Gatwick Airport Northern Runway Project

Environmental Statement
Appendix 11.9.7: Wastewater Assessment

Book 5

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0 Executive Summary

0.1.1 The proposal to make best use of London Gatwick Airport's existing runways and infrastructure will increase the number of passengers and staff at the airport that will increase the volume of wastewater produced requiring treatment prior to discharge.

0.1.2 This appendix provides the technical information that supports the assessment of impact of the potential increase in wastewater volumes reported in **Environmental Statement Chapter 11: Water Environment** (Doc Ref 5.1).

0.1.3 The assessment demonstrates that with the provision of new infrastructure as part of the Project, Gatwick's network can safely cope with the additional wastewater flows. A response from Thames Water regarding the ability of their infrastructure to convey and treat the increased flows is awaited, but to date no indication of impediment has bene received by GAL.

1 Introduction

1.1 General

1.1.1 This document forms **Environmental Statement (ES) Appendix 11.9.7** (Doc Ref 5.3) of the prepared on behalf of Gatwick Airport Limited (GAL) for the proposal to make best use of London Gatwick Airport's (Gatwick) existing runways and infrastructure (referred to within this report as 'the Project').

1.1.2 This document provides the background to the assessment of impacts and effects of the Project upon the GAL-owned wastewater infrastructure.

1.1.3 Separate figures are included with this appendix that provide details of the Gatwick wastewater network, specific asset location plans are included within the main body of this appendix.

1.1.4 The local sewerage undertaker Thames Water (TW), as part of their long-term planning, will undertake an assessment of the impact of wider projected growth in the local area on their sewage treatment works at Horley and Crawley, which would include the impact of the Project.

1.2 Purpose of this Report

1.2.1 The expansion of passenger numbers at Gatwick as a result of the Project would result in an increase in wastewater flows. A study has been undertaken to construct a wastewater hydraulic model of the foul network, calibrate it, use it as a tool for assessing the current performance, then as an aid to planning the provision of foul drainage and to inform the assessment of the environmental effects of the Project as reported in the **ES Chapter 11: Water Environment** (Doc Ref. 5.1).

1.2.2 The scope was as follows:

- Construct hydraulic model of the foul network serving the North and South Terminal catchment areas
- Calibrate the model against observed data
- Characterise system capacity
- Identify consequence of increasing demand due to projected Baseline growth and as a result of the Project
- Formulate optimised strategies to meet demand
- Recommend improvements to the system

2 Catchment Description

2.1 Catchment

2.1.1 The wastewater network for Gatwick is split into two main catchments:

- The North Terminal area including the terminal building, cargo area, the fuel farm, the engineering site, hangars and fire training ground plus hotels and a vehicle fuel service station; and
- The South Terminal area including the terminal building, the railway station, offices, hotels and car parking and car hire facilities plus two fast food outlets and a hotel in the North Terminal area.

2.1.2 **ES Appendix 11.9.7 Figure 2.1.1** (Doc Ref. 5.3) shows the North and South terminal catchments.

2.1.3 Flows discharging into the system are primarily domestic foul discharges from passengers and workers, but there are a significant number of restaurants of various types within the terminals and two fast food establishments on the east side of the South Terminal, which would discharge food preparation and cleaning water plus fat and grease. There are also domestic type foul flows discharged from incoming passenger aircraft which discharge into a sanitary block to the west of the North Terminal; there is a similar facility to the south of the South Terminal which is not currently in use. In addition, there are also trade effluent discharges, primarily from the aircraft washing plant on the west side of the airport and the car hire centres to the east of the South Terminal (car washing), plus several other minor discharges in the servicing areas.

2.1.4 In addition to foul flows, there is a road area south of the fuel farm which has gullies connected and which discharges storm runoff into the foul system, and there are also base infiltration flows due to minor leaks in the piped network.

2.2 Foul sewer network

Data Sources

2.2.1 The following data was used to construct the wastewater model:

- GIS data of the foul drainage system;
- Schematics of the network;
- Foul system survey data supplied by Mason Land Surveys Limited reporting on a manhole survey conducted on an unknown date;
- Foul and storm system survey data supplied by Engineering Surveys Limited reporting on a manhole survey conducted in 1996;
- Survey data gathered during an investigation for the Bloc Hotel development in the South Terminal;
- Observed flow, depth, pump operation data and reports from GAL's Andover control system;
- Manhole inspection data gathered during the flow survey pre-inspections; and
- Limited drawings and data on selected pumping station, water supply and fuel hydrant assets.
- 1996/97 Unistride manhole survey data

