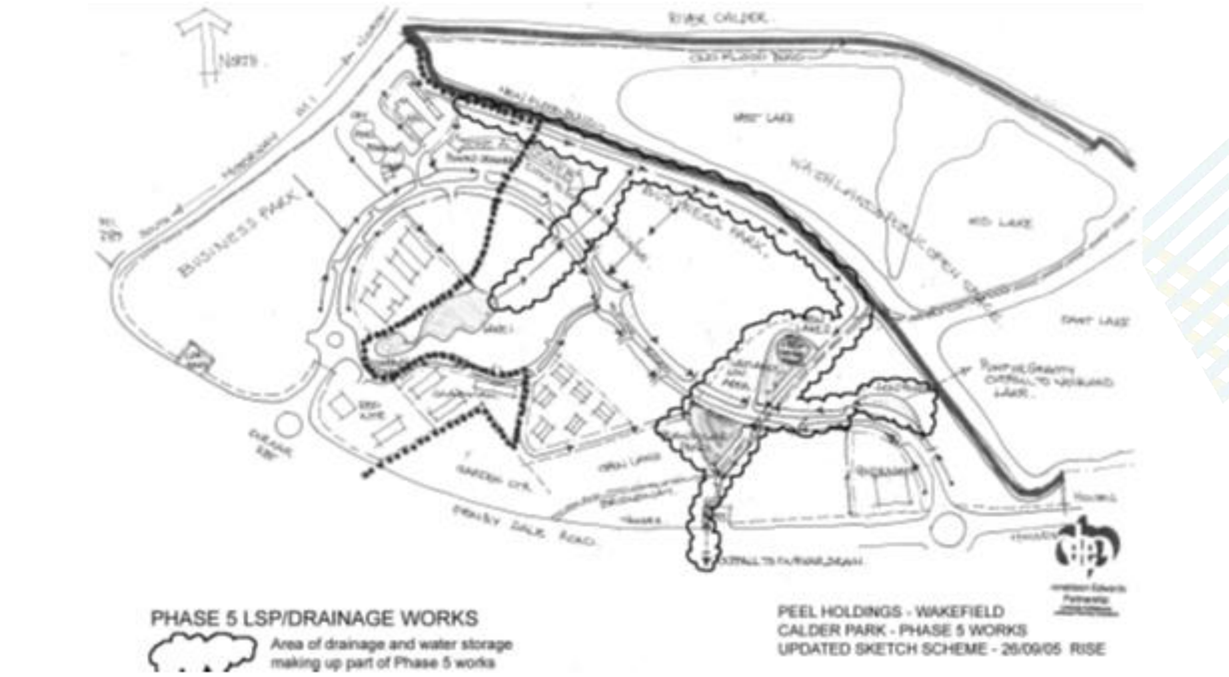
The drainage scheme for the Calder Park site utilises Sustainable Drainage System (SuDS) principles and is an integral part of the site design. The buildings, car parks and access roads are situated on raised platforms. A series of wide ditches (see Figure 2-1) run alongside the roads and between the building plots. There are also a few lakes that form part of the drainage scheme as well as improving the aesthetics of the site.

Figure 2-1: Typical swale at Calder Park

The site has been split into two surface water management zones, A and B (see Figure-2-2). Zone A utilises more traditional storm water drainage where runoff from roofs and roads is piped to a holding pond. This pond is then pumped directly to the River Calder at a rate of up to 1m^3/s. The system was designed such that there was sufficient storage in the pond to cope with runoff from a 100-year storm (including a 20% increase in rainfall intensity to allow for climate change).

Since the pond was designed the climate change allowance has been updated so that a 40% increase in rainfall intensity should be applied. Runoff calculations to the storage pond during a 1 in 100-year storm with an allowance of 40% increase in rainfall intensity show the pond level would rise to 26.0mAOD. That is 1m below the bank top (27.0mAOD). Therefore, the pond is large enough to deal with the revised allowances for climate change.

Runoff from all permeable areas and from subsequent plot developments (Zone B) goes directly to the ditches and is held in the land drainage system.



Note: Zone A is west of the dotted line and Zone B is to the east.

Figure 2-2: Calder Park drainage zones

The only outflow from the Calder Park site is to Durkar Drain. Water flows in a culvert (600mm diameter) under the Denby Dale Road. The ditch system at Calder Park connects to this culvert. The effective invert level of the culvert under the Denby Dale Road has been surveyed at 23.49mAOD. The lowest levels at Calder Park (in the north-east corner of the site) are lower than this. Therefore, it is not possible to drain the whole of Calder Park to Durkar Drain. A pumping station operates during high water levels to transfer water to the adjacent washland at a rate of 50l/s.

Surface water drainage for Calder Park has been calculated in three parts:

- Calculation of the volumes and rates of runoff during design events
- Hydraulic modelling of the ditch system to derive design water levels
- Calculations for the capacity of the culvert under Denby Dale Road.

2.2 Rainfall runoff calculations

Design rainfall depths for the site were calculated from the Flood Estimation Handbook (FEH). Two durations of storm were considered: 60-minutes and 7-hours for a 1%AEP (100-year return period) flood event.

Further consideration was given to the effects of climate change. The latest government guidance states that a 40% increase in rainfall should be used to account for climate change up to the year 2115 in the case of essential infrastructure .

A further storm profile comprising two consecutive 7-hour storms for a 3.3%AEP (30-year return period) was considered. This combination was used to assess the capacity of the surface water drainage system to recover from one storm before a further event occurs.

Table 2-1 summarises the design storms.

Table 2-1: Volume of storm rainfall

Storm Duration	3.3%AEP (30-year return period)			1%AEP (100-year return period)		
	Rainfall depth (mm)	Volume over site (m^3)	Peak storm intensity (mm/hr)	Rainfall depth (mm)	Volume over site (m^3)	Peak storm intensity (mm/hr)
60-minute				40	22,287	168
7-hour	90	50,298	18	62	34,234	24
with 40% allowance for climate change						
60-minute	Climate change scenario only applies to the 1% AEP (100-year flood event)			56	31,202	235
7-hour				87	47,928	34

For the purposes of calculating surface water runoff at Calder Park, the site was divided into 21 sub-catchments (see Figure 1-1). The sub-catchments represent different development plots across the Calder Park site. The calculations for each sub-catchment and the split between permeable and impermeable areas were adjusted based on plans supplied by WSP. For this study the percentage impermeable area of zones 17 and 18 was increased from 70% to 85%.

The division of the site enables the calculation of runoff volumes to be split between sites draining to the storage pond for pumping to the Calder and sites draining to the open ditches and eventual discharge to Durkar Drain. Storm water runoff volumes were calculated assuming that impermeable areas had a 100% runoff rate and 30% for permeable areas. Table 2-2 summarises volume of runoff for each design storm.

Table 2-2: Volume (m^3) of surface water runoff

Design scenario	Storm duration	Runoff destination	
		Pumped discharge	Durkar Drain
3.3%AEP	Two consecutive 7-hour storms	13,537	24,688
1%AEP (100-year return period)	1-hour	5,975	10,887
	7-hour	9,177	16,722
1% AEP with climate change	1-hour	8,365	15,242
	7-hour	12,848	23,411

Further runoff calculations were undertaken to assess the effect of saturated ground conditions which assumed the runoff from permeable areas was also 100%. Table 2.3 summarises the volume of runoff for this situation.

Table 2-3: Volume (m^3) of surface water runoff

Design scenario	Storm duration	Runoff destination	
		Pumped discharge	Durkar Drain
3.3%AEP	Two consecutive 7-hour storms	13,537	36,962
1%AEP (100-year return period)	1-hour	5,975	16,313
	7-hour	9,177	25,057
1% AEP with climate change	1-hour	8,365	22,838
	7-hour	12,848	35,080

There is a further discharge to the drainage system from the SSU building of WYP. The building is heated/cooled using an open loop groundwater heat extraction system. Depending on the energy demand this may result in peak flows of 18l/s to the ditch system. Over a 60-minute storm this would result in, at most, an extra 65m^3 of flow. Over a seven-hour storm the maximum runoff would be 454m^3 assuming the system operates at peak capacity throughout the storm.

Although the discharge from the SSU is small it is continuous and so there is the potential for the flow to reduce the capacity of the drainage ditches prior to a storm commencing. Therefore, the simulations of the drainage system have assumed the ditches are partially full due to continuous flow from the groundwater heat extraction system.

2.3 Hydraulic modelling

2.3.1 HEC-RAS model

Modelling of the drainage ditches was undertaken using HEC-RAS. HEC-RAS is an unsteady state one-dimensional river modelling package developed by the Hydrologic Engineering Center of the US Army Corps of Engineers. The software allows the user to calculate the variation of water surface within a channel network during a storm. The system can handle a looped network of channels, a branched system or just a single river reach. The model also allows the simulation of the effect of flood storage on routing flood flows. The model runs quickly and so was able to represent the draining down of the drainage system following an event.

The HEC-RAS model was originally set up with ditch profiles based on information supplied by Peel Holdings. The ditches are generally constructed by making a 1-in-3 slope down from the plot level to the proposed ditch invert level, cutting a flat bed 1.5m wide at this level and then making a 1 in 3 slope up to meet the opposite bank height. The actual ditch widths therefore depend on the adjacent ground heights and the ditch invert level.

Figure 2-3 shows the model for the completed ditch system.

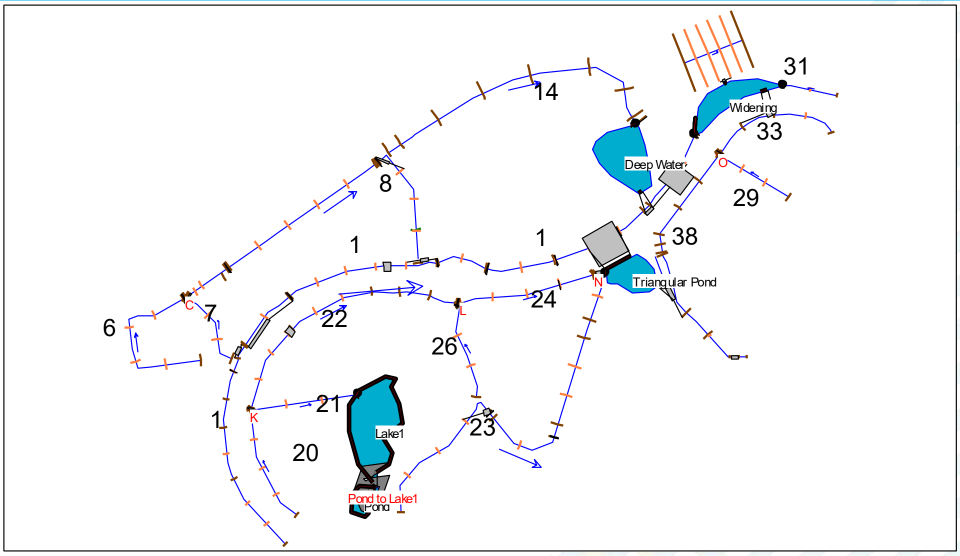


Figure 2-3: HEC-RAS schematic of completed drainage system

A combination of lateral weirs (sometimes incorporating culverts) and open channel junctions were used for connections between ditches and other ditches or lakes. To control water levels, retain the design standing water level and aid model stability (to prevent reaches becoming dry), a number of weirs were defined at intervals along the ditches. The weirs in the model were only defined where required to retain standing water levels.

Manning's *n* values (representing the roughness of the drains) were set to 0.03 at all locations. Since the drains are essentially a storage system (with flow controlled by weirs) the model results were expected to be insensitive to Manning's n values. However, sensitivity tests for this were undertaken (see Section 2.4.2). The model was run with a timestep of 6 seconds.

2.3.2 Standing water levels

At the start of a model simulation it is necessary to define the initial water levels in the ditches. In the case of the drainage system at Calder Park this is especially important because it will define the excess flood storage capacity in the system. The level of water in the ditches will depend on four factors:

- Drainage system design
- Groundwater levels
- Discharges from the SSU groundwater heat extraction system
- Recent storms, which have not fully drained away

To allow for high groundwater levels (see Chapter 4), the ditches have been designed to have a standing water depth of 0.5m with 1.5m depth in the lakes. Only ditches to the east of the Calder Park site would have deeper water (0.65m).

Initial conditions for the model simulations have been set assuming the groundwater heat extraction system is continuously discharging. This means that storage in the ditches at the start of the simulation has been partially used up before the storm begins.

Model simulations were also undertaken to assess the effect of two consecutive storms occurring. These tested the effect of high standing water levels that had not been able to drain out of the system before the onset of another storm. In this situation the standing water levels at the start of the storm are defined by the model following the simulation of runoff from the first storm.

2.4 Model results

The completed HEC-RAS model was run for the following three storms (all using realistic Runoff calculations – see Table 2-1), allowing outflow to Durkar Drain:

- 100-year, 60-minute Summer Storm
- 100-year, 7-hour Winter Storm
- Two consecutive 30-year, 7-hour Winter Storms

The tail-water level of the Denby Dale Road culvert was set at 25.2mAOD for the period of all the simulations. The flood level of 25.2mAOD was based on advice from Wakefield Council and represents the observed flood level on Denby Dale Road during the flood of the 25 June 2007. This is equivalent to a surcharged condition for the culvert and represents the case of impeded discharge from Durkar Drain to the River Calder (see section 2.5). Under these conditions there is backflow from Durkar Drain into the Calder Park site.

2.4.1 Water levels

The model simulations showed that generally across the site the two consecutive storms had the most impact on water levels in the drainage system. This is because high water levels at Denby Dale Road (25.2mAOD) prevent runoff escaping to Durkar Drain.

Therefore, water levels in the drainage system are sensitive to the runoff volume as opposed to the rainfall intensity.

This conclusion is different to that found during the original FRA in 2008. That study assumed a lower flood level at Denby Dale Road of 25.0mAOD and allowed water to leave Calder Park between consecutive storms.

Table 2-4 shows the highest water level in each ditch from each of the three simulations based on the final ditch layout with development completed. The ditch reach labels can be found on Figure 2-3. Simulated flood levels remained in bank throughout the ditch system.

Table 2-4: Maximum water levels in ditches - Completed drainage system

Upper site		Lower site	
Ditch	Maximum water level (mAOD)	Ditch	Maximum water level (mAOD)
1	26.28	23	25.76
6	25.28	24	25.62
7	25.29	26	25.62
8	25.33	29	25.22
14	25.31	31	25.22
20	26.18	33	25.22
21	26.18	38	25.22
22	26.18		

The effect of increased runoff from the site of the proposed development due to a greater impermeable area has a marginal impact on water levels. The only ditches where an impact was seen were ditches 29 and 38 where the water level increased by just 0.01m.

The proposed plot for the Tungsten Development will be set to 26.5mAOD. The maximum water level in the pond Deep Water and the ditches surrounding the development does not rise above 25.33mAOD. Therefore, there is over 1m freeboard.

2.4.2 Increased runoff (saturated ground conditions)

A sensitivity test with increased runoff to account for saturated ground conditions was undertaken. Runoff from the permeable areas of the site was assumed to be 100% (see section 2.2) and the SSU open loop groundwater heat extraction system was flowing at its maximum rate of 18l/s. This increased the total volume of runoff from the site and raised water levels in the ditches.

Table 2-5 shows the maximum water levels in the ditches for the completed system.

Table 2-5: Maximum water levels in ditches – 100% runoff

Upper site		Lower site	
Ditch	Maximum water level (mAOD)	Ditch	Maximum water level (mAOD)
1	26.60	23	25.83
6	25.47	24	25.83
7	25.58	26	25.83
8	25.47	29	25.24
14	25.36	31	25.24
20	26.23	33	25.24
21	26.23	38	25.24
22	26.23		

Generally, increasing the runoff has raised water levels across the site by the order of a few centimetres. The only significant impact on water level were in Ditches 1, 6, 7, 8, 24 and 26 where levels were raised by 0.32m (ditch 1), 0.29m (ditch 7), 0.14m (ditch 8), 0.21m (ditches 24 and 26). Even so, with saturated runoff, peak water levels were contained within bank.

Even with saturated runoff, the freeboard for the Tungsten Development in the pond Deep Water and the ditches surrounding the development is maintained at over 1m.

2.4.3 Climate change

A sensitivity test for the effects of climate change was also undertaken by applying a 40% increase in the volume of runoff. Table 2-6 shows the peak water levels in each ditch for the climate change scenario.

Table 2-6: Maximum water levels in ditches – Climate change

Upper site		Lower site	
Ditch	Maximum water level (mAOD)	Ditch	Maximum water level (mAOD)
1	26.36	23	25.81
6	25.33	24	25.78
7	25.35	26	25.78
8	25.41	29	25.24
14	25.39	31	25.24
20	26.23	33	25.24
21	26.23	38	25.24
22	26.23		

The impact of climate change was generally less than that due to saturated ground conditions and water levels were retained within the ditch system. For the climate change scenario, the freeboard for the Tungsten Development in the pond Deep Water and the ditches surrounding the development is maintained at over 1m.

2.4.4 Channel Roughness

The sensitivity of the model results to increased channel roughness was assessed by doubling the Manning's *n* coefficient from 0.03 to 0.06 across the site. There was very little sensitivity to Manning's *n* in the lower part of the site which is storage-dominated (only ditch 23 in this area showing an increase in water levels of 0.07m). A more widespread sensitivity was seen in the upper site where flux is more important, but most increases were less than 0.03m.

2.5 Durkar Drain

The outflow from Calder Park to Durkar Drain passes in culvert under the Denby Dale Road, then through a short section of open channel near a foul water pumping station and then into another culvert.

As part of the Wakefield Flood Defence Strategy developed by the EA, the outflow of Durkar Drain to the River Calder is flapped. It is expected that the flap will close for events greater than the 1-in-2-year flood on the River Calder. A further flap valve at the area known as 'Pugneys Entrance' closes during a 1-in-40-year flood event on the River Calder. This results in the Durkar Drain being 'tidelocked'. Durkar Drain is an ordinary watercourse and so is the responsibility of Wakefield Council.

Wakefield Council has purchased mobile pumps that can be deployed across their council area. Therefore, emergency pumps could be deployed to pump excess water from Durkar Drain to the River Calder during flood events. However, consultations with Wakefield Council indicate that this is not certain.

Therefore, the design runs for to test the surface water drainage system at Calder Park have made a conservative assumption that emergency pumps are not deployed to Durkar Drain. In this case water levels in Durkar Drain will backup and so a downstream level of 25.2mAOD has been applied. Under this scenario there will be backflow to the Calder Park site. In normal circumstances this situation should be avoided because of the risk of polluted water stagnating within the system of swales at Calder Park.

If emergency pumps are deployed, they should lower levels in Durkar Drain provided it is kept clear downstream of Denby Dale Road. With these lower levels (24.5mAOD) there would be no backflow on to the Calder Park site. In this case the drainage system would only be required to store runoff from Calder Park.

2.6 Culvert capacity

The culvert under the Denby Dale Road is a 600m diameter pipe. The invert level of the culvert is approximately 23.27mAOD. This information contrasts with a spot level survey (Donaldson Edwards Partnership (DEP), June 2005) which indicated that the invert at the culvert entrance is 23.49mAOD. This apparent discrepancy is probably caused by the large amount of siltation at the culvert. It is likely that the DEP level is the effective invert level due to siltation of the bed (i.e. 0.22m of silt blocking the bottom of the culvert). At the time WSP stated it was not possible to determine the invert of the pipe through the silt. The WSP information (from heights relative to footpath levels) indicated a downstream invert level of 23.22mAOD. The road is approximately 30m wide, which would give the culvert a slope of 1 in 500.

Surface runoff from Calder Park will not be restricted. Modelling (with a clear culvert) indicates that unrestricted flow leaving the site will exceed the agricultural runoff rates. This should not cause problems for Durkar Drain because in general rainfall on the site will enter Durkar Drain before the River Calder rises, 'tidelocking' the drain.

When the Durkar Drain is 'tidelocked', emergency pumps may be deployed by Wakefield Council to pump some of the flow that could be expected in Durkar Drain. If the culvert were totally blocked water levels could rise to 25.2mAOD, the boundary condition used to test the performance of the surface water drainage system at Calder Park (see 2.4).

3 Conclusions

3.1 Surface water drainage

The drainage scheme for Calder Park uses SuDS principles as an integral part of the site design. A series of wide ditches run alongside the roads and between the development platforms. There are also a number of lakes that form part of the drainage scheme as well as providing an aesthetic attraction.

Generally peak levels in the drainage system across the site were caused by consecutive storm events. This is because the model tests used high water levels at Denby Dale Road (25.2mAOD) that prevent runoff escaping to Durkar Drain. Therefore, water levels in the drainage system are sensitive to the runoff volume as opposed to the rainfall intensity.

Model simulations of the surface water drainage system have been undertaken for the proposed completed drainage system with a full developed business park. The simulations tested the impact of increased runoff from the proposed development on water levels in the surface water drainage system.

The effect of increased runoff from the site of the proposed development due to a greater impermeable area was shown to have a marginal impact on water levels. The maximum increase in water level of 0.01m was confined to just two ditches (number 29 and 38). However, simulated flood levels remained in bank throughout the ditch system.

With respect to the proposed Tungsten Development, flood levels in the Deep Water pond and surrounding ditches/ swales are of sufficient volume to prevent flooding of the Property. Under a 1 in 100-year event with an extra 40% allowance for climate change these is a freeboard of over 1m in the drainage system surrounding the site. Therefore, the surface water network can accommodate the free discharge of runoff from the Property based on an impermeable area of 51,950m^2.

3.2 Durkar Drain

The discharge of surface water from Calder Park is to Durkar Drain via a culvert under Denby Dale Road. As part of the Wakefield Flood Defence Strategy developed by the EA, the outflow of Durkar Drain to the River Calder flows through two flapped structures. As a results, during a 1-in-40-year flood event on the River Calder, Durkar Drain is 'tidelocked'. Durkar Drain is an ordinary watercourse and so is the responsibility of Wakefield Council.

Wakefield Council has purchased mobile pumps that can be deployed across their council area. Therefore, emergency pumps could be deployed to pump excess water from Durkar Drain to the River Calder during flood events. However, consultations with Wakefield Council indicate that this is not certain.

Therefore, design runs for this FRA have made a conservative assumption that emergency pumps are not deployed to Durkar Drain. In this case water levels in Durkar Drain will backup and so a downstream level of 25.2mAOD has been applied. Under this scenario there will be backflow to the Calder Park site. In normal circumstances this situation should be avoided because of the risk of polluted water stagnating within the system of swales at Calder Park.

If emergency pumps are deployed, they should lower levels in Durkar Drain provided it is kept clear downstream of Denby Dale Road. With these lower levels (24.5mAOD) there would be no backflow on to the Calder Park site. In this case the drainage system would only be required to store runoff from Calder Park.

Appendix A Proposed development at Calder Park

DO NOT SCALE!
ALL DIMENSIONS SHOULD BE CHECKED ON SITE BEFORE WORK COMMENCES

BOUNDARY SUBJECT TO TITLE PLAN CHECK

SUBJECT TO REVIEW OF LEVELS AND FALLS

Rev.	Date	Description	Drawn
C	29.09.20	Office area amended to include SF. Areas updated to include gatehouse / control tower. Parking numbers added. Note for attenuation tank added.	DW
B	05.08.20	Sprinkler tanks moved	DW
A	04.08.20	Boundary amended inline with information from solicitors - drawing Calder Park - Ordnance Survey Mastermap. Site plan amended accordingly and detailing updated.	DW

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client
Tungsten

project
Calder Park, Wakefield

drawing title
Proposed Site Plan

date August 2020
status Feasibility
scale 1:500 @ A0
drawn DW checked
job no. 2479 dwg no. F404 rev. C

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REPORT on the DRAINAGE STRATEGY for the PROPOSED DEVELOPMENT at CALDER PARK COMMERCIAL DEVELOPMENT PEEL AVENUE, WAKEFIELD, WEST YORKSHIRE WF4 3FL

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Date	Revised	Revision
02/12/2020	-	-
03/12/2020	-BMS	-Rev A
Reference: 63150	-	-

CONTENTS

EXECUTIVE SUMMARY

This drainage strategy report has been written in support of a planning application to develop of 15.215ha of land for a large commercial unit. The development is located adjacent to Peel Avenue, Calder Park Commercial Estate, Wakefield within 1km of junction 39 of the M1 motorway.

Foul effluent from the development will be discharged ultimately to an existing foul sewer located ithin Peel Avenue which has been installed by the vendor to support the development. All foul sewerage on site and within Peel Avenue is private and so a pre development enquiry has not been submitted toi Yorkshire Water.

Surface water from the development will be freely discharging into large deep swales surrounding three boundaries on the site, with a balancing pond located on the eastend boundary. A previous Surface Water Assessment report prepared by JBA Consulting indiactes that the peripheral surface water drainage infrastucture has been designed to cater for all the surface water run-off from the site and attenuation and SuDS mitigation measures are within the entire surface water drainage network.

The on-site and off-site network has been designed for a 1 in 100 year storm event plus 40% allowance for climate change and a factor of safety of 2 included in the calculations. Approval will be required from the regulatory bodies before the detail design is finalised.

The use of soakaways and other infiltration techniques has been considered but not deemed viable since the site is underlain by several meters of made ground which is predominantly of a impermeable cohesive nature and so in accordance with the SuDS hierarchy the surface water is being discharged into the surrounding swales.

Water quality will be improved through the use of permeable paving for the car parking areas. However, there is little opportunity to provide further public amenity or biodiversity to the site. Although there are some areas which will be landscaoed and shown on the architects appended plans.

This report demonstrates that a suitable Sustainable Urban Drainage System (SuDS) can be implemented for this scheme which complies with the NPPF. Both water quantity and water quality issues have been addressed and demonstrate that the environmental issues will not be compromised as a result of this scheme.

1. INTRODUCTION

1.1. This drainage strategy report has been commissioned by Carbide Properties Limited who are proposing to construct a 304,550sq ft (2.83ha) warehouse unit. The new unit will occupy most of the land with the remaining area being concrete yard and permeable car parking. There will be offices and car parking for 302 cars, along with 22 lorry delivery docks and a driver rest area allocated within the new design.

1.2. Around all boundaries there is a landscaping margin which is up to 19m wide along the Peel Avenue frontage.

1.3. The objective of the report is to demonstrate that proper consideration has been given to SuDS principles for the surface water drainage design. This report summarises the proposals in the context of current legislation.

1.4. In terms of flood risk, this has been addressed in a separate flood risk report titled Calder Park – Tungsten Development. Surface Water Assessment - prepared by JBA Consulting Dated November 2020. This report is an update of the original Flood Risk Report dated January 2018, at the time of writing the original report there was no site layout and so an assumption was made with regard to the impermeable area. This original report was updated to account for the proposed site layout and appended to this report. Although the report principally deals with the flood risk to the site, the issue of surface water outfalls from the site is covered extensively, this is because the run-off from this site is just a small part of a much larger private surface water network which covers the whole of Calder Park.

1.5. The site is located adjacent to the northern side of Peel Avenue, Wakefield WF4 3FL and is currently undergoing enabling works including significant earthworks to raise the levels. Peel Avenue is a private road and is connected at both ends to Denby Dale Road.

1.6. The proposal includes the construction of two separate accesses, one for car parking and one for commercial vehicles but both will enter from Peel Avenue. Large delivery and service vehicles will directly enter the service yard from the highway to the front of the building, this area will be designed as impermeable with a gate house control point. The car park will be located at the eastern side of the main warehouse and will consist of impermeable circulatory route with parking bays to be designed to a permeable pavement specification.

1.7. The layout is outlined in drawings produced by HTC Architects and this is appended to this report. Discussions with the client and the highways engineer have revealed that there will be no adoptable roads within the industrial estate. It is understood that on-site sewerage will remain private and therefore will NOT be adopted by Yorkshire Water. SuDS features are to be maintained by the owner via a management company and not the local water authority.

1.8. The floor level for the building has been set at 27.00m AOD which is in line with the original flood risk assessment report as this is at a similar level to the 1 in 1000-year level for the washland adjacent to the River Calder (27.04mAOD). Given that the building will be nominally 150mm higher than the surrounding ground level this is considered adequate and in line with the JBA Consulting proposals.

1.9. The general conditions contained in the appendices are applied to this report and it should be read and construed accordingly.

2. SITE LOCATION AND INFORMATION

2.1. The site is located within Calder Park Commercial Estate on Peel Avenue, Wakefield, WF4 3FL which is a private road off A636, Denby Dale Road. The site is located approximately 1km east of junction 39 of the M1 and has a NGR of 431822, 417996. The site has an area of 15.215ha and is presently open waste land undergoing enabling works.

2.2. The site is generally flat although at the present time it is undergoing earthworks to raise the levels and create a plateau of 26.500m AOD. The site is bounded on the north by the River Calder catchment and a flood embankment (managed by the Environment Agency), to the east there is an attenuation pond (Deepwater) and to the west there is a new police station and forensic services building. On the northern, southern and western boundaries there is a large swale which in some places is 3m wide at the bottom, this is designed to act as attenuation and also convey the surface water through the surface water network. The final outfall for the whole of the Calder Park development is at Durkar drain to the south of the site.

2.3. The immediate surrounding land uses are:

Boundary	Adjacent Land Use	Proximal Land Use
North	Open countryside	Open countryside
East	Business unit	Commercial business unit
South	Private highway	Commercial business unit
West	Business units	Commercial business unit

3. PROPOSED DRAINAGE SCHEME

3.1. Foul water – Existing Conditions

3.1.1. For this type of development, it is usual to submit a Pre-Development enquiry to Yorkshire Water which would detail where the foul and surface water sewers are located and at what invert level however, this has not been undertaken since all the foul and stormwater sewers remain private for this development.

3.1.2. The site has been sold as a serviced site and consequently foul water connections are being provided, a copy of the engineers drawing is appended to this report to show the location and depths of these private sewers.

3.2. Foul Water Drainage Proposals

3.2.1. There are generally three methods for dealing with foul effluent that is produced from a commercial development: -

A) Public/private sewers – The use of private sewers is always the preferred option since they are relatively maintenance free and require no additional area to discharge the foul water.

B) Septic tank – This is quite simply a tank, usually buried below ground, which collects the foul effluent and is periodically emptied, occasionally it also has a herringbone drainage system fitted for removing the 'clean' water that collects in the upper layers of the tank, these are seldom used in a commercial context, as they require high levels of maintenance and can produce foul odours if not maintained correctly. A septic tank would be unsuitable for a development of this type for a multiple number of reasons including lack of available space within a densely occupied area. The Environment Agency are also preventing the installation of new septic tanks and encouraging the installation of small treatment stations where other proposals are not suitable or not available.

C) A small treatment works – These are generally used where there is no public sewer within the vicinity, but only if the General Binding Rules for Small Sewage Discharges are met, or if a permit is obtained from the Environment Agency. Small treatment works discharge either to the ground or to surface water. Treatment works are seldom used where foul sewers are available for connections.

3.2.2. As stated above, the preferred option for this type of development is to discharge into a public/private foul sewer. Alan Wood and Partners have provided sewer details showing the approximate point of connection and confirmation that proposed outfall can discharge into the private sewers.

3.2.3. An indirect section 106 application will be required for the indirect connection into the public sewer with the appropriate application fee but this should not be a significant issue to obtain during the design stage.

3.2.4. The architectural site plans indicate that the new unit will be larger than the existing unit and so there may well be an increase in staff number. For the purposes of initial design, reference has been made to British Water Code of Practice Loads and Flows – 4, it is assumed that the warehouse will have canteen facilities and so the flow/day for a commercial development is 100l/day. The layout indicates that there is 302 car parking and so the predicted maximum daily flow of 30 200litres or 0.4l/sec based upon a 24-hour day, at full warehouse capacity. It should be noted that some of the parking spaces will be used for visitors and therefore the above figure could be seen as a conservative value.