2.2.2 The schematic for the network is shown in Figure 2.2.1.

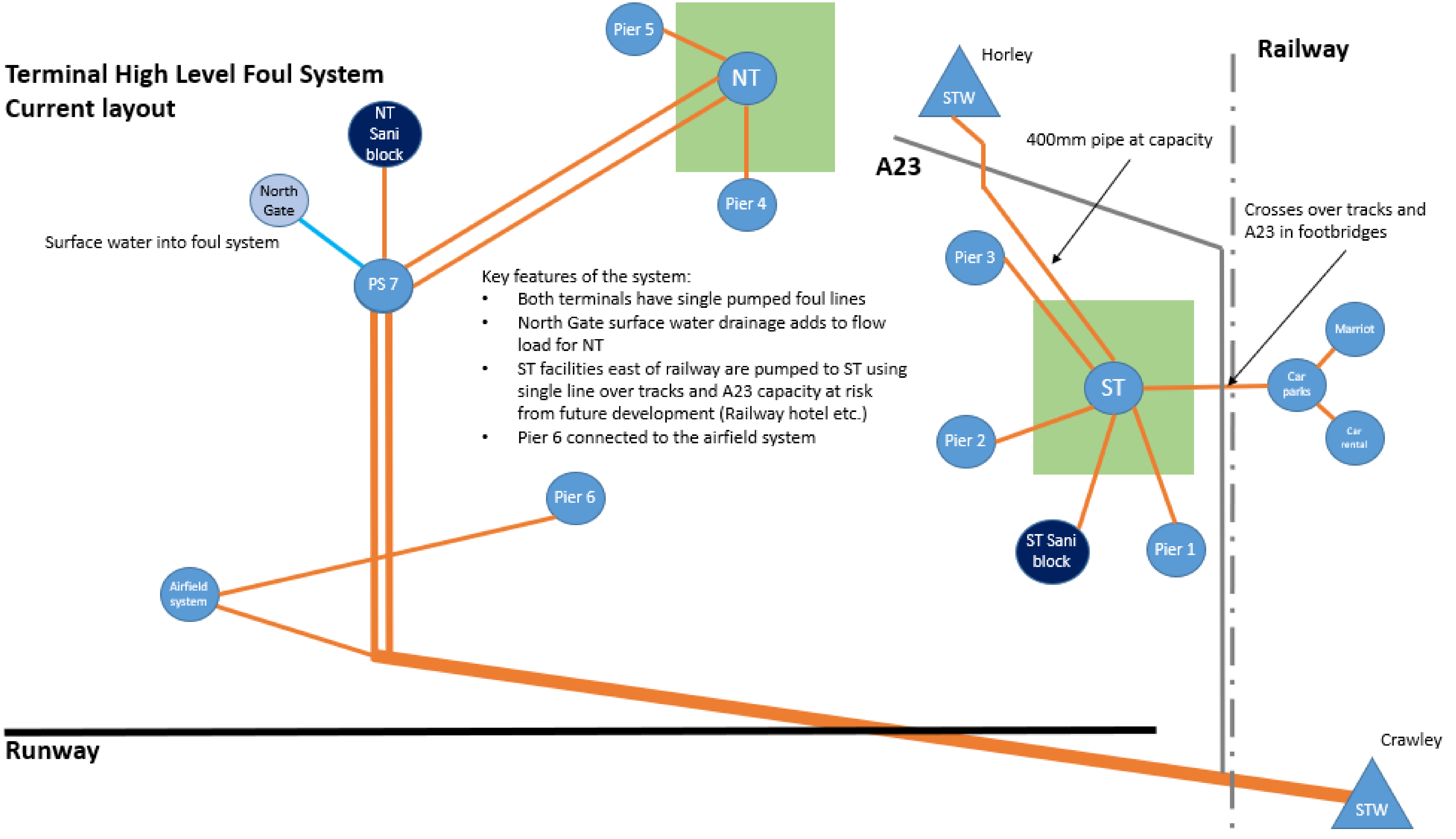


Figure 2.2.1 Foul Network Schematic showing the discrete North Terminal (NT) and South Terminal (ST) catchments

North Terminal System

2.2.3 The head of the North Terminal wastewater system on the east side is a small gravity network serving the Premier Inn, the Contractor Support Centre and adjacent Shell garage. This is pumped by Pumping Station (PS) 10 to the terminal gravity system which serves the Hampton Hilton Hotel, baggage centre and the north part of the terminal. A minor pumping station PS11 discharges low flows from the south part of the terminal into a trunk gravity sewer serving Pier 4, the Sofitel Hotel, Jubilee House and the south part of the terminal building. Both gravity systems discharge via 225mm or 300mm diameter pipes into PS8 which is currently off-line with a temporary replacement pumping system in operation in the upstream manhole. This discharges to a gravity network serving the sanitation block and fuel farm, and discharges into terminal pumping station PS7 via a 300mm diameter pipe. PS7 also receives flow from the cargo sheds, via PS6. Furthermore, it has been found that highway drains in Timberham Farm Road discharge surface water into the gravity system discharging to PS7.