3.2.5. It should be noted that the effluent from any staff canteen will be fitted with a proprietary grease trap to ensure that fat and grease are not discharged into the main foul sewer. The grease trap will also be maintained as part of the unit's maintenance and health and hygiene regime.

4. SURFACE WATER DRAINAGE SYSTEM

4.1. Existing Conditions

4.1.1. The site has not been developed previously and prior to construction of Calder Park it was open countryside and no doubt the run off from the fields passed into a series of ditches, however as Calder Park has been developed the surface water drainage system, as described in the flood risk assessment, for the whole development has been installed.

4.1.2. The site is surrounded on three sides with swales and these are up to 3m wide at the bottom on the northern elevation, the swales have been designed to accommodate run-off from this site and it has been designed to not flood for a 1 in 100 year event plus 40% for climate change. The swales act as carrier drains and provide a treatment train for SuDS purposes.

4.1.3. The existing network is a combination of lateral weirs (sometimes incorporating culverts) and open channel junctions to connect swales and ditches to lakes.

4.2. Surface Water Drainage Proposals

4.2.1. Government directives in the form of the Building Regulations Part H3 indicate that the following hierarchy should be adopted as a means disposing of the surface water run-off: -

- Soakaways
- Discharge into a watercourse at an appropriate rate.
- Discharge into a surface water sewer at an agreed rate.

4.2.2. In accordance with The SuDS Manual, the design of the surface water system should consider water quantity, water quality, amenity and biodiversity and this report considers each of these four categories.

4.2.3. **Water Quantity** – the whole development has been designed as a single network with attenuation and flow control throughout the system, the HEC-RAS model shown in the report assumes free flowing discharge into the swales and this flows around the network and eventually outfalls into Durkar Drain. Surface water modelling indicated in clause 2.4.3 of the FRA (1 in 100 year plus 40% for climate change) indicates that the maximum water level in Deep Water will not exceed 25.330m AOD which is over 1m lower than the finished plateau level. Therefore an unattenuated outflow from the site will not constitute a problem to downstream flooding.

4.2.4. The system has been designed for an impermeable are of 51 950m^2 which represents all hard surfacing and building across the site, however there will be 303 car parking spaces and associated access roads which will be designed as porous paving and although this will be designed as a type 3 system not all the run-off will outfall the site, this area equates to 7094m^2. The outflow from this area will be attenuated as the run-off passes through the Corse granular aggregate below.

4.2.5. The onsite surface water network will remain private, but approval from the regulator will still be required.

4.2.6. Given that the runoff from the development as a whole will be restricted to the agreed run-off from the site, water quantity issues have been addressed.

4.2.7. **Water Quality** – With reference to The SuDS Manual, CIRIA C753, consideration is also given to water quality. The opportunities to provide an improvement to the water quality are limited as the site will be developed in keeping with the current surroundings but nevertheless some SuDS techniques are discussed below. With reference to the architect's site layout drawings and table 4.3 of the SuDS Manual the pollution hazard level is low for a majority of the site, however the service areas around the buildings are considered to be a medium pollution hazard potential.

4.2.8. Green Roofs - both extensive and intensive green roofs have been considered; given the infrastructure that will be present on the roof and the increase in structural support and maintenance required it would be unlikely that a green roof would be cost effective for this type of development. It is also extremely unlikely that the planning officers would permit this in view of the fact that none of the surrounding buildings have these types of roofs. On this basis, green roofs are not considered appropriate.

4.2.9. Blue Roofs – Blue roofs are flat roofs used to hold water and release it over a period of time; these are not viable for the site. A Blue roof is not in keeping with the local architecture and would require significant additional structural support.

4.2.10. Infiltration Systems – These include infiltration blankets, soakaways, trenches and infiltration basins. Filter drains improve water quality by reducing pollutant levels in runoff by filtering out fine sediments, metals, hydrocarbons and other pollutants. They also encourage adsorption and biodegradation processes. The use of filter drains on this site has been considered and although no permeability testing has been undertaken, filter drains are not considered appropriate since the underlying soil is predominantly cohesive and the amount of standing water on site indicates that infiltration systems are unlikely to work on this site. the limited infiltration that is available will be maximized by discharging all the run-off into the surrounding swales and detention basin.

4.2.11. Pervious Pavements - It is proposed to construct all 300 car parking spaces as permeable paving so that the surface water is cleaned as it passes through coarse graded aggregate sub-base. Due to the underlying geology, a Type C system will be used, with filter and carrier pipes to distribute the attenuated run-off throughout the permeable paving structure. The pipes will be laid in trenches integral with the sub-base. The main access routes into the car parking and to the offices will be in impermeable bituminous construction but the roads will be cambers so that all the run-off passes through the porous paving car parking.

4.2.12. However, it is not possible to use permeable paving for the service yards, as the weight of Heavy Goods Vehicles would damage the permeable surfaces and it would degrade very quickly. There is only a low to medium potential hazard for pollutants from this site and the use of porous paving is considered entirely appropriate for a development of this type. The exact type of porous paving is not known at present and will have to be approved by the planning officer; however, the use of concrete block permeable paving is the preferred option. With reference to Table 26.3 of the SuDS Manual permeable paving would provide suitable SuDS mitigation indices for a development of this type and therefore when combined with the development wide attenuation, the paving construction would provide an adequate storm water treatment train for the site by filtering out the silts and hydrocarbons.

4.2.13. The pervious pavements will be maintained by the end user in accordance with the section 6 of this report. The maintenance schedule will be included in the Operation and Maintenance manual for the buildings for future reference.

4.2.14. **Amenity** - defined as ‘’a useful or pleasant facility or service’’ in the SuDS Manual. The development has the opportunity to provide an amenity within the landscaped areas and with a suitably landscaped attenuation pond the development is considered suitable.

4.2.15. **Biodiversity** – there are soft landscaped areas along all four boundaries to this site with the widest margin being up to 19m wide on the front elevation and this will provide some biodiversity within the development. On the eastern side of the site there is an attenuation pond which will be sympathetically landscaped and planted to provide a suitable area for wildlife.

4.2.16. A plan showing the on site preliminary drainage strategy for the development is appended to this report in Appendix III.

4.3. Design Consideration of Exceedance

4.3.1. Whilst the offsite network has been designed to demonstrate that there will be no flooding for a 1 in 100 year event plus 40% for climate change, it is important to consider exceedance, in the event of an exceptional rainfall event, that is with a return period of over 100 years, or in the event of a system failure. Should this occur, the water would back up into the impermeable yard area and the spare capacity within the permeable paving attenuation. No allowance will be made for the storage in the permeable paving sub-base in the normal case. Should the sub-base become completely saturated, water would pond above ground within the car park area. The proposed level in the porous paved areas will also be retained by a 125mm half battered kerb.

4.3.2. The external levels will generally be set 150mm lower than the ground floor slab although level thresholds will be provided, this will further reduce the propensity for any run-off to enter into the buildings in the event of exceedance occurring.

4.3.3. The measures described above will be sufficient to provide enough time for employees and guests to exit the building in the event of an abnormal flood occurring.

5. MAINTENANCE OF THE SURFACE WATER DRAINAGE SYSTEM

5.1. It is recognised that the surface water drainage system has been designed with future maintenance in mind. Pervious paving construction design and catch-pits will trap the vast majority of silt to prevent entry into the attenuation tank.

5.2. A maintenance and management plan has been proposed and is appended to this report.

5.3. The measures are suggestions and the frequency of maintenance should be adjusted if necessary, to suit levels of silt build up and defects in the system. The schedule appended to this report will be included in the Operation and Maintenance manual for future reference.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1. Foul water – the foul effluent from the development can be discharged into the existing foul sewers located along Peel Avenue. The foul effluent will discharge into an existing private sewer as confirmed by the vendor. The connection to a private sewer will be subject to a S106 agreement with Yorkshire Water.

6.2. Surface water – The surface water drainage system from the development will discharge into swales and an attenuation pond surrounding the site, further treatment trains will be provided by discharging the surface water from the car parking areas into the permeable paving prior to discharging into the balancing pond.

6.3. This report demonstrates that the opportunities for suitable Sustainable Urban Drainage Systems are limited for this site but sufficient techniques can be implemented to comply with the NPPF. Both water quantity and water quality issues have been addressed and enhanced, the report also demonstrates that environmental issues will not be compromised as a result of this scheme.

6.4. A management company will maintain the surface water drainage system, and a maintenance and management plan has been appended to this report.

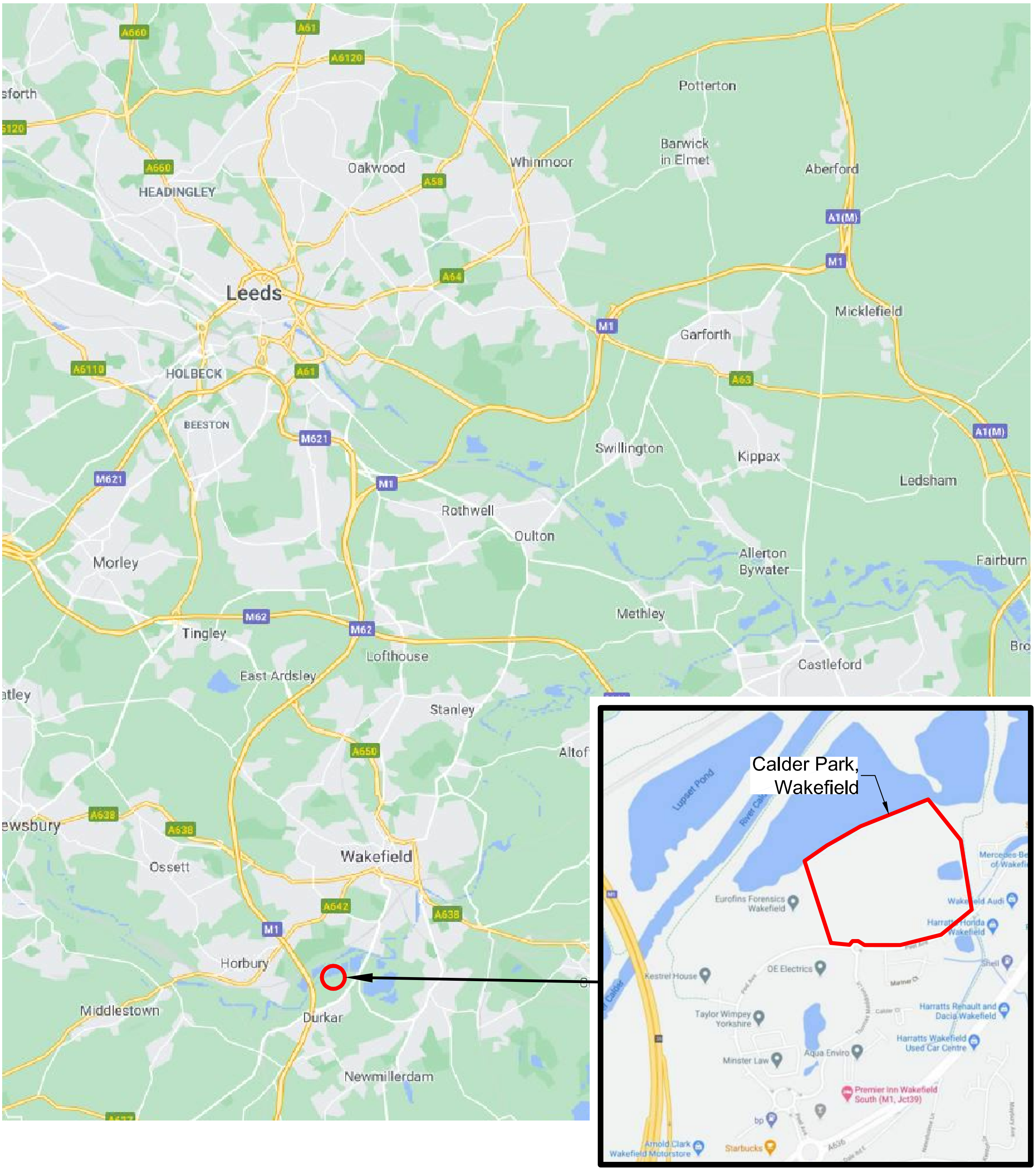
Paul Daniel,
Drainage Engineer,
PRP UK Ltd

B.M. Smith

Barry Smith
Director
PRP UK Ltd

APPENDIX I

SITE LOCATION PLAN



Project:	Title:
Calder Park, Wakefield	Site Location Plan

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Status: **PRELIMINARY**

Engineer:	KH	Date:	Nov 2020
Drawn:	GF	Scales @ A4:	NTS
Checked:	BMS		
Project No:	63128	Drg No: SK01	Rev: P1

APPENDIX II

ARCHITECT LAYOUT PLAN

DO NOT SCALE!
ALL DIMENSIONS SHOULD BE CHECKED ON SITE BEFORE WORK COMMENCES

BOUNDARY SUBJECT TO TITLE PLAN CHECK

SUBJECT TO REVIEW OF LEVELS AND FALLS

Rev.	Date	Description	Drawn
D	05.11.20	Amended inline with client instruction	DW
C	29.09.20	Office area amended to include SF. Areas updated to include gatehouse / control tower. Parking numbers added. Note for attenuation tank added.	DW
B	05.08.20	Sprinkler tanks moved	DW
A	04.08.20	Boundary amended inline with information from solicitors - drawing Calder Park - Ordnance Survey Mastermap. Site plan amended accordingly and detailing updated.	DW

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client
Tungsten

project
Calder Park, Wakefield

drawing title
Proposed Site Plan

date August 2020
status Feasibility
scale 1:500 @ A0
drawn DW checked
job no. 2479 dwg no. F404 rev. D

APPENDIX III

PROPOSED PRELIMINARY DRAINAGE LAYOUTS

Notes:

1. DO NOT SCALE FROM THIS DRAWING.
2. All dimensions are in millimetres Unless Noted Otherwise (u.n.o.)
3. Drawing is to be read in conjunction with all relevant architect's drawings. Any inconsistencies should be reported to PRP immediately.
4. All levels and dimensions are to be checked on site before any work commences.
5. For more information see PRP drawings:
   63104 - 100series - Drainage and External Works
   63104 - 200series - Foundations
   63104 - 300series - Superstructure
6. The Health and Safety at Work act is to be complied with at all times. Attention is drawn to the wearing of hard hats, safety boots, reflective clothing, and the use of any other required safety equipment.

Drainage:

1. The position, line, level and diameter of all existing drainage apparatus should be confirmed on site prior to the commencement of the works. Any discrepancies should be reported to PRP immediately.
2. The connection of foul and surface water drainage to the existing public sewer system shall be subject to the approval of the water authority
3. For positions of all rainwater pipes & foul outlets refer to Architect's drawings.
4. All drainage works shall be carried out in accordance with WRc "Sewers for Adoption - 7th edition"
5. All joints between precast manhole components shall have a minimum uncompressed thickness of 10mm of proprietary bitumen or resin mastic sealant.
6. Chambers & manholes with outgoing pipes of greater than Ø600mm shall be fitted with removable stainless steel (grade 316) safety chains or polypropylene rope.
7. Storm & foul branch connections are to be laid at gradients of between 1:10 & 1:80
8. All in-situ concrete shall be minimum grade GEN3.
9. Precast concrete cover & reducing slabs to be heavy duty reinforced concrete to BS 5911.
10. Rising mains shall be black MDPE SDR11 as WI 4-32-03 & joints & fittings to be in accordance with WI 4-32-04. Other approved pipe materials to be in accordance with their relevant BS.
11. Manhole covers & frames shall be manufactured in cast iron or ductile iron & shall comply with requirements of BS EN 124 & shall be kite marked or equivalent.
12. Where there is no intermediate manhole between the start of a surface water pipe run and the soakaway the gradient of the run shall be not less than 1 : 60.
13. All completed work shall be suitably protected from damage by construction work. Damaged drainage will not be accepted. It is recommended that no heavy loading or underground work is permitted above or near unprotected drainage, and that dumpers, trucks, fork lifts or other heavy vehicles are not driven along or near pipe runs.
14. Inspection chambers, soakaways and flow control units are to be installed strictly in accordance with manufacturer guidance and instructions

Drainage Legend:

- Existing Surface Water
- Existing Foul
- New Surface Water
- New Foul
- Perforated Pipe
- Rodding Eye
- Road Gully
- Foul Gully
- ACO / Channel Drain
- Type C Permeable Paving
- Existing Levels
- +100.000 New Levels
- 1/X Slope
- Flood Exceedance Route

Rev	Date	Description	By	Chk
P1	02/12/2020	Issued For Planning	SK	/BMS

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Client:
Tungsten Properties Ltd

Architect:
HTC Architects

Project:
Calder Park, Wakefield

Title:
Drainage Strategy

Status:
PRELIMINARY

Engineer:	SK	Date:	NOVEMBER 2020
Drawn:	SK	Scales @ A0:	1:500
Checked:	BMS		
Drawing No:	101	Rev:	P1

APPENDIX IV

MICRODRAINAGE CALCULATIONS

Summary of Results for 100 year Return Period (+40%)

Half Drain Time exceeds 7 days.

Outflow is too low. Design is unsatisfactory.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Volume (m³)	Status
15 min Summer	26.305	0.305	0.0	16.9	O K
30 min Summer	26.365	0.365	0.0	24.1	O K
60 min Summer	26.421	0.421	0.0	32.0	O K
120 min Summer	26.473	0.473	0.0	40.5	O K
180 min Summer	26.502	0.502	0.0	45.6	Flood Risk
240 min Summer	26.521	0.521	0.0	49.2	Flood Risk
360 min Summer	26.548	0.548	0.0	54.3	Flood Risk
480 min Summer	26.567	0.567	0.0	58.3	Flood Risk
600 min Summer	26.582	0.582	0.0	61.4	Flood Risk
720 min Summer	26.594	0.594	0.0	64.0	Flood Risk
960 min Summer	26.614	0.614	0.0	68.2	Flood Risk
1440 min Summer	26.640	0.640	0.0	74.3	Flood Risk
2160 min Summer	26.666	0.666	0.0	80.4	Flood Risk
2880 min Summer	26.683	0.683	0.0	84.6	Flood Risk
4320 min Summer	26.705	0.705	0.0	90.1	Flood Risk
5760 min Summer	26.718	0.718	0.0	93.5	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Time-Peak (mins)
15 min Summer	125.869	0.0	27
30 min Summer	84.215	0.0	42
60 min Summer	53.779	0.0	72
120 min Summer	33.180	0.0	132
180 min Summer	24.659	0.0	192
240 min Summer	19.841	0.0	252
360 min Summer	14.545	0.0	372
480 min Summer	11.669	0.0	492
600 min Summer	9.828	0.0	612
720 min Summer	8.537	0.0	732
960 min Summer	6.830	0.0	972
1440 min Summer	4.978	0.0	1452
2160 min Summer	3.621	0.0	2172
2880 min Summer	2.885	0.0	2892
4320 min Summer	2.092	0.0	4332
5760 min Summer	1.663	0.0	5776

Summary of Results for 100 year Return Period (+40%)

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Volume (m³)	Status
7200 min Summer	26.727	0.727	0.0	95.8	Flood Risk
8640 min Summer	26.733	0.733	0.0	97.2	Flood Risk
10080 min Summer	26.736	0.736	0.0	98.3	Flood Risk
15 min Winter	26.328	0.328	0.0	19.5	O K
30 min Winter	26.390	0.390	0.0	27.6	O K
60 min Winter	26.449	0.449	0.0	36.4	O K
120 min Winter	26.504	0.504	0.0	46.0	Flood Risk
180 min Winter	26.534	0.534	0.0	51.7	Flood Risk
240 min Winter	26.554	0.554	0.0	55.7	Flood Risk
360 min Winter	26.583	0.583	0.0	61.5	Flood Risk
480 min Winter	26.603	0.603	0.0	65.9	Flood Risk
600 min Winter	26.619	0.619	0.0	69.5	Flood Risk
720 min Winter	26.632	0.632	0.0	72.4	Flood Risk
960 min Winter	26.653	0.653	0.0	77.2	Flood Risk
1440 min Winter	26.681	0.681	0.0	84.1	Flood Risk
2160 min Winter	26.709	0.709	0.0	91.1	Flood Risk
2880 min Winter	26.728	0.728	0.0	95.9	Flood Risk
4320 min Winter	26.752	0.752	0.0	102.5	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Time-Peak (mins)
7200 min Summer	1.391	0.0	7208
8640 min Summer	1.201	0.0	8648
10080 min Summer	1.063	0.0	10088
15 min Winter	125.869	0.0	27
30 min Winter	84.215	0.0	42
60 min Winter	53.779	0.0	72
120 min Winter	33.180	0.0	132
180 min Winter	24.659	0.0	192
240 min Winter	19.841	0.0	250
360 min Winter	14.545	0.0	370
480 min Winter	11.669	0.0	488
600 min Winter	9.828	0.0	608
720 min Winter	8.537	0.0	726
960 min Winter	6.830	0.0	964
1440 min Winter	4.978	0.0	1442
2160 min Winter	3.621	0.0	2152
2880 min Winter	2.885	0.0	2864
4320 min Winter	2.092	0.0	4284

Summary of Results for 100 year Return Period (+40%)

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Volume (m³)	Status
5760 min Winter	26.768	0.768	0.0	106.8	Flood Risk
7200 min Winter	26.778	0.778	0.0	109.6	Flood Risk
8640 min Winter	26.786	0.786	0.0	111.7	Flood Risk
10080 min Winter	26.792	0.792	0.0	113.2	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Time-Peak (mins)
5760 min Winter	1.663	0.0	5712
7200 min Winter	1.391	0.0	7136
8640 min Winter	1.201	0.0	8560
10080 min Winter	1.063	0.0	9896

Rainfall Details

Rainfall Model	FSR	Winter Storms	Yes
Return Period (years)	100	Cv (Summer)	0.750
Region	England and Wales	Cv (Winter)	0.840
M5-60 (mm)	19.000	Shortest Storm (mins)	15
Ratio R	0.360	Longest Storm (mins)	10080
Summer Storms	Yes	Climate Change %	+40

Time Area Diagram

Total Area (ha) 0.091

Time From:	**(mins) To:**	**Area (ha)**	**Time From:**	**(mins) To:**	**Area (ha)**	**Time From:**	**(mins) To:**	**Area (ha)**
0	4	0.031	4	8	0.030	8	12	0.030

Model Details

Storage is Online Cover Level (m) 26.800

Porous Car Park Structure

Infiltration Coefficient Base (m/hr)	0.00000	Width (m)	15.1
Membrane Percolation (mm/hr)	1000	Length (m)	60.0
Max Percolation (l/s)	251.7	Slope (1:X)	80.0
Safety Factor	2.0	Depression Storage (mm)	5
Porosity	0.30	Evaporation (mm/day)	3
Invert Level (m)	26.000	Membrane Depth (m)	1

Summary of Results for 100 year Return Period (+40%)

Half Drain Time exceeds 7 days.

Outflow is too low. Design is unsatisfactory.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Volume (m³)	Status
15 min Summer	26.292	0.292	0.0	46.0	O K
30 min Summer	26.353	0.353	0.0	67.2	O K
60 min Summer	26.409	0.409	0.0	90.3	O K
120 min Summer	26.462	0.462	0.0	115.1	O K
180 min Summer	26.491	0.491	0.0	129.9	O K
240 min Summer	26.510	0.510	0.0	140.3	Flood Risk
360 min Summer	26.536	0.536	0.0	155.3	Flood Risk
480 min Summer	26.555	0.555	0.0	166.6	Flood Risk
600 min Summer	26.570	0.570	0.0	175.6	Flood Risk
720 min Summer	26.582	0.582	0.0	183.1	Flood Risk
960 min Summer	26.601	0.601	0.0	195.1	Flood Risk
1440 min Summer	26.627	0.627	0.0	212.2	Flood Risk
2160 min Summer	26.651	0.651	0.0	228.9	Flood Risk
2880 min Summer	26.667	0.667	0.0	240.2	Flood Risk
4320 min Summer	26.686	0.686	0.0	254.3	Flood Risk
5760 min Summer	26.697	0.697	0.0	262.3	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Time-Peak (mins)
15 min Summer	125.869	0.0	27
30 min Summer	84.215	0.0	42
60 min Summer	53.779	0.0	72
120 min Summer	33.180	0.0	132
180 min Summer	24.659	0.0	192
240 min Summer	19.841	0.0	252
360 min Summer	14.545	0.0	372
480 min Summer	11.669	0.0	492
600 min Summer	9.828	0.0	612
720 min Summer	8.537	0.0	732
960 min Summer	6.830	0.0	972
1440 min Summer	4.978	0.0	1452
2160 min Summer	3.621	0.0	2172
2880 min Summer	2.885	0.0	2892
4320 min Summer	2.092	0.0	4332
5760 min Summer	1.663	0.0	5768

Summary of Results for 100 year Return Period (+40%)

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Volume (m^3)	Status
7200 min Summer	26.703	0.703	0.0	266.7	Flood Risk
8640 min Summer	26.706	0.706	0.0	268.8	Flood Risk
10080 min Summer	26.707	0.707	0.0	269.8	Flood Risk
15 min Winter	26.315	0.315	0.0	53.5	O K
30 min Winter	26.378	0.378	0.0	77.3	O K
60 min Winter	26.437	0.437	0.0	103.3	O K
120 min Winter	26.493	0.493	0.0	131.1	O K
180 min Winter	26.523	0.523	0.0	147.7	Flood Risk
240 min Winter	26.543	0.543	0.0	159.4	Flood Risk
360 min Winter	26.571	0.571	0.0	176.3	Flood Risk
480 min Winter	26.592	0.592	0.0	189.1	Flood Risk
600 min Winter	26.608	0.608	0.0	199.3	Flood Risk
720 min Winter	26.620	0.620	0.0	207.8	Flood Risk
960 min Winter	26.640	0.640	0.0	221.5	Flood Risk
1440 min Winter	26.668	0.668	0.0	241.0	Flood Risk
2160 min Winter	26.694	0.694	0.0	260.4	Flood Risk
2880 min Winter	26.712	0.712	0.0	273.7	Flood Risk
4320 min Winter	26.734	0.734	0.0	290.9	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m^3)	Time-Peak (mins)
7200 min Summer	1.391	0.0	7208
8640 min Summer	1.201	0.0	8648
10080 min Summer	1.063	0.0	10088
15 min Winter	125.869	0.0	27
30 min Winter	84.215	0.0	42
60 min Winter	53.779	0.0	72
120 min Winter	33.180	0.0	132
180 min Winter	24.659	0.0	190
240 min Winter	19.841	0.0	250
360 min Winter	14.545	0.0	370
480 min Winter	11.669	0.0	488
600 min Winter	9.828	0.0	606
720 min Winter	8.537	0.0	726
960 min Winter	6.830	0.0	964
1440 min Winter	4.978	0.0	1438
2160 min Winter	3.621	0.0	2152
2880 min Winter	2.885	0.0	2860
4320 min Winter	2.092	0.0	4284

Summary of Results for 100 year Return Period (+40%)

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Volume (m^3)	Status
5760 min Winter	26.747	0.747	0.0	301.2	Flood Risk
7200 min Winter	26.755	0.755	0.0	307.6	Flood Risk
8640 min Winter	26.760	0.760	0.0	311.6	Flood Risk
10080 min Winter	26.763	0.763	0.0	314.2	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m^3)	Time-Peak (mins)
5760 min Winter	1.663	0.0	5712
7200 min Winter	1.391	0.0	7072
8640 min Winter	1.201	0.0	8480
10080 min Winter	1.063	0.0	9888

Rainfall Details

Rainfall Model	FSR	Winter Storms	Yes
Return Period (years)	100	Cv (Summer)	0.750
Region	England and Wales	Cv (Winter)	0.840
M5-60 (mm)	19.000	Shortest Storm (mins)	15
Ratio R	0.360	Longest Storm (mins)	10080
Summer Storms	Yes	Climate Change %	+40

Time Area Diagram

Total Area (ha) 0.267

Time From:	**(mins) To:**	**Area (ha)**	**Time From:**	**(mins) To:**	**Area (ha)**	**Time From:**	**(mins) To:**	**Area (ha)**
0	4	0.089	4	8	0.089	8	12	0.089

Model Details

Storage is Online Cover Level (m) 26.800

Porous Car Park Structure

Infiltration Coefficient Base (m/hr)	0.00000	Width (m)	45.0
Membrane Percolation (mm/hr)	1000	Length (m)	75.0
Max Percolation (l/s)	937.5	Slope (1:X)	80.0
Safety Factor	2.0	Depression Storage (mm)	5
Porosity	0.30	Evaporation (mm/day)	3
Invert Level (m)	26.000	Membrane Depth (m)	1

Summary of Results for 100 year Return Period (+40%)

Half Drain Time exceeds 7 days.

Outflow is too low. Design is unsatisfactory.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Volume (m³)	Status
15 min Summer	26.295	0.295	0.0	6.3	O K
30 min Summer	26.353	0.353	0.0	9.0	O K
60 min Summer	26.407	0.407	0.0	11.9	O K
120 min Summer	26.458	0.458	0.0	15.1	O K
180 min Summer	26.486	0.486	0.0	17.0	O K
240 min Summer	26.504	0.504	0.0	18.3	Flood Risk
360 min Summer	26.530	0.530	0.0	20.2	Flood Risk
480 min Summer	26.549	0.549	0.0	21.7	Flood Risk
600 min Summer	26.564	0.564	0.0	22.9	Flood Risk
720 min Summer	26.576	0.576	0.0	23.9	Flood Risk
960 min Summer	26.594	0.594	0.0	25.4	Flood Risk
1440 min Summer	26.620	0.620	0.0	27.7	Flood Risk
2160 min Summer	26.645	0.645	0.0	29.9	Flood Risk
2880 min Summer	26.661	0.661	0.0	31.5	Flood Risk
4320 min Summer	26.682	0.682	0.0	33.5	Flood Risk
5760 min Summer	26.695	0.695	0.0	34.8	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Time-Peak (mins)
15 min Summer	125.869	0.0	27
30 min Summer	84.215	0.0	42
60 min Summer	53.779	0.0	72
120 min Summer	33.180	0.0	132
180 min Summer	24.659	0.0	192
240 min Summer	19.841	0.0	252
360 min Summer	14.545	0.0	372
480 min Summer	11.669	0.0	492
600 min Summer	9.828	0.0	612
720 min Summer	8.537	0.0	732
960 min Summer	6.830	0.0	972
1440 min Summer	4.978	0.0	1452
2160 min Summer	3.621	0.0	2172
2880 min Summer	2.885	0.0	2892
4320 min Summer	2.092	0.0	4332
5760 min Summer	1.663	0.0	5776

Summary of Results for 100 year Return Period (+40%)