2.2.4 The second major branch of the North Terminal system serves the fire training ground at the extreme west end of the airfield via PS45, the Hangar 7 complex, Central Area Recycling Establishment and motor transport facilities – some via minor pumping stations PS4 and PS5 - into terminal pumping station PS3. In addition, flows from Pier 6 of the terminal also discharge to PS3 via PS44 which discharges to PS2 where additional flows are conveyed to PS3 from the control tower and airfield lighting building. None of the gravity sewers serving this area exceed 225mm diameter. PS7 discharges via twin rising mains to the TW trunk sewer running east along the A23 south of the airport. PS3 injects flow into the westerly rising main from PS7 but generally does not operate at the same time as the PS7 pumps. The TW gravity sewer discharges to Crawley Sewage Treatment Works on the east side of the railway, south of the airport land. All pumping stations are of the submersible types with pumps located in wet wells. The connectivity is shown in **ES Appendix 11.9.7 Figure 2.2.2** (Doc Ref. 5.3) .

2.2.5 Details of the existing pumping station critical assets and their contributing areas are given in Table 2.2.1: the pumping rates are those derived from the Andover reports.

South Terminal System

2.2.6 The main gravity system serving the South Terminal starts at the currently disused sanitation block to the south of the terminal on the west side of the railway line and routes along the service road, northwards as a 225mm diameter pipe: it collects flows from the boiler house and Concorde House. A second branch conveys flows from the car hire centre and car parks on the east side of the railway line to PS19 which discharges to the gravity sewer just downstream of Concorde House, on the west side of the London to Brighton railway. The gravity sewer continues to the north as a 300mm diameter pipe collecting flows from the part of the main terminal building including the Bloc Hotel. The head of the other foul drainage system on the east side of the railway land serves activities ancillary to the airport operation, mainly offices – one via minor pumping station PS31 – and the Marriot and Hilton hotels and these flows all discharge to PS23 which pumps into a branch gravity sewer on the west side of the railway which also collects flows from the railway station. North of the terminal building the gravity sewer increases to 375mm diameter and collects flows from Ashdown House, Atlantic House and a 150mm diameter gravity sewer running along the west side of the terminal which conveys flows discharged by PS40 serving part of the terminal building and Pier 2 (via PS15 and PS16). Downstream of this connection there are two branches discharging from the west serving Pier 3 whereupon it changes dimension to 500mm diameter. At the police station there is a branch sewer from the west serving the local area then a pumped discharge from the new Premier Inn at the North Terminal which pumps into this system via an attenuation tank during the night. The airport flow then discharges to a 600mm diameter TW trunk gravity sewer which crosses the A23 and routes to Horley Sewage Treatment Works (STW) to the northwest. The connectivity is shown in **ES Appendix 11.9.7 Figure 2.2.3** (Doc Ref. 5.3). Details of the pumping station and gravity sewer critical assets and their contributing areas are given in Table 2.2.2 the pumping rates are those derived from the Andover reports.

Table 2.2.1 North Terminal System Critical Pumping Stations

Asset	Catchment	Configuration	Discharges to:	Flow Rate (l/s))
PS2	Fire station/Airfield Lighting building/Control Tower + PS 44	Duty/Standby	PS3	10
PS3	Virgin hangar Central Area Recycling Establishment and motor transport facilities+ PS2 + PS5 + PS45	Duty/Assist/ Standby	PS7 West Pumping Main	37
PS5	Garage/Old Control Tower	Duty/Standby	PS3	5
PS6	Cargo Sheds (west)	Duty/Standby	PS7	6
PS7	Cargo (east)/Sanitation block/Sweeper tip/Fuel farm + Surface Water System	Duty/Duty/ Standby	TW system	58
PS8	NT Concourse/Baggage Centre/Pier 4 (east)/Pier 5/Sofitel/Hilton Hotels + PS10 + PS11	Temporary pumps Duty/Assist/ Standby/Standby	PS7	31
PS10	Old Premier Inn/Shell Garage/Contractor Support Centre	Duty/Standby	PS8	10
PS11	Pier 4 (west)	Duty/Standby	PS8	6
PS44	Pier 6	Duty/Standby	PS2	10
PS45	Fire Training Ground	Duty/Standby	PS3	6

Table 2.2.2: South Terminal System Critical Pumping Stations and Gravity Sewer Critical Assets