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Volume (m³)	Status
7200 min Summer	26.703	0.703	0.0	35.6	Flood Risk
8640 min Summer	26.708	0.708	0.0	36.1	Flood Risk
10080 min Summer	26.712	0.712	0.0	36.5	Flood Risk
15 min Winter	26.317	0.317	0.0	7.2	O K
30 min Winter	26.377	0.377	0.0	10.3	O K
60 min Winter	26.434	0.434	0.0	13.6	O K
120 min Winter	26.488	0.488	0.0	17.1	O K
180 min Winter	26.517	0.517	0.0	19.3	Flood Risk
240 min Winter	26.537	0.537	0.0	20.7	Flood Risk
360 min Winter	26.564	0.564	0.0	22.9	Flood Risk
480 min Winter	26.584	0.584	0.0	24.6	Flood Risk
600 min Winter	26.600	0.600	0.0	25.9	Flood Risk
720 min Winter	26.612	0.612	0.0	27.0	Flood Risk
960 min Winter	26.632	0.632	0.0	28.8	Flood Risk
1440 min Winter	26.660	0.660	0.0	31.3	Flood Risk
2160 min Winter	26.686	0.686	0.0	33.9	Flood Risk
2880 min Winter	26.705	0.705	0.0	35.7	Flood Risk
4320 min Winter	26.728	0.728	0.0	38.2	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Time-Peak (mins)
7200 min Summer	1.391	0.0	7208
8640 min Summer	1.201	0.0	8648
10080 min Summer	1.063	0.0	10088
15 min Winter	125.869	0.0	27
30 min Winter	84.215	0.0	42
60 min Winter	53.779	0.0	72
120 min Winter	33.180	0.0	132
180 min Winter	24.659	0.0	192
240 min Winter	19.841	0.0	250
360 min Winter	14.545	0.0	370
480 min Winter	11.669	0.0	488
600 min Winter	9.828	0.0	608
720 min Winter	8.537	0.0	726
960 min Winter	6.830	0.0	964
1440 min Winter	4.978	0.0	1442
2160 min Winter	3.621	0.0	2152
2880 min Winter	2.885	0.0	2864
4320 min Winter	2.092	0.0	4284

Summary of Results for 100 year Return Period (+40%)

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Volume (m³)	Status
5760 min Winter	26.743	0.743	0.0	39.7	Flood Risk
7200 min Winter	26.753	0.753	0.0	40.8	Flood Risk
8640 min Winter	26.760	0.760	0.0	41.5	Flood Risk
10080 min Winter	26.765	0.765	0.0	42.1	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Time-Peak (mins)
5760 min Winter	1.663	0.0	5712
7200 min Winter	1.391	0.0	7136
8640 min Winter	1.201	0.0	8560
10080 min Winter	1.063	0.0	9896

Rainfall Details

Rainfall Model	FSR	Winter Storms	Yes
Return Period (years)	100	Cv (Summer)	0.750
Region	England and Wales	Cv (Winter)	0.840
M5-60 (mm)	19.000	Shortest Storm (mins)	15
Ratio R	0.360	Longest Storm (mins)	10080
Summer Storms	Yes	Climate Change %	+40

Time Area Diagram

Total Area (ha) 0.034

Time From:	**(mins) To:**	**Area (ha)**	**Time From:**	**(mins) To:**	**Area (ha)**	**Time From:**	**(mins) To:**	**Area (ha)**
0	4	0.012	4	8	0.011	8	12	0.011

Model Details

Storage is Online Cover Level (m) 26.800

Porous Car Park Structure

Infiltration Coefficient Base (m/hr)	0.00000	Width (m)	6.0
Membrane Percolation (mm/hr)	1000	Length (m)	58.0
Max Percolation (l/s)	96.7	Slope (1:X)	80.0
Safety Factor	2.0	Depression Storage (mm)	5
Porosity	0.30	Evaporation (mm/day)	3
Invert Level (m)	26.000	Membrane Depth (m)	1

APPENDIX V

SURFACE WATER ASSESSMNET INCLUDING FLOOD RISK ASSESSMENT

Calder Park

Tungsten Development. Surface Water Assessment.

Final Report

November 2020

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Revision History

Revision Ref / Date	Amendments	Issued to
13 November 2020	Draft Report	Mark Barwood
17 November 2020	Final Report	Mark Barwood

Contract

This report describes work commissioned by Mark Barwood, on behalf of Alan Wood and Partners, by email dated the 4 November 2020. Alan Wood and Partners' representative for the contract was Mark Barwood. Mark Bentley of JBA Consulting carried out this work.

Prepared by .... Mark Bentley BSc CEng CEnv FCIWEM C.WEM

Technical Director

Reviewed by .... Gavin Hodson BSc FdSc

Team Leader

Purpose

This document has been prepared as a Draft Report for Alan Wood and Partners. JBA Consulting accepts no responsibility or liability for any use that is made of this document other than by the Client for the purposes for which it was originally commissioned and prepared.

JBA Consulting has no liability regarding the use of this report except to Alan Wood and Partners.

Acknowledgements

The help with arranging the provision of data by Mark Barwood is gratefully acknowledged.

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JBA is aiming to reduce its per capita carbon emissions.

Contents

Appendices

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Abbreviations

AEP	Annual Exceedance Probability
DEP	Donaldson Edwards Partnership
FEH	Flood Estimation Handbook
FRA	Flood Risk Assessment
HEC-RAS	Hydrologic Engineering Center – River Analysis System
JBA	Jeremy Benn Associates
mAOD	Metres Above Ordnance Datum
SSU	Scientific Support Unit
SuDS	Sustainable Drainage System
WYP	West Yorkshire Police

1 Introduction

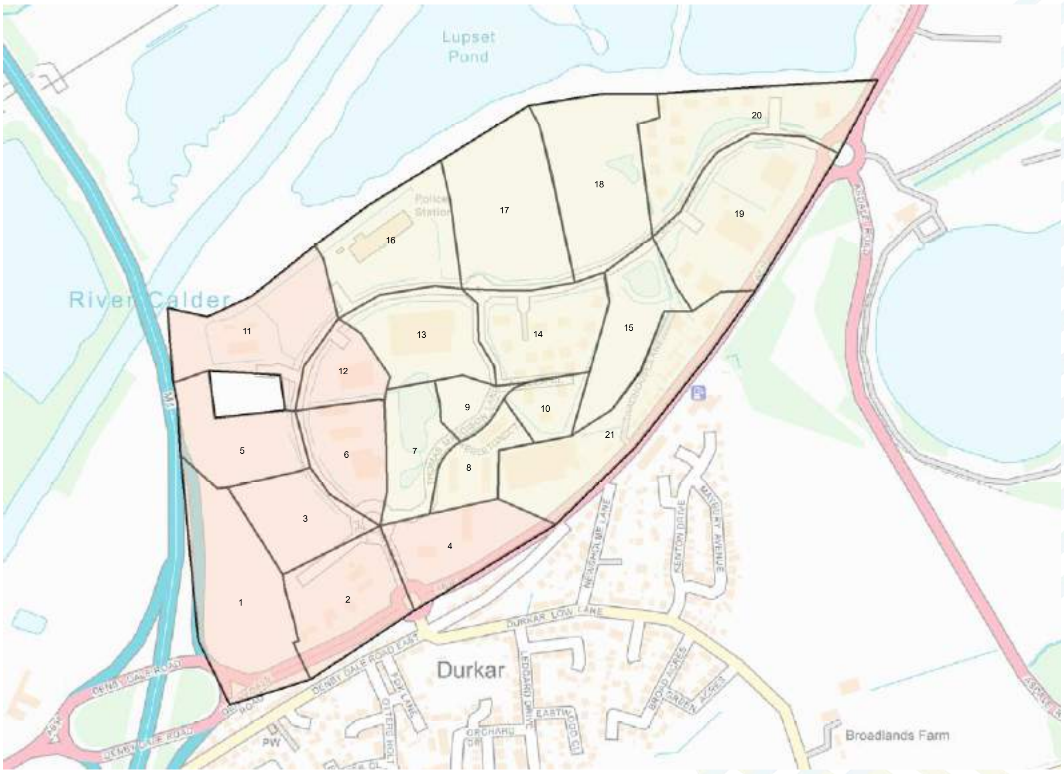
1.1 Flood risk assessment

The Calder Park development site is adjacent to Junction 39 of the M1. The site is bounded to the north and east by a flood embankment of the River Calder, to the west by the M1 motorway and to the south by Denby Dale Road. Between 2002 and 2018, JBA undertook work assessing the flood risk to the site and assisting with the design of the surface water drainage system. The surface water drainage system was designed to use a network of dry open channels (swales) and storage ponds to attenuate the runoff from the development plots.

There are proposals to develop a remaining plot of land at Calder Park. The proposed plot of land lies on the northern edge of Calder Park, just to the east of the Scientific Support Unit (SSU) building of West Yorkshire Police (WYP). The proposed platform level for the site will be set at 26.5mAOD. Appendix A contains drawings showing the proposed development. The total area of the plot of land for the new development is 60,700m^2 of which 51,950m^2 will comprise impermeable area. Therefore, 85% of the proposed will be impermeable which is greater than the value of 70% that was assumed for the design of the drainage system. This study has assessed the effect of increased runoff due to the greater impermeable area at the proposed site on water levels in the surface water drainage system.

JBA have re-used data previously collected on behalf of Peel Holdings, the developer of Calder Park. This includes topographical survey data showing ground levels and a computer model built for the business park surface water drainage system.

Figure 1-1 shows the Calder Park site. The proposed development will be in zones 17 and 18.



Not to scale

Figure 1-1: Calder Park location plan

1.2 Report structure

The report is presented in three sections:

1. Introduction – this section sets the study in context
2. Surface Water Runoff – flood risk from runoff within the Calder Park development
3. Conclusions

2 Surface water drainage

2.1 Surface water drainage system

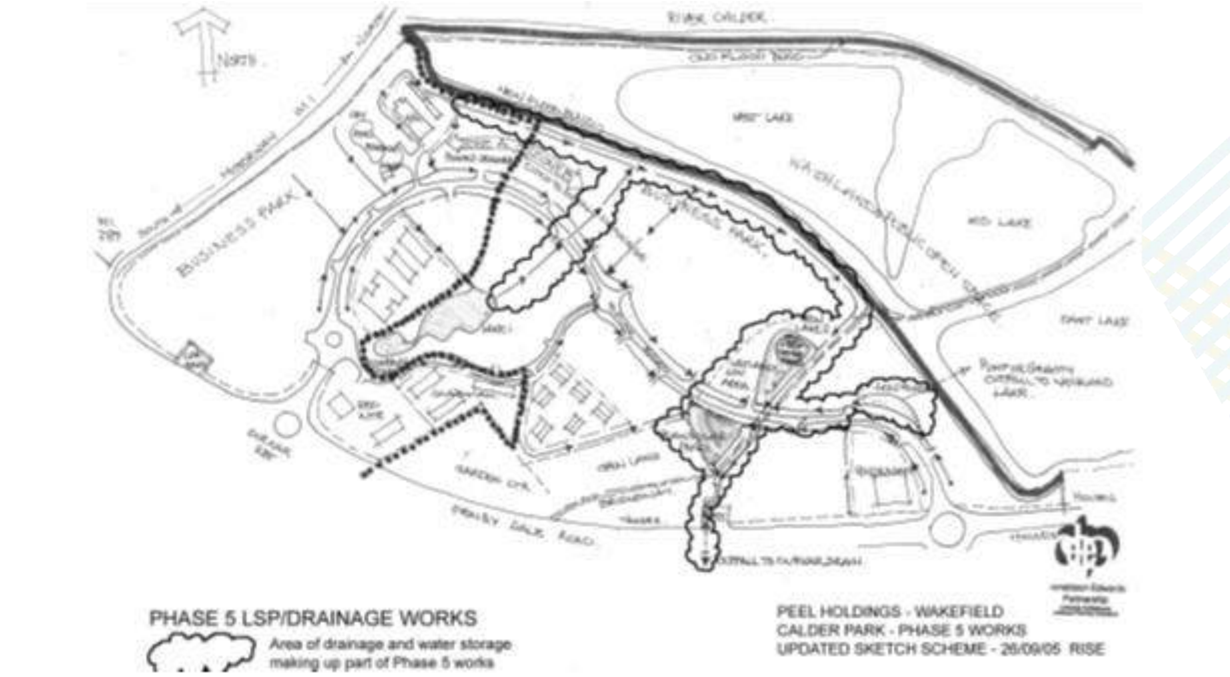
The drainage scheme for the Calder Park site utilises Sustainable Drainage System (SuDS) principles and is an integral part of the site design. The buildings, car parks and access roads are situated on raised platforms. A series of wide ditches (see Figure 2-1) run alongside the roads and between the building plots. There are also a few lakes that form part of the drainage scheme as well as improving the aesthetics of the site.

Figure 2-1: Typical swale at Calder Park

The site has been split into two surface water management zones, A and B (see Figure-2-2). Zone A utilises more traditional storm water drainage where runoff from roofs and roads is piped to a holding pond. This pond is then pumped directly to the River Calder at a rate of up to 1m^3/s. The system was designed such that there was sufficient storage in the pond to cope with runoff from a 100-year storm (including a 20% increase in rainfall intensity to allow for climate change).

Since the pond was designed the climate change allowance has been updated so that a 40% increase in rainfall intensity should be applied. Runoff calculations to the storage pond during a 1 in 100-year storm with an allowance of 40% increase in rainfall intensity show the pond level would rise to 26.0mAOD. That is 1m below the bank top (27.0mAOD). Therefore, the pond is large enough to deal with the revised allowances for climate change.

Runoff from all permeable areas and from subsequent plot developments (Zone B) goes directly to the ditches and is held in the land drainage system.



Note: Zone A is west of the dotted line and Zone B is to the east.

Figure 2-2: Calder Park drainage zones

The only outflow from the Calder Park site is to Durkar Drain. Water flows in a culvert (600mm diameter) under the Denby Dale Road. The ditch system at Calder Park connects to this culvert. The effective invert level of the culvert under the Denby Dale Road has been surveyed at 23.49mAOD. The lowest levels at Calder Park (in the north-east corner of the site) are lower than this. Therefore, it is not possible to drain the whole of Calder Park to Durkar Drain. A pumping station operates during high water levels to transfer water to the adjacent washland at a rate of 50l/s.

Surface water drainage for Calder Park has been calculated in three parts:

- Calculation of the volumes and rates of runoff during design events
- Hydraulic modelling of the ditch system to derive design water levels
- Calculations for the capacity of the culvert under Denby Dale Road.

2.2 Rainfall runoff calculations

Design rainfall depths for the site were calculated from the Flood Estimation Handbook (FEH). Two durations of storm were considered: 60-minutes and 7-hours for a 1%AEP (100-year return period) flood event.

Further consideration was given to the effects of climate change. The latest government guidance states that a 40% increase in rainfall should be used to account for climate change up to the year 2115 in the case of essential infrastructure .

A further storm profile comprising two consecutive 7-hour storms for a 3.3%AEP (30-year return period) was considered. This combination was used to assess the capacity of the surface water drainage system to recover from one storm before a further event occurs.

Table 2-1 summarises the design storms.

Table 2-1: Volume of storm rainfall

Storm Duration	3.3% AEP (30-year return period)			1% AEP (100-year return period)		
	Rainfall depth (mm)	Volume over site (m^3)	Peak storm intensity (mm/hr)	Rainfall depth (mm)	Volume over site (m^3)	Peak storm intensity (mm/hr)
60-minute				40	22,287	168
7-hour	90	50,298	18	62	34,234	24
with 40% allowance for climate change						
60-minute	Climate change scenario only applies to the 1% AEP (100-year flood event)			56	31,202	235
7-hour				87	47,928	34

For the purposes of calculating surface water runoff at Calder Park, the site was divided into 21 sub-catchments (see Figure 1-1). The sub-catchments represent different development plots across the Calder Park site. The calculations for each sub-catchment and the split between permeable and impermeable areas were adjusted based on plans supplied by WSP. For this study the percentage impermeable area of zones 17 and 18 was increased from 70% to 85%.

The division of the site enables the calculation of runoff volumes to be split between sites draining to the storage pond for pumping to the Calder and sites draining to the open ditches and eventual discharge to Durkar Drain. Storm water runoff volumes were calculated assuming that impermeable areas had a 100% runoff rate and 30% for permeable areas. Table 2-2 summarises volume of runoff for each design storm.