Asset	Catchment	Configuration	Discharges to:	Flow Rate (l/s))
PS19	Station/Car Hire centre + PS20	Duty/Standby	180mm Pipe	13
PS20	Car Park Office + PS21	Duty/Standby	PS19	5
PS21	Car hire centre	Duty/Standby	PS20	5
PS23	McDonalds/KFC/BP Garage/Offices/Marriot Hotel/Hilton Hotel + PS31	Duty/Standby/ Standby	225mm Pipe	26
PS31	Office A	Duty/Standby	PS23	5
PS40	Pier 2 (PS15+ PS16)	Duty/Standby/Standby	150mm Pipe	14
NPR	New Premier Inn (attenuation tank)	Duty/Standby	600mm Pipe	10l/s (estimated)
225mm Pipe (south)	South Sanitation Block/Boiler House/Concorde House	-	225mm Pipe (north)	-
225mm Pipe (north)	South Terminal Concourse/Bloc Hotel and PS19 catchment	-	300mm Pipe	-
375mm Pipe (south)	Shuttle Terminal/South Terminal main/Ashdown House and PS23 catchment	-	400mm Pipe	-
400mm Pipe (north)	Atlantic House/Pier 3/Police Station + New Premier Inn (NPR) PS	-	600mm Pipe TW system	-

3 Model Build

3.1 Network

3.1.1 The model was built primarily from the 1996/97 Unistride manhole survey data, as this included a database which could be exported into a form that can be imported into the modelling software: InfoWorks ICM Version 9.0. The survey data was not complete as there were missing sections of pipe and some manholes could not be entered to obtain the dimensions and levels. Approximately 80% of the system was complete with levels and chamber dimensions and 90% of the pipe data included sizes and invert levels. Other survey data was available, including from a survey conducted for the Bloc Hotel expansion and the installation survey for the current flow survey provided by the contractor. To complete the network model, operational staff confirmed pipe routing and sizes, and invert levels were inferred from existing survey data or using typical sizes and gradients based on the catchment served. The modelled networks terminate as free discharge outfalls where the Gatwick system discharges to the TW trunk gravity sewers.

3.1.2 Head losses at ends have been applied using the in-built tool in InfoWorks, adjusted where more than one pipe discharges at any particular manhole. Pipe roughness has been assumed as 3.0mm for use with the Colebrook-White method for calculating pipe flow capacity. This is a water industry accepted typical value for a slimed foul pipe. The exception to this is where CCTV survey data was available for parts of the South Terminal gravity network which was surveyed as part of the design for the Bloc hotel and individual top and bottom pipe roughnesses have been applied commensurate with the condition reported in these sewers (taken from the document 20206-XX-B-800-SUR-000005 Bloc Hotel Extension – Survey Report). No sediment was reported in the system so none has been modelled.

3.1.3 InfoWorks models usually include a storage compensation volume to account for the un-modelled small diameter sewers and individual lateral pipes from properties into the main modelled sewers. However, the standard methodology for modelling these is based on a residential population in a notional housing development and is considered inappropriate for an airport system. Therefore, no allowance for any additional storage in the network has been allowed, but given that many of the smaller diameter pipes are included in the model, this is not considered to have a detrimental impact on model performance and if anything will give slightly conservative predicted depths.

3.2 Pumping Stations

3.2.1 Some pumping stations had been surveyed with basic details such as wet well diameter and depth, but others had no detail and assumptions were made based on typical pumping stations of that capacity. The rising main routes were derived from the GIS data which also provided some supplementary data. Pumping rates and operating levels were derived from the Andover control system data, supplemented by flow survey data and operational knowledge. The representation of the pumping stations is described in the subsequent sections. PS21 and PS31 have not been modelled and therefore are not listed below as they serve only one building and the flows are captured within the downstream PS catchments (PS20 and PS23 respectively).

3.2.2 The model network is shown in **ES Appendix 11.9.7 Figure 2.1.1** (Doc Ref. 5.3).

Pumping Station 2.

3.2.3 This installation is located near to the control tower and operates on a Duty/Standby basis. A single pump has been modelled with a fixed pumping rate of 10l/s based on operational knowledge. No details of the wet well are known so it is assumed as 1.8m diameter and 3.45m deep to provide a sump below the lowest incoming pipe. Based on the Andover pump operation the wet well only operates four times per day discharging a large volume at PS3 which indicates that the start level must be high and utilizing the branch sewers as storage, so the start and stop levels are modelled as 57.00m and 55.70m AOD respectively.

Pumping Station 3

3.2.4 This installation is located in the transport engineering area and operates on a Duty/Assist/Standby basis. A single pump has been modelled with a fixed pumping rate of 32l/s as derived from the Andover average pump flow rate data for dry days, but Duty/Assist operation was recorded during wet weather on 10th June 2019 so a second pump link with a flow rate of 5l/s was added to start a little higher and stopping at the same level as the Duty pump as the pump operation data records. No details of the wet well are known so it is assumed as 1.8m diameter and 3.45m deep to provide a sump below the lowest incoming pipe. Based on the Andover pump operation and depth data, the start and stop levels are modelled as 54.61m AOD Duty/54.63m AOD Assist and 54.29m AOD respectively. Since these pumps are inhibited from operating when the pumps at PS7 operate, a Real Time Control rule has been applied to replicate this in the model. However, it was found that the inhibit was over-ridden during wet weather and both PS3 and PS7 operated in concert over a long period, although with PS7 operating the PS3 pump rate was limited to 32l/s, so the model control logic reflected this.

Pumping Station 5

3.2.5 This installation is located in the motor transport facilities and operates on a Duty/Standby basis. A single pump has been modelled with a fixed pumping rate of 5l/s. The wet well is included in the manhole data set and is 1.8m diameter and 3.13m deep. The operation is not recorded on the Andover system so the pump operation has been assumed and the start and stop levels are modelled as 56.10m AOD and 55.90m AOD respectively.

Pumping Station 6

3.2.6 This installation is located north of the cargo sheds and operates on a Duty/Standby basis. A single pump has been modelled with a fixed pumping rate of 6l/s. No details of the wet well are known so it is assumed as 1.8m diameter and 3.62m deep to provide a sump below the lowest incoming pipe. The operation is not recorded on the Andover system so the pump operation has been assumed and the start and stop levels are modelled as 56.00m AOD and 55.70m AOD respectively.

Pumping Station 7

3.2.7 This installation is located to the east of the cargo sheds and operates on a Duty/Duty/Standby basis. Two pumps were originally modelled each with a fixed pumping rate of 29l/s as derived from the Andover average pump flow rate data, but during the calibration it was found that they did not pump evenly and one pump was set to 26l/s and the other 31l/s to reflect the data recorded during the wet weather period. The wet well is included in the manhole data set and is 1.6m diameter and 3.82m deep. Based on the Andover pump operation and depth data, the start and stop levels are modelled as 54.10m AOD and 53.73m AOD respectively for both pumps.

Pumping Station 8

3.2.8 This installation is located beneath Pier 5 at the North Terminal and operates on a Duty/Assist/Standby/Standby basis. A single pump has been modelled with a fixed pumping rate of 31l/s as derived from the Andover average pump flow rate data. The installation comprises multiple temporary submersible pumps located in manhole FW27414607. No details of the wet well are known so it is assumed as 1.2m diameter (the next size up from the upstream manholes) and 7.46m deep based on the invert level of the outgoing pipe. Based on the flow survey pump operation and depth data, the start and stop levels are modelled as 52.31m AOD and 52.04m AOD respectively.

Pumping Station 10

3.2.9 This installation is located adjacent to the Northgate Building and operates on a Duty/Standby basis. A single pump has been modelled with a fixed pumping rate of 10l/s. The wet well is included in the manhole data set and is 1.8m diameter and 3.68m deep. The operation is not recorded on the Andover system so the pump operation has been assumed and the start and stop levels are modelled as 54.60m AOD and 54.30m AOD respectively.

Pumping Station 11

3.2.10 This installation is located beneath Pier 4 at the North Terminal and operates on a Duty/Standby basis. A single pump has been modelled with a fixed pumping rate of 6l/s. The wet well is included in the manhole data set and is 1.2m diameter and 3.54m deep. The operation is not recorded on the Andover system so the pump operation has been assumed and the start and stop levels are modelled as 54.00m AOD and 53.70m AOD respectively.

Pumping Station 19

3.2.11 This installation is located on the east side of the railway in the car hire centre by the South Terminal and operates on a Duty/Standby basis. No details of the wet well are known so it is estimated from a photograph as 2.4m diameter and 6.45m deep to provide a sump below the lowest incoming pipe. Based on the flow survey level data the start and stop levels are modelled as 53.39m AOD and 52.46m AOD respectively. The pump rate was given as 16l/s, but the pump flow data gave an average of 13l/s during the early June 2019 dry period and this has been used in the model, although the pump rate was quite variable in practice and for the wet period 10-11 June 2019, the rate was predominantly 16l/s, so an Assist pump was modelled to give the higher rate when more flow was in the wet well.