Table 2-2: Volume (m^3) of surface water runoff

Design scenario	Storm duration	Runoff destination	
		Pumped discharge	Durkar Drain
3.3%AEP	Two consecutive 7-hour storms	13,537	24,688
1%AEP (100-year return period)	1-hour	5,975	10,887
	7-hour	9,177	16,722
1% AEP with climate change	1-hour	8,365	15,242
	7-hour	12,848	23,411

Further runoff calculations were undertaken to assess the effect of saturated ground conditions which assumed the runoff from permeable areas was also 100%. Table 2.3 summarises the volume of runoff for this situation.

Table 2-3: Volume (m^3) of surface water runoff

Design scenario	Storm duration	Runoff destination	
		Pumped discharge	Durkar Drain
3.3%AEP	Two consecutive 7-hour storms	13,537	36,962
1%AEP (100-year return period)	1-hour	5,975	16,313
	7-hour	9,177	25,057
1% AEP with climate change	1-hour	8,365	22,838
	7-hour	12,848	35,080

There is a further discharge to the drainage system from the SSU building of WYP. The building is heated/cooled using an open loop groundwater heat extraction system. Depending on the energy demand this may result in peak flows of 18l/s to the ditch system. Over a 60-minute storm this would result in, at most, an extra 65m^3 of flow. Over a seven-hour storm the maximum runoff would be 454m^3 assuming the system operates at peak capacity throughout the storm.

Although the discharge from the SSU is small it is continuous and so there is the potential for the flow to reduce the capacity of the drainage ditches prior to a storm commencing. Therefore, the simulations of the drainage system have assumed the ditches are partially full due to continuous flow from the groundwater heat extraction system.

2.3 Hydraulic modelling

2.3.1 HEC-RAS model

Modelling of the drainage ditches was undertaken using HEC-RAS. HEC-RAS is an unsteady state one-dimensional river modelling package developed by the Hydrologic Engineering Center of the US Army Corps of Engineers. The software allows the user to calculate the variation of water surface within a channel network during a storm. The system can handle a looped network of channels, a branched system or just a single river reach. The model also allows the simulation of the effect of flood storage on routing flood flows. The model runs quickly and so was able to represent the draining down of the drainage system following an event.

The HEC-RAS model was originally set up with ditch profiles based on information supplied by Peel Holdings. The ditches are generally constructed by making a 1-in-3 slope down from the plot level to the proposed ditch invert level, cutting a flat bed 1.5m wide at this level and then making a 1 in 3 slope up to meet the opposite bank height. The actual ditch widths therefore depend on the adjacent ground heights and the ditch invert level.

Figure 2-3 shows the model for the completed ditch system.

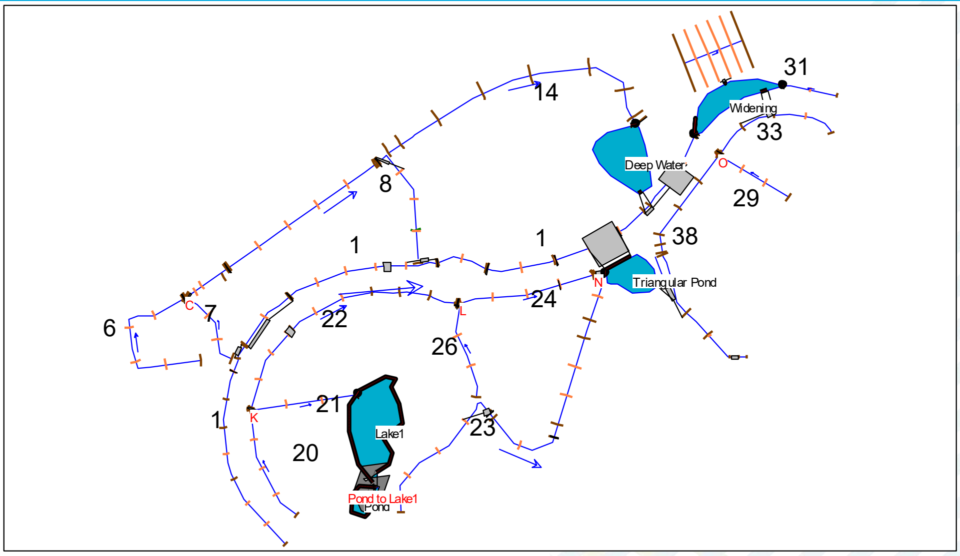


Figure 2-3: HEC-RAS schematic of completed drainage system

A combination of lateral weirs (sometimes incorporating culverts) and open channel junctions were used for connections between ditches and other ditches or lakes. To control water levels, retain the design standing water level and aid model stability (to prevent reaches becoming dry), a number of weirs were defined at intervals along the ditches. The weirs in the model were only defined where required to retain standing water levels.

Manning's *n* values (representing the roughness of the drains) were set to 0.03 at all locations. Since the drains are essentially a storage system (with flow controlled by weirs) the model results were expected to be insensitive to Manning's n values. However, sensitivity tests for this were undertaken (see Section 2.4.2). The model was run with a timestep of 6 seconds.

2.3.2 Standing water levels

At the start of a model simulation it is necessary to define the initial water levels in the ditches. In the case of the drainage system at Calder Park this is especially important because it will define the excess flood storage capacity in the system. The level of water in the ditches will depend on four factors:

- Drainage system design
- Groundwater levels
- Discharges from the SSU groundwater heat extraction system
- Recent storms, which have not fully drained away

To allow for high groundwater levels (see Chapter 4), the ditches have been designed to have a standing water depth of 0.5m with 1.5m depth in the lakes. Only ditches to the east of the Calder Park site would have deeper water (0.65m).

Initial conditions for the model simulations have been set assuming the groundwater heat extraction system is continuously discharging. This means that storage in the ditches at the start of the simulation has been partially used up before the storm begins.

Model simulations were also undertaken to assess the effect of two consecutive storms occurring. These tested the effect of high standing water levels that had not been able to drain out of the system before the onset of another storm. In this situation the standing water levels at the start of the storm are defined by the model following the simulation of runoff from the first storm.

2.4 Model results

The completed HEC-RAS model was run for the following three storms (all using realistic Runoff calculations – see Table 2-1), allowing outflow to Durkar Drain:

- 100-year, 60-minute Summer Storm
- 100-year, 7-hour Winter Storm
- Two consecutive 30-year, 7-hour Winter Storms

The tail-water level of the Denby Dale Road culvert was set at 25.2mAOD for the period of all the simulations. The flood level of 25.2mAOD was based on advice from Wakefield Council and represents the observed flood level on Denby Dale Road during the flood of the 25 June 2007. This is equivalent to a surcharged condition for the culvert and represents the case of impeded discharge from Durkar Drain to the River Calder (see section 2.5). Under these conditions there is backflow from Durkar Drain into the Calder Park site.

2.4.1 Water levels

The model simulations showed that generally across the site the two consecutive storms had the most impact on water levels in the drainage system. This is because high water levels at Denby Dale Road (25.2mAOD) prevent runoff escaping to Durkar Drain.

Therefore, water levels in the drainage system are sensitive to the runoff volume as opposed to the rainfall intensity.

This conclusion is different to that found during the original FRA in 2008. That study assumed a lower flood level at Denby Dale Road of 25.0mAOD and allowed water to leave Calder Park between consecutive storms.

Table 2-4 shows the highest water level in each ditch from each of the three simulations based on the final ditch layout with development completed. The ditch reach labels can be found on Figure 2-3. Simulated flood levels remained in bank throughout the ditch system.

Table 2-4: Maximum water levels in ditches - Completed drainage system

Upper site		Lower site	
Ditch	Maximum water level (mAOD)	Ditch	Maximum water level (mAOD)
1	26.28	23	25.76
6	25.28	24	25.62
7	25.29	26	25.62
8	25.33	29	25.22
14	25.31	31	25.22
20	26.18	33	25.22
21	26.18	38	25.22
22	26.18		

The effect of increased runoff from the site of the proposed development due to a greater impermeable area has a marginal impact on water levels. The only ditches where an impact was seen were ditches 29 and 38 where the water level increased by just 0.01m.

The proposed plot for the Tungsten Development will be set to 26.5mAOD. The maximum water level in the pond Deep Water and the ditches surrounding the development does not rise above 25.33mAOD. Therefore, there is over 1m freeboard.

2.4.2 Increased runoff (saturated ground conditions)

A sensitivity test with increased runoff to account for saturated ground conditions was undertaken. Runoff from the permeable areas of the site was assumed to be 100% (see section 2.2) and the SSU open loop groundwater heat extraction system was flowing at its maximum rate of 18l/s. This increased the total volume of runoff from the site and raised water levels in the ditches.

Table 2-5 shows the maximum water levels in the ditches for the completed system.

Table 2-5: Maximum water levels in ditches – 100% runoff

Upper site		Lower site	
Ditch	Maximum water level (mAOD)	Ditch	Maximum water level (mAOD)
1	26.60	23	25.83
6	25.47	24	25.83
7	25.58	26	25.83
8	25.47	29	25.24
14	25.36	31	25.24
20	26.23	33	25.24
21	26.23	38	25.24
22	26.23		

Generally, increasing the runoff has raised water levels across the site by the order of a few centimetres. The only significant impact on water level were in Ditches 1, 6, 7, 8, 24 and 26 where levels were raised by 0.32m (ditch 1), 0.29m (ditch 7), 0.14m (ditch 8), 0.21m (ditches 24 and 26). Even so, with saturated runoff, peak water levels were contained within bank.

Even with saturated runoff, the freeboard for the Tungsten Development in the pond Deep Water and the ditches surrounding the development is maintained at over 1m.

2.4.3 Climate change

A sensitivity test for the effects of climate change was also undertaken by applying a 40% increase in the volume of runoff. Table 2-6 shows the peak water levels in each ditch for the climate change scenario.

Table 2-6: Maximum water levels in ditches – Climate change

Upper site		Lower site	
Ditch	Maximum water level (mAOD)	Ditch	Maximum water level (mAOD)
1	26.36	23	25.81
6	25.33	24	25.78
7	25.35	26	25.78
8	25.41	29	25.24
14	25.39	31	25.24
20	26.23	33	25.24
21	26.23	38	25.24
22	26.23		

The impact of climate change was generally less than that due to saturated ground conditions and water levels were retained within the ditch system. For the climate change scenario, the freeboard for the Tungsten Development in the pond Deep Water and the ditches surrounding the development is maintained at over 1m.

2.4.4 Channel Roughness

The sensitivity of the model results to increased channel roughness was assessed by doubling the Manning's *n* coefficient from 0.03 to 0.06 across the site. There was very little sensitivity to Manning's *n* in the lower part of the site which is storage-dominated (only ditch 23 in this area showing an increase in water levels of 0.07m). A more widespread sensitivity was seen in the upper site where flux is more important, but most increases were less than 0.03m.

2.5 Durkar Drain

The outflow from Calder Park to Durkar Drain passes in culvert under the Denby Dale Road, then through a short section of open channel near a foul water pumping station and then into another culvert.

As part of the Wakefield Flood Defence Strategy developed by the EA, the outflow of Durkar Drain to the River Calder is flapped. It is expected that the flap will close for events greater than the 1-in-2-year flood on the River Calder. A further flap valve at the area known as 'Pugneys Entrance' closes during a 1-in-40-year flood event on the River Calder. This results in the Durkar Drain being 'tidelocked'. Durkar Drain is an ordinary watercourse and so is the responsibility of Wakefield Council.

Wakefield Council has purchased mobile pumps that can be deployed across their council area. Therefore, emergency pumps could be deployed to pump excess water from Durkar Drain to the River Calder during flood events. However, consultations with Wakefield Council indicate that this is not certain.

Therefore, the design runs for to test the surface water drainage system at Calder Park have made a conservative assumption that emergency pumps are not deployed to Durkar Drain. In this case water levels in Durkar Drain will backup and so a downstream level of 25.2mAOD has been applied. Under this scenario there will be backflow to the Calder Park site. In normal circumstances this situation should be avoided because of the risk of polluted water stagnating within the system of swales at Calder Park.

If emergency pumps are deployed, they should lower levels in Durkar Drain provided it is kept clear downstream of Denby Dale Road. With these lower levels (24.5mAOD) there would be no backflow on to the Calder Park site. In this case the drainage system would only be required to store runoff from Calder Park.

2.6 Culvert capacity

The culvert under the Denby Dale Road is a 600m diameter pipe. The invert level of the culvert is approximately 23.27mAOD. This information contrasts with a spot level survey (Donaldson Edwards Partnership (DEP), June 2005) which indicated that the invert at the culvert entrance is 23.49mAOD. This apparent discrepancy is probably caused by the large amount of siltation at the culvert. It is likely that the DEP level is the effective invert level due to siltation of the bed (i.e. 0.22m of silt blocking the bottom of the culvert). At the time WSP stated it was not possible to determine the invert of the pipe through the silt. The WSP information (from heights relative to footpath levels) indicated a downstream invert level of 23.22mAOD. The road is approximately 30m wide, which would give the culvert a slope of 1 in 500.

Surface runoff from Calder Park will not be restricted. Modelling (with a clear culvert) indicates that unrestricted flow leaving the site will exceed the agricultural runoff rates. This should not cause problems for Durkar Drain because in general rainfall on the site will enter Durkar Drain before the River Calder rises, 'tidelocking' the drain.

When the Durkar Drain is 'tidelocked', emergency pumps may be deployed by Wakefield Council to pump some of the flow that could be expected in Durkar Drain. If the culvert were totally blocked water levels could rise to 25.2mAOD, the boundary condition used to test the performance of the surface water drainage system at Calder Park (see 2.4).

3 Conclusions

3.1 Surface water drainage

The drainage scheme for Calder Park uses SuDS principles as an integral part of the site design. A series of wide ditches run alongside the roads and between the development platforms. There are also a number of lakes that form part of the drainage scheme as well as providing an aesthetic attraction.

Generally peak levels in the drainage system across the site were caused by consecutive storm events. This is because the model tests used high water levels at Denby Dale Road (25.2mAOD) that prevent runoff escaping to Durkar Drain. Therefore, water levels in the drainage system are sensitive to the runoff volume as opposed to the rainfall intensity.

Model simulations of the surface water drainage system have been undertaken for the proposed completed drainage system with a full developed business park. The simulations tested the impact of increased runoff from the proposed development on water levels in the surface water drainage system.

The effect of increased runoff from the site of the proposed development due to a greater impermeable area was shown to have a marginal impact on water levels. The maximum increase in water level of 0.01m was confined to just two ditches (number 29 and 38). However, simulated flood levels remained in bank throughout the ditch system.

With respect to the proposed Tungsten Development, flood levels in the Deep Water pond and surrounding ditches/ swales are of sufficient volume to prevent flooding of the Property. Under a 1 in 100-year event with an extra 40% allowance for climate change these is a freeboard of over 1m in the drainage system surrounding the site. Therefore, the surface water network can accommodate the free discharge of runoff from the Property based on an impermeable area of 51,950m^2.

3.2 Durkar Drain

The discharge of surface water from Calder Park is to Durkar Drain via a culvert under Denby Dale Road. As part of the Wakefield Flood Defence Strategy developed by the EA, the outflow of Durkar Drain to the River Calder flows through two flapped structures. As a results, during a 1-in-40-year flood event on the River Calder, Durkar Drain is 'tidelocked'. Durkar Drain is an ordinary watercourse and so is the responsibility of Wakefield Council.

Wakefield Council has purchased mobile pumps that can be deployed across their council area. Therefore, emergency pumps could be deployed to pump excess water from Durkar Drain to the River Calder during flood events. However, consultations with Wakefield Council indicate that this is not certain.

Therefore, design runs for this FRA have made a conservative assumption that emergency pumps are not deployed to Durkar Drain. In this case water levels in Durkar Drain will backup and so a downstream level of 25.2mAOD has been applied. Under this scenario there will be backflow to the Calder Park site. In normal circumstances this situation should be avoided because of the risk of polluted water stagnating within the system of swales at Calder Park.

If emergency pumps are deployed, they should lower levels in Durkar Drain provided it is kept clear downstream of Denby Dale Road. With these lower levels (24.5mAOD) there would be no backflow on to the Calder Park site. In this case the drainage system would only be required to store runoff from Calder Park.