Pumping Station 20

3.2.12 This installation is located on the east side of the railway by the coach park and operates on a Duty/Standby basis. A single pump has been modelled with a fixed pumping rate of 5l/s. No details of the wet well are known so it is assumed as 1.5m diameter and 2.70m deep to provide a sump below the lowest incoming pipe. The operation is not recorded on the Andover system so the pump operation has been assumed and the start and stop levels are modelled as 55.70m AOD and 55.50m AOD respectively.

Pumping Station 23

3.2.13 This installation is located on the east side of the railway on Ring Road North and operates on a Duty/Standby/Standby basis. A single pump has been modelled with a fixed pumping rate of 26l/s as derived from the Andover average pump flow rate data. The wet well is included in the manhole data set and is 2.65m diameter and 6.72m deep. Based on the Andover pump operation and depth data, the start and stop levels are modelled as 52.755m AOD and 52.190m AOD respectively. The pump rate was given as 29l/s, but the pump flow data gave an average of 26l/s and this has been used in the model, although the pump rate was quite variable in practice.

Pumping Station 40

3.2.14 This installation is located beneath the main South Terminal building and operates on a Duty/Standby/Standby basis, although at times it was observed that two pumps were operating as one pump failed to maintain the flows presumably due to a fault. A single pump has been modelled with a fixed pumping rate of 14l/s. No details of the wet well are known so it is assumed as 1.5m diameter and 2.55m deep to provide a sump below the lowest incoming pipe. Based on the Andover pump operation and depth data, the start and stop levels are modelled as 56.62m AOD and 56.40m AOD respectively. The pump rate was given as 16l/s, but the pump flow data gave an average of 14l/s and this has been used in the model, although the pump rate was quite variable in practice.

Pumping Station 44

3.2.15 This installation is located beneath Pier 6 at the North Terminal and operates on a Duty/Standby basis. A single pump has been modelled with a fixed pumping rate of 10l/s. No details of the wet well are known so it is assumed as 1.2m diameter and 2.00m deep to provide a sump below the lowest incoming pipe. The operation is not recorded on the Andover system so the pump operation has been assumed and the start and stop levels are modelled as 56.30m AOD and 56.00m AOD respectively.

Pumping Station 45

3.2.16 This installation is located on the west side of the Uniform Taxiway near to the North Terminal and operates on a Duty/Standby basis. A single pump has been modelled with a fixed pumping rate of 6l/s. No details of the wet well are known so it is assumed as 1.2m diameter and 2.00m deep to provide a sump below the lowest incoming pipe. The operation is not recorded on the Andover system so the pump operation has been assumed and the start and stop levels are modelled as 56.00m AOD and 55.80m AOD respectively.

3.3 Contributing Areas

3.3.1 The geographical contributing areas for the gravity and pumped network were added to the model as sub-catchments and derived from an assessment of the sewer network and the background plans and mapping, although some areas were difficult to judge as the internal drainage routing in the terminal buildings – particularly for the South Terminal - was not obvious and assumptions were made as to the discharge locations. Since the model has to accommodate the flows from disparate sources, overlapping sub-catchments were used with different categories of dischargers, such as passengers and staff.

3.3.2 Flow survey data confirms that there is little storm response from the South Terminal system, but there is a storm response in the North Terminal system as highway drainage is known to connect into the foul sewer on Timberham Farm Road and in the vicinity of PS3. The area connected was identified from the storm system model and from operational staff, and the runoff characteristics from the storm system model were applied to the runoff for consistency.

3.4 Sources of Foul Flows

3.4.1 Various sources of foul flow discharge into the foul sewer system as follows:

- Passenger foul flows
- Worker foul flows, including in offices and ancillary buildings
- Aircraft discharge foul flows, emptying of aircraft toilet tanks via the sanitation block
- Hotel flows, which can also include conference and restaurant facilities
- Fast food restaurant flows (domestic and washing etc.)
- Fuel service station and railway station flows
- Non-domestic flows, which could be characterized as trade effluent including from:

- The chilling station
- Car hire workshops and washing areas
- Aircraft washing area
- Fire training ground

3.4.2 The model has to take account of both the quantum of the flows discharged, and the pattern of discharges on an hourly and seasonal basis from all sources.

Passenger Foul Flows

3.4.3 Passenger foul flows have been estimated from passenger data in the form of spreadsheets, which included annual (2014 to 2018), monthly, daily and hourly throughputs broken down for the daily flows by terminal and the pier in which the passengers embarked or disembarked. General aviation figures were included in the North Terminal figures as they were negligible taken in isolation. The data used in the model initially was based on the figures for the flow survey period 31st May to 7th June 2019. 2019 data is being used for this assessment as a pre-COVID year provides a more robust representation of flows for future years than more recent data. The manner in which populations are modelled in InfoWorks is as follows:

- A fixed number embodied in the model sub-catchments
- A daily diurnal profile of discharges based on a fixed discharge flow per day per head of population which is defined in a Wastewater File. This profile can also be given a seasonal profile to reflect the changes in passenger numbers on a monthly basis.