Appendix

A Proposed development at Calder Park

DO NOT SCALE!
ALL DIMENSIONS SHOULD BE CHECKED ON SITE BEFORE WORK COMMENCES

BOUNDARY SUBJECT TO TITLE PLAN CHECK

SUBJECT TO REVIEW OF LEVELS AND FALLS

Rev.	Date	Description	Drawn
C	29.09.20	Office area amended to include SF. Areas updated to include gatehouse / control tower. Parking numbers added. Note for attenuation tank added.	DW
B	05.08.20	Sprinkler tanks moved	DW
A	04.08.20	Boundary amended inline with information from solicitors - drawing Calder Park - Ordnance Survey Mastermap. Site plan amended accordingly and detailing updated.	DW

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client
Tungsten

project
Calder Park, Wakefield

drawing title
Proposed Site Plan

date August 2020
status Feasibility
scale 1:500 @ A0
drawn DW checked
job no. 2479 dwg no. F404 rev. C

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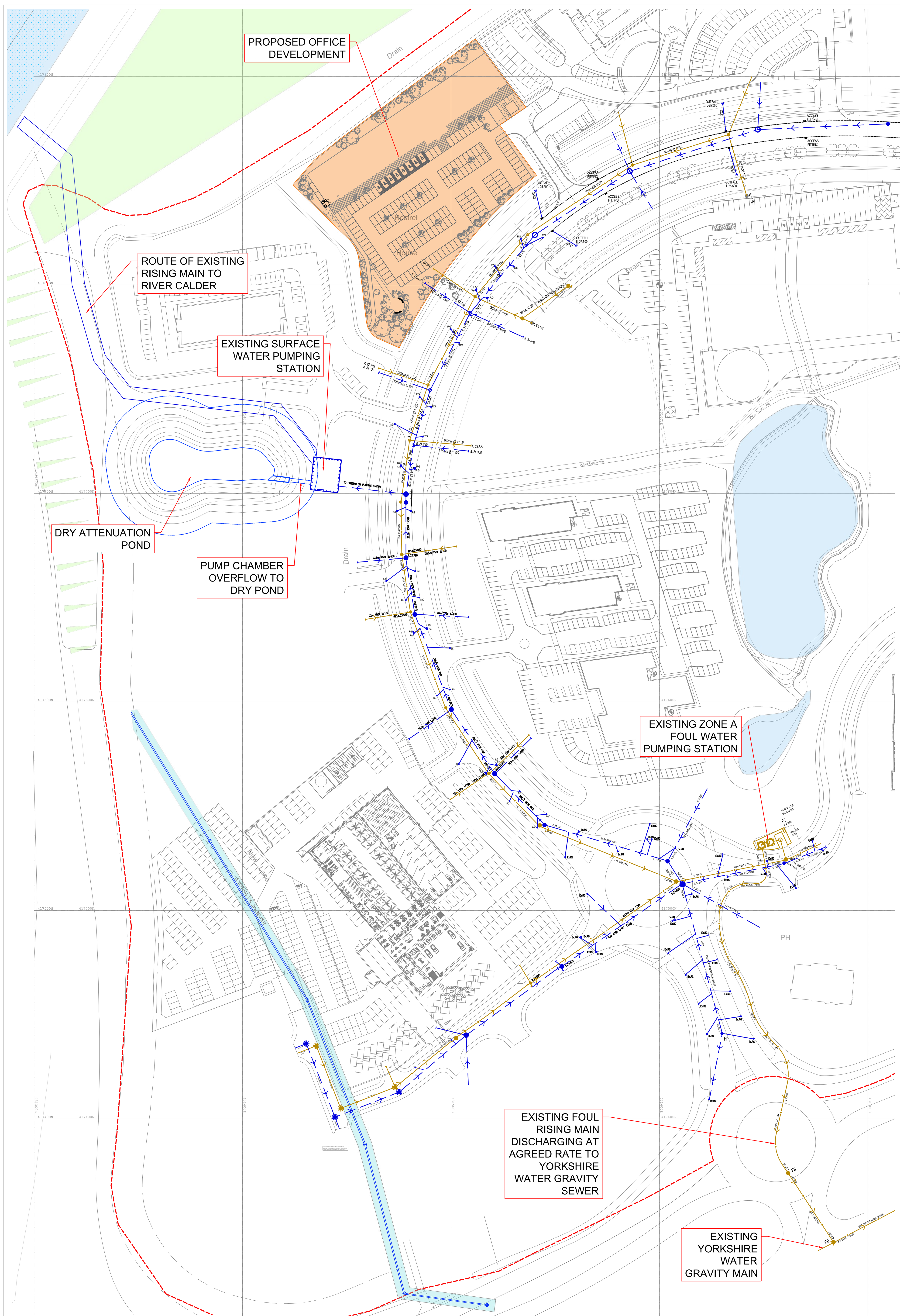
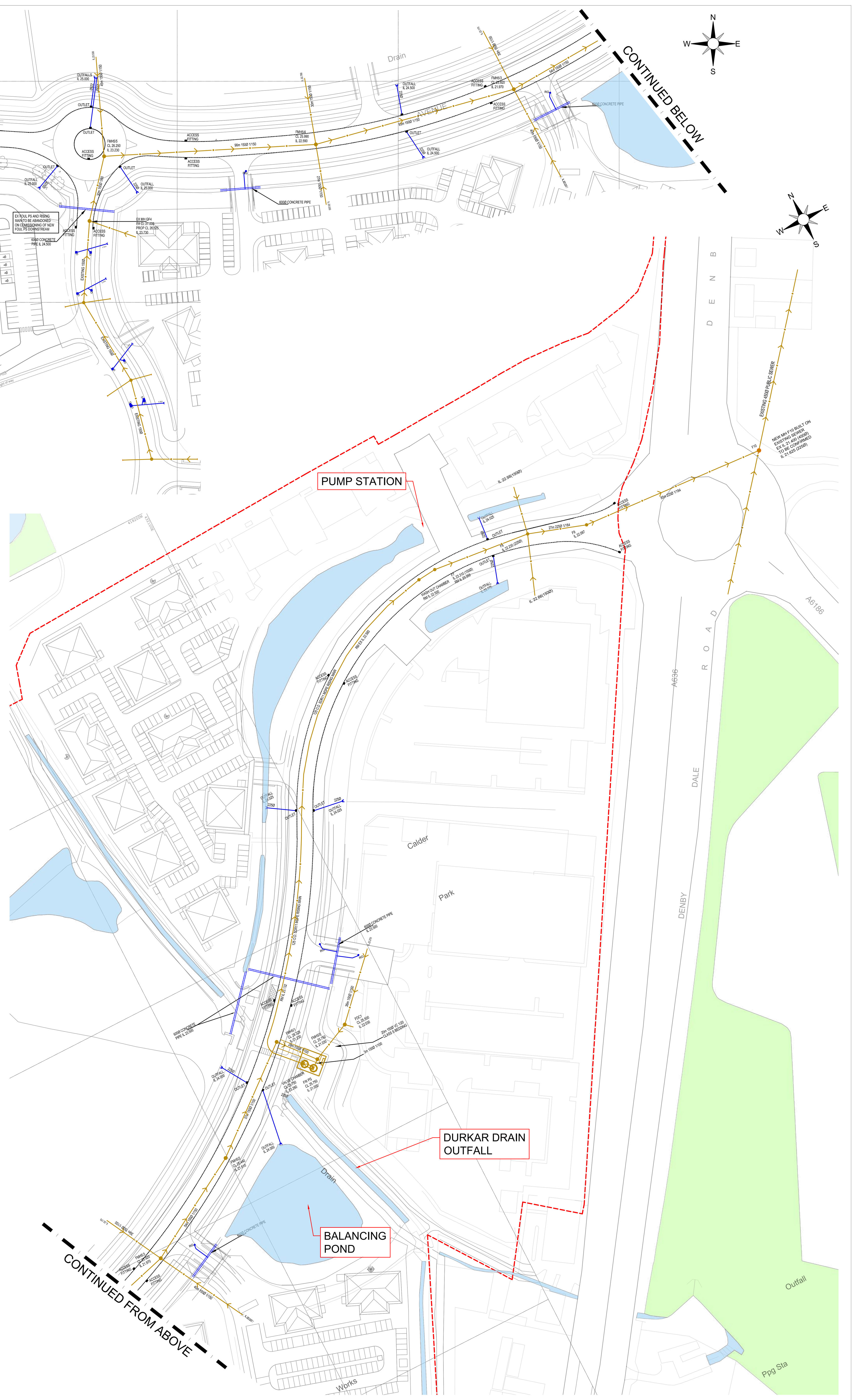
APPENDIX VI

EXISTING FOUL CONNECTION DRAWING



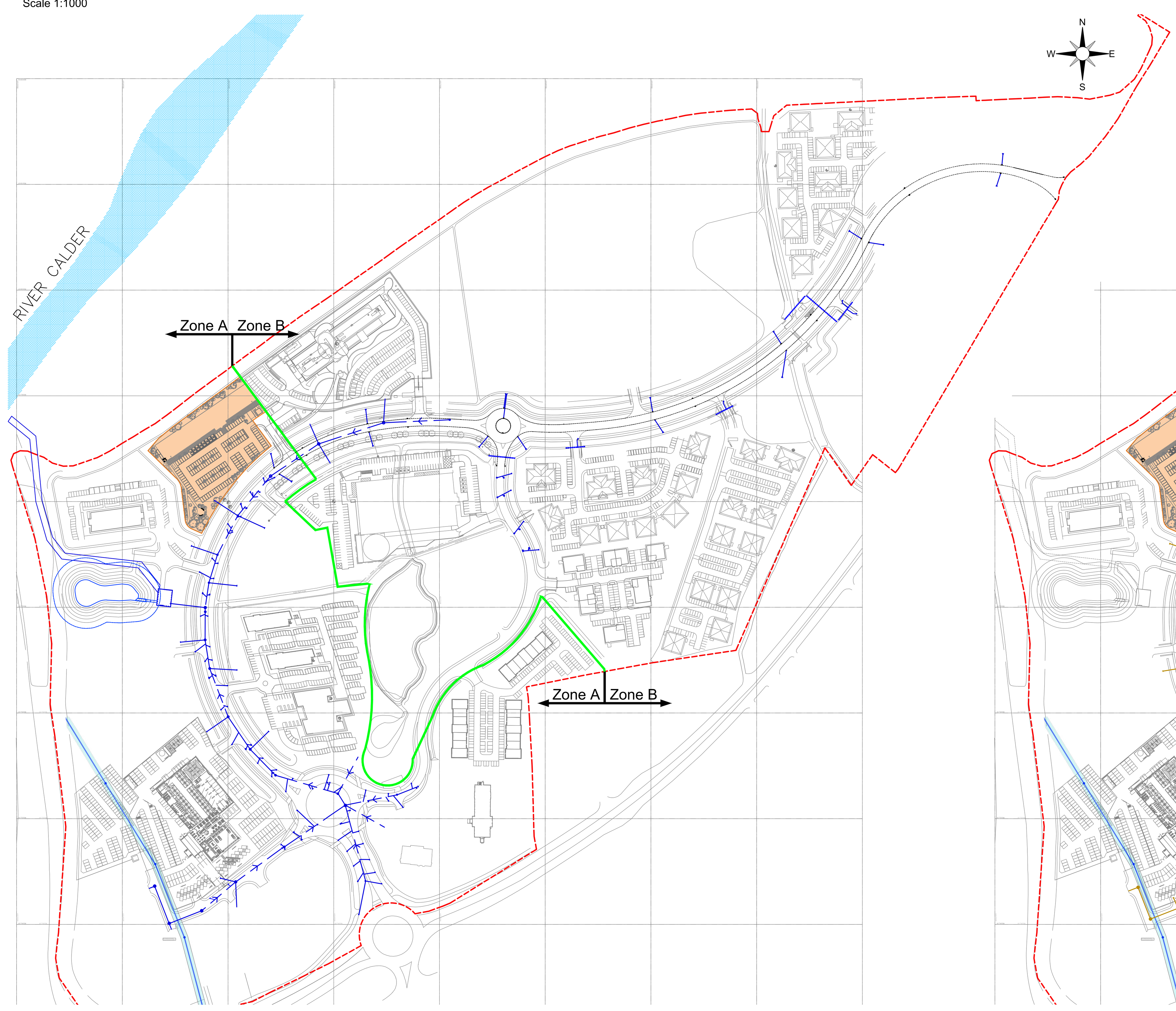
Drainage Layout - West

Scale 1:1000



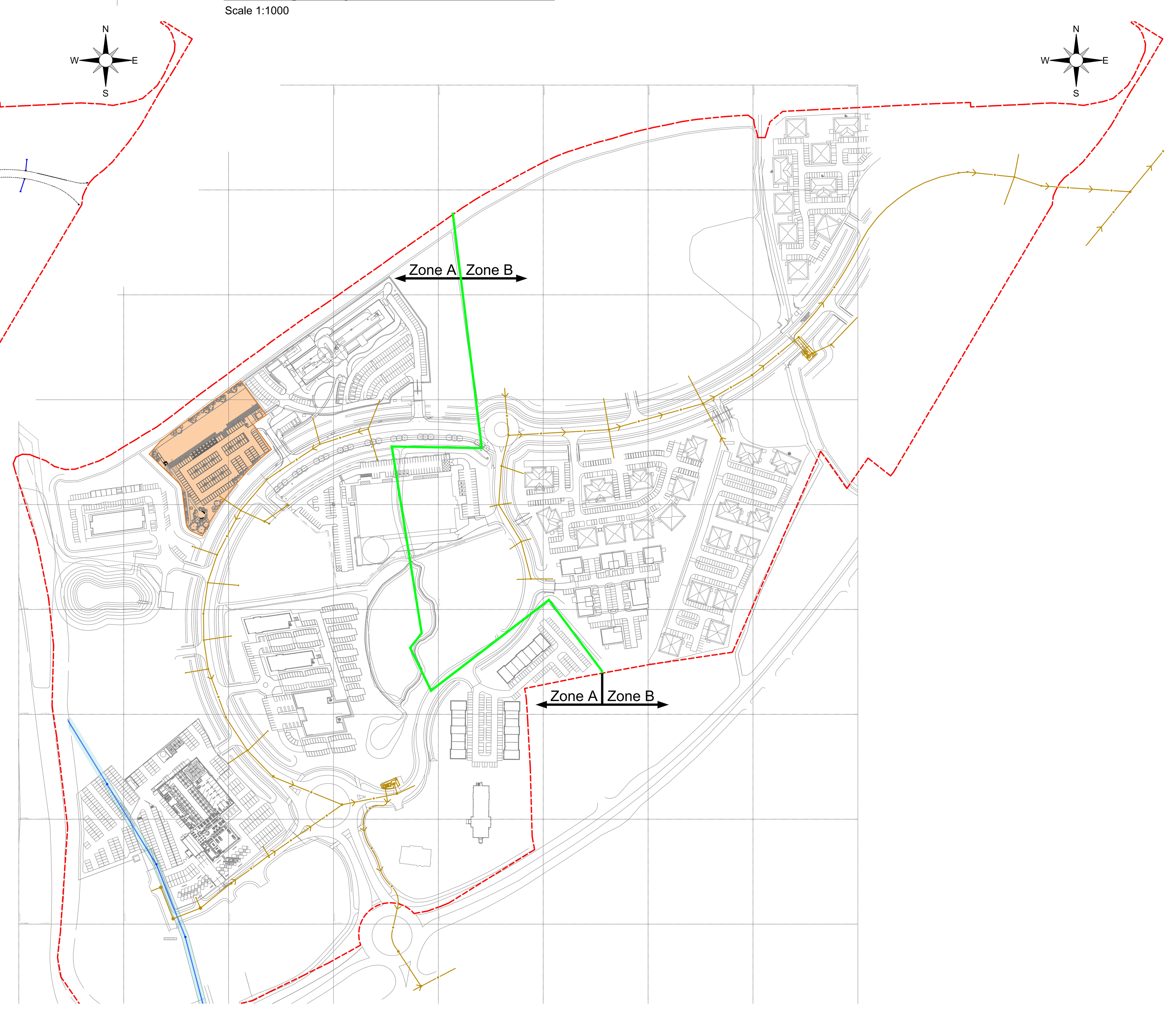
Drainage Layout Continued - East

Scale 1:1000



Indicative surface water boundary between Zone A & Zone B

Scale 1:2500



Indicative foul water boundary between Zone A & Zone B

Scale 1:2500

NOTES:

1. THESE NOTES ARE INTENDED TO AUGMENT DRAWINGS AND SPECIFICATIONS. WHERE CONFLICT OF REQUIREMENTS EXIST THE ORDER OF PRECEDENCE SHALL BE AS SHOWN IN THE SPECIFICATION. OTHERWISE THE STRICTEST PROVISION SHALL GOVERN.
2. THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ALL OTHER RELEVANT ENGINEERS AND ARCHITECTS DRAWINGS.
3. DRAWINGS NOT TO BE SCALED. ALL DIMENSIONS TO BE CHECKED ON SITE BY THE CONTRACTOR. ANY DISCREPANCIES TO BE NOTIFIED TO THE ENGINEER AND FURTHER INSTRUCTIONS OBTAINED BEFORE WORK IS COMMENCED.
4. THE STRUCTURE IS DESIGNED TO BE SELF-SUPPORTING AND STABLE AFTER THE BUILDING IS FULLY COMPLETED. IT IS THE CONTRACTORS SOLE RESPONSIBILITY TO DETERMINE THE ERECTION PROCEDURE AND SEQUENCE AND ENSURE THAT THE BUILDING AND ITS COMPONENTS ARE SAFE DURING ERECTION. THIS INCLUDES THE ADDITION OF WHATEVER TEMPORARY BRACING, GUYS OR TIE-DOWNS WHICH MAY BE NECESSARY, SUCH MATERIAL REMAINING THE THE PROPERTY OF THE CONTRACTOR ON COMPLETION, AND FOR ENSURING THAT THE WORKS AND ANY ADJACENT PROPERTIES ARE SAFE IN THE TEMPORARY CONDITION.

100mm at A0

Legend
- Existing SW
- Existing FW
- Zone A / Zone B Drainage Boundary

Rev	Description	Date	By	Chk	App
P02	EASTERN AREA ADDED	13.12.19	NB	MB	MB
P01	FIRST ISSUE	13.11.19	NB	MB	MB

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Project:	Calder Park - Zone 1 Earth Works / Landrover Enabling Works, Wakefield
Client:	Peel Investments (North) Ltd
Drawing:	Overall Calder Park Drainage GA Layout (Surface & Foul Water)
Role:	Civil
Drawing Status:	PRELIMINARY
Job. no.	42499
Scale@ A0:	As Noted
Rev.	P02

Project - Originator - Volume - Level - Type - Role - Number

42499 - AWP - XX - XX - DR - C - 2150

APPENDIX VII

MAINTENANCE AND MANAGEMENT PLAN

MAINTENANCE AND ACTION SCHEDULE FOR SURFACE WATER DRAINAGE

at

Peel Avenue, Calder Park, Wakefield

Project No:	**63150**	Prepared by:	PRP Catherine House Old Harboorough Road Brixworth Northampton NN6 9BX
For:	Peel Avenue, Calder Park		
Date:	November 2020		

1. Manholes and inspection chambers should be regularly inspected and debris/silt removed, if this is not removed then it is likely to become hard packed requiring considerable effort to remove it.

2. The following are guidelines for when inspections and treatment should be carried out based on a typical private surface water drainage system and foul water drainage system. The rate of silt and debris accumulation should be monitored and the frequency of inspection may need to be adjusted based on this.

   2.1. Monthly:

   - Hard surfacing should be swept regularly to prevent silt being washed into the surface. This will minimise necessary maintenance of the permeable surface. (every two months during Spring and Summer)
   - Approved herbicide to be applied to permeable surface to prevent foliage growing and blocking voids. (every two months during Spring and Summer)
   - Lift all final outflow manhole cover and inspect to make sure that the outfall and inlet are clear. (every two months during Spring and Summer)
   - Remove litter (including leaf litter) and debris from surface of permeable paving and access chambers.

   2.2. Annually:

   - Inspect all gutters and gullies for sediment and debris and remove as necessary to prevent it from entering into the attenuation tank.
   - Any roots that have entered the system should be removed.
   - Inspect manholes and remove any silt or debris from base and ensure that they are clean.
   - Any residual weeds to be removed manually.

   2.3. As required:

   - Jet all pipework: perforated and solid wall.
   - Power wash permeable surface and relay with new permeable stone tank beneath, if required.

Maintenance Schedule	Required Action	Typical Frequency
Regular Maintenance	Inspect all gutters and gullies for sediment and debris and remove as necessary to prevent it from entering into the system	Annually
	Any roots that have entered the system should be removed	Annually
	Roads and all car parking spaces should be swept regularly to prevent silt being washed into the surface. This will minimise maintenance	Monthly during the Autumn and Winter but to be assessed on the site
Occasional Maintenance	Inspect manholes and other chamber and remove any silt or debris from base and ensure that they are clean	Annually
Remedial Actions	Reconstruct permeable paving and/or replace if performance deteriorates of failure occurs	As required
Monitoring	Inspect silt traps and note rate of sediment accumulation	Monthly in the first year and then annually
	Check outfalls to ensure all is working correctly	Annually

Catchpits, manholes and inspection chambers should be regularly inspected and debris/silt removed, if this is not removed then it is likely to become hard packed requiring considerable effort to remove it. Replacement of the coarse granular aggregate and the membrane will be necessary if the system becomes blocked with silt. Effective monitoring will give information on changes in infiltration and provide a warning of potential failure in the long term.

APPENDIX VIII

GENERAL CONDITIONS

1. This report has been prepared and written specifically for the Client named in the introduction and is exclusively for their benefit. No reliance may be placed in the contents of this report by any third party except with the express agreement of the original Client and the written agreement of PRP. Such written agreement may require the payment of an additional fee.

2. The 'Recommendations' section of this report only provide an overview of the guidance and should not be specifically relied upon in their own right but should be considered in relation to the whole report and the development described in this report.

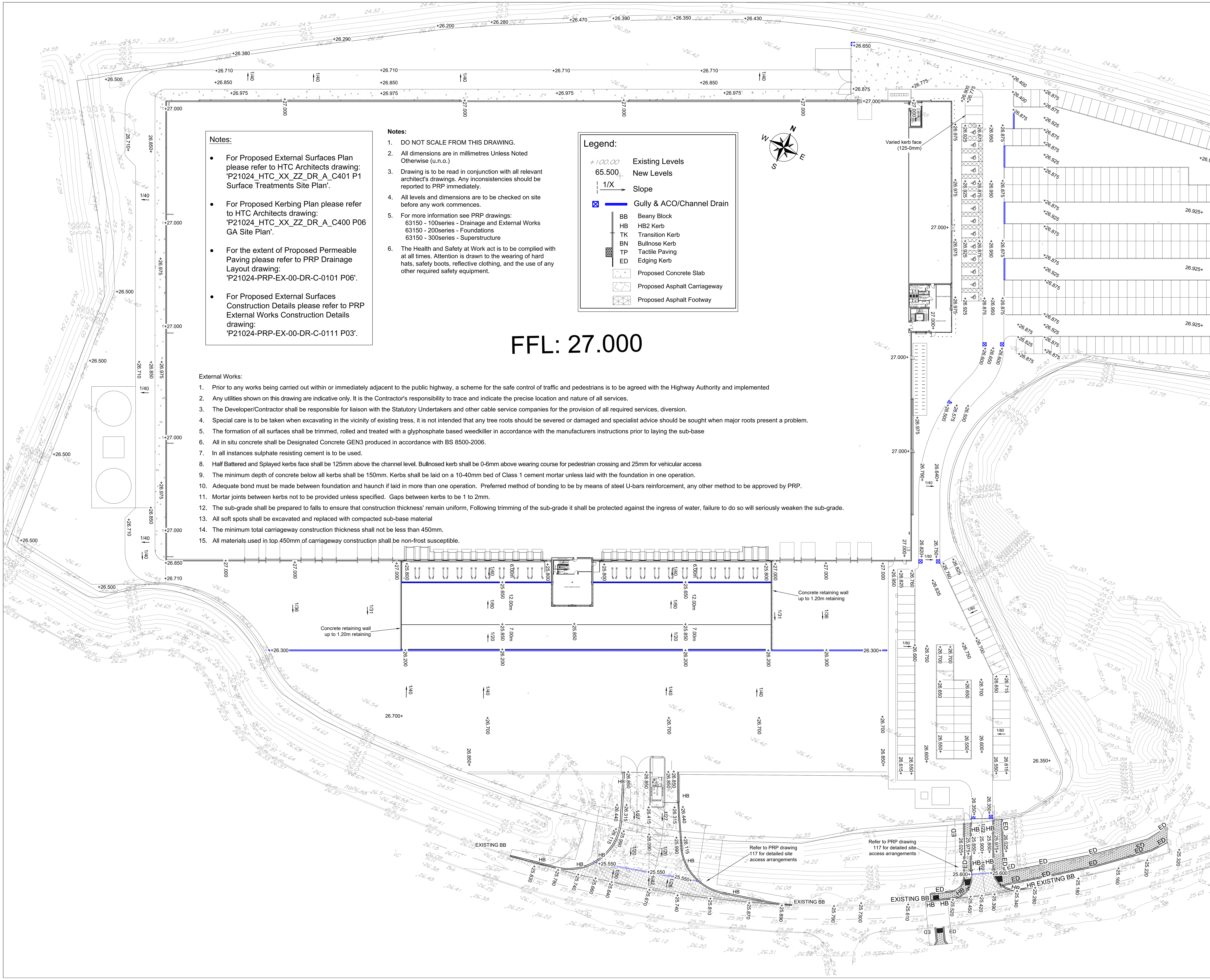
3. This report has been prepared and written in the context of the proposals for the development of the site as stated by the Client and will not be valid in a differing context. Furthermore, new information, improved practices, or legislation may necessitate alterations to the report in whole or in part after its submission. Therefore, with any change in circumstances or after the expiry of one year from the date of this report, it should be referred to us for re-assessment.

4. Any assessments made in this report are based on the ground conditions and as revealed by the test pits and boreholes undertaken by others and where appropriate other relevant data which may have been obtained for the site. The sources of such information are detailed in this report and while PRP use only such sources as are believed to be reliable, PRP will not be liable for the authenticity or reliability of information obtained from others.

5. Where data has been provided or is made available to PRP and this has been used in the report, it has been assumed that the information is correct. No responsibility can be accepted by PRP for inaccuracies within the data supplied.

6. The copyright in this report and other related plans and documents prepared by PRP is owned by them and no such report, plan or document may be reproduced, published or adapted without their written consent. Complete copies of the report may however be made and distributed by the Client as an expedient in dealing with matters related to its commission.

7. This report has been prepared solely for the Client's purposes in obtaining planning permission related to the proposed development indicated in the report.



Notes:

- For Proposed External Surfaces Plan please refer to HTC Architects drawing: 'P21024_HTC_XX_ZZ_DR_A_C401 P1 Surface Treatments Site Plan'.
- For Proposed Kerbing Plan please refer to HTC Architects drawing: 'P21024_HTC_XX_ZZ_DR_A_C400 P06 GA Site Plan'.
- For the extent of Proposed Permeable Paving please refer to PRP Drainage Layout drawing: 'P21024-PRP-EX-00-DR-C-0101 P06'.
- For Proposed External Surfaces Construction Details please refer to PRP External Works Construction Details drawing: 'P21024-PRP-EX-00-DR-C-0111 P03'.

Notes:

1. DO NOT SCALE FROM THIS DRAWING.
2. All dimensions are in millimetres Unless Noted Otherwise (u.n.o.)
3. Drawing is to be read in conjunction with all relevant architect's drawings. Any inconsistencies should be reported to PRP immediately.
4. All levels and dimensions are to be checked on site before any work commences.
5. For more information see PRP drawings:
   63150 - 100series - Drainage and External Works
   63150 - 200series - Foundations
   63150 - 300series - Superstructure
6. The Health and Safety at Work act is to be complied with at all times. Attention is drawn to the wearing of hard hats, safety boots, reflective clothing, and the use of any other required safety equipment.

Legend:

+100.00	Existing Levels
65.500	New Levels
1/X	Slope
	Gully & ACO/Channel Drain
BB	Beany Block
HB	HB2 Kerb
TK	Transition Kerb
BN	Bullnose Kerb
TP	Tactile Paving
ED	Edging Kerb
	Proposed Concrete Slab
	Proposed Asphalt Carriageway
	Proposed Asphalt Footway

FFL: 27.000

External Works:

1. Prior to any works being carried out within or immediately adjacent to the public highway, a scheme for the safe control of traffic and pedestrians is to be agreed with the Highway Authority and implemented
2. Any utilities shown on this drawing are indicative only. It is the Contractor's responsibility to trace and indicate the precise location and nature of all services.
3. The Developer/Contractor shall be responsible for liaison with the Statutory Undertakers and other cable service companies for the provision of all required services, diversion.
4. Special care is to be taken when excavating in the vicinity of existing tress, it is not intended that any tree roots should be severed or damaged and specialist advice should be sought when major roots present a problem.
5. The formation of all surfaces shall be trimmed, rolled and treated with a glyphosphate based weedkiller in accordance with the manufacturers instructions prior to laying the sub-base
6. All in situ concrete shall be Designated Concrete GEN3 produced in accordance with BS 8500-2006.
7. In all instances sulphate resisting cement is to be used.
8. Half Battered and Splayed kerbs face shall be 125mm above the channel level. Bullnosed kerb shall be 0-6mm above wearing course for pedestrian crossing and 25mm for vehicular access
9. The minimum depth of concrete below all kerbs shall be 150mm. Kerbs shall be laid on a 10-40mm bed of Class 1 cement mortar unless laid with the foundation in one operation.
10. Adequate bond must be made between foundation and haunch if laid in more than one operation. Preferred method of bonding to be by means of steel U-bars reinforcement, any other method to be approved by PRP.
11. Mortar joints between kerbs not to be provided unless specified. Gaps between kerbs to be 1 to 2mm.
12. The sub-grade shall be prepared to falls to ensure that construction thickness' remain uniform, Following trimming of the sub-grade it shall be protected against the ingress of water, failure to do so will seriously weaken the sub-grade.
13. All soft spots shall be excavated and replaced with compacted sub-base material
14. The minimum total carriageway construction thickness shall not be less than 450mm.
15. All materials used in top 450mm of carriageway construction shall be non-frost susceptible.

Rev	Date	Description	By / Chk
C01	26/11/2021	Amendments to site accesses	ST / JM
P07	15/11/2021	Porous car park simplified, levels added	ST / JM
P06	25/10/2021	Latest Architect's Site Plan added. Proposed Levels updated. Levels added around accesses.	SK / JM
P05	03/09/2021	Site plan updated to Architects latest layout. Proposed levels updated accordingly.	SK / JM
P04	12/07/2021	External works levels updated.	SK /BMS
P03	17/06/2021	dwg 0103 changed to 0110, BIM nos added	KMV/BMS
P2	12/02/2021	Bund levels added as land architect dets..	BMS/BMS
P1	15/12/2020	Issued for Comment.	SK /BMS

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Client: TUNGSTEN PROPERTIES LTD

Architect: HTC Architects

Project: Calder Park, Wakefield, WK4 3FL

Title: External Works

Status: **CONSTRUCTION**

Engineer:	SK	Date:	Nov 2020
Drawn:	SK	Scales @ A1:	1:500
Checked:	BMS		
PRP Ref No:	63150	Software:	2D AutoCAD 2010
Drg No:	P21024-PRP-EX-00-DR-C-0110	Rev:	C01