3.4.4 The software cannot accommodate changing populations either on a daily basis or over the course of a day, so multiple copies of the model – called scenarios – have to be created to enable the calibration for different days. The wastewater profile can be used to replicate the change in passengers over the 24 hour cycle, although the discharge rate has to be fixed for all passengers, which may not be strictly correct, as more use by passengers of eating facilities could be expected where terminal dwell periods cover traditional meal times. In order to reduce the variables, the distribution between piers was assumed to be constant, as were the flows generated per passenger. The pier distributions for the calibrated model are:

South Terminal

- Pier 1 21%
- Pier 2 61%
- Pier 3 18%

North Terminal

- Pier 4 30%
- Pier 5 24%
- Pier 6 46%

3.4.5 Since the pattern of arriving and departing passengers differs at each terminal, multiple wastewater profiles had to be created to reflect the individual passenger flow patterns. The daily patterns for the peak passenger flow date in 2018 (24 August) is shown in Figure 3.4.1.

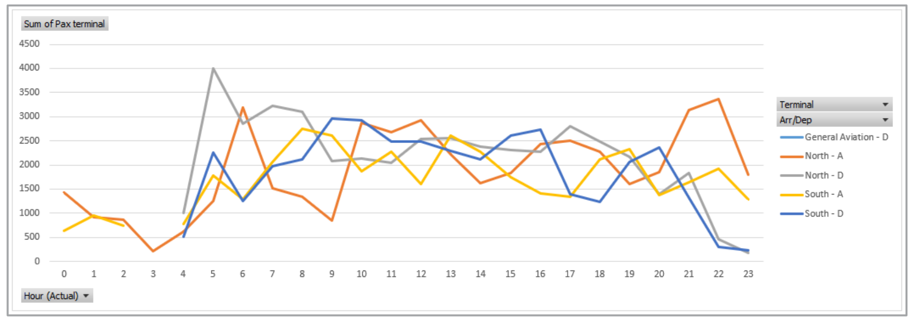


Figure 3.4.1: Daily passenger flow patterns

3.4.6 During model calibration it was found that the flows at the South Terminal were complicated by the various discharge points into the gravity systems that run under the terminal. In order to achieve a reasonable calibration, the flows for the departing and arriving passengers had to be split into four discharge locations as follows:

- Sewer in service road under the landside terminal area
- PS40 gravity
- Sewer in service road under north of terminal building
- Sewer in service road under the airside terminal area

3.4.7 The passengers are represented in the model as follows: once when they are within the main lounge areas where most of the eating and drinking establishments reside, and again in the individual piers 2, 3 and 6 where refreshment facilities are located, or where the toilet facilities are likely to be used pre-flight.

3.4.8 With respect to the actual discharges by passengers and the facilities they use (modelled as a per capita water consumption), the initial value used was 15 litres per head per day (l/h/d), which is the published rate based on US design standards (there is no equivalent UK standard). However, this was found to generate grossly exaggerated flows in the model, and the values from the calibration were 8 l/h/d for the lounges and 1 l/h/d for the piers.

There is no published data for arriving passengers, so these were estimated as 1.25 l/h/d based on using toilet and light refreshment facilities and also assuming a contribution from ‘meeters and greeters’ who would otherwise not be accounted for.

Worker Foul Flows

3.4.9 The worker flows have been split into those working in the operational airport at various locations and those occupying the various offices both in the terminal areas and on the east side of the railway. Since the airport is a 24 hour transport hub, the airport workers operate on shifts, but the shift patterns vary between employers, so there is variability in the locations and patterns of flow over a 24 hour period. In developing the model, two profiles have been assumed: one for the day shift which is extended into the evening; and one for the night shift as shown in Figure 3.4.2. Some overlap is assumed as the profiles are applied to different categories of workers, e.g. there are some terminal staff working late into the evening which overlap with night shift aircraft maintenance staff.

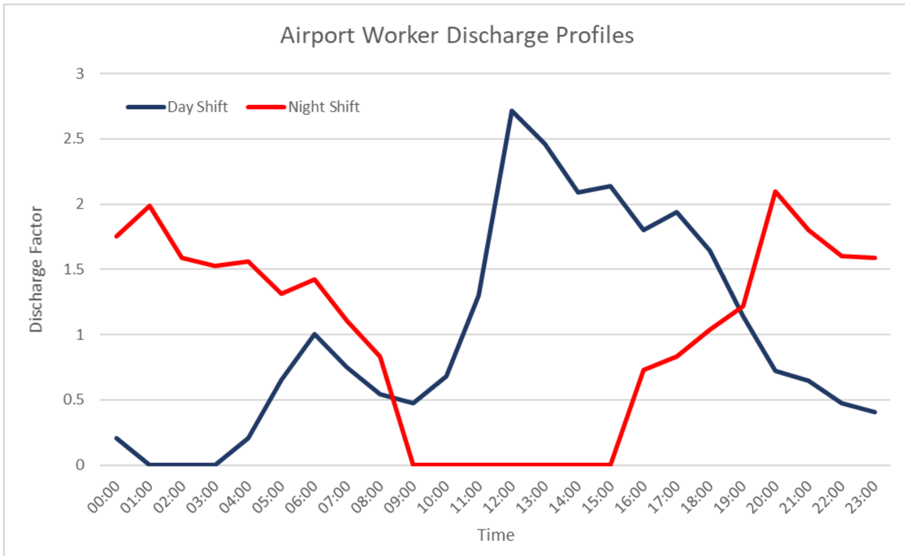


Figure 3.4.2: Assumed diurnal wastewater discharge profiles for airport workers

3.4.10 The number of workers and the flows per worker have been estimated as they cannot be disaggregated from the available flow data which includes the discharges from all sources. Published data suggests a consumption rate of 40 l/h/d which includes for the associated messing facilities and this was adopted.

3.4.11 The numbers of workers were derived in two ways: where the workers were located in discrete areas, such as the fuel farm, Virgin hangar and cargo terminal, an aerial photograph was used to count the parking spaces and these were used as a surrogate for the number of workers during the day shift. For the workers located in the main terminal buildings who use remote parking, the numbers were estimated during the calibration based on the 40 l/h/d consumption rate, after deducting the predominant passenger-related flows and other flows which had a greater degree of confidence. The final calibrated model assumes the following airport worker numbers (excluding discrete office blocks) as shown in Table 3.4.1.

Table 3.4.1: Modelled Airport Workers

Day Shift		Night Shift	
Location	**Population**	**Location**	**Population**
Car Hire Centre	20	Aircraft Servicing Night Shift	50
Car Hire Workshops	20	Virgin Hangar Night Shift	50
Boiler House	10	North Terminal Staff Night	150
Baggage Centre North	100	Fire Station Night Shift	30
Station	25	Concorde House Night	50
Fuel Farm	40		
Cargo Sheds West	50		
Cargo Sheds East	60		
Virgin Hangar	80		
Old Control Tower	25		
Motor transport facilities/Garage	60		
Central Area Recycling Establishment	40		
Fire Station	30		
Airfield Lighting/Control Tower Area	75		
North Terminal Staff	300		
South Terminal Staff 1	500		
130/140 Stands	50		
Car Park Plaza 1	10		
Car Park Office	20		
Car Park Plaza 2	5		
South Terminal Staff 2	100		
South Terminal Staff 3	200		
Baggage Handling South	20		

3.4.12 The combination of the numbers of workers at each location and the assumed consumption rate resulted in a good calibration.

3.4.13 In addition to the airport workers listed in Table 3.4.1, there are also discrete office blocks with additional airport workers. For offices, the standard published consumption rate is 50 l/h/d including provision of a canteen, although this was found to be too high and was revised to 40 l/h/d during the calibration. For the offices on the east side of the railway, the numbers of workers were estimated based on the number of car parking places and for the offices adjacent to terminals without car parks, the number was estimated based on the total floor area. The modelled office workers and their locations are given in Table 3.4.2.

Table 3.4.2: Modelled Airport Office Workers

Location	Population
First Point Office	300
Schlumberger House Office	450
Concorde House	250
Ashdown House	500
Atlantic House	150
Police Station	50
Contractor Support Centre	10
Jubilee House	100
Office B495	10

3.4.14 It is acknowledged that the numbers, distribution and consumption rate are approximate. These flows are not critical and the allowance for workers is considered to be an adequate representation for modelling purposes.

Aircraft Discharge Foul Flows

3.4.15 The flows from the aircraft toilet tanks are collected and discharged at the North Terminal Sanitation Block at present. The volume of discharge is approximately 120m^3 per day. This has been converted to an instantaneous flow rate and a profile applied which is a composite of the North and South Terminal arriving passenger profiles.

Hotel Flows

3.4.16 The published guidance on hotel flows is based on an overall flow rate per room, rather than separate discharges for guests, workers and other flows such as those arising from catering. The guidance differentiates between those hotels offering a full service, including catering and conference facilities, and those that provide budget accommodation with limited catering facilities. The number of rooms was sourced from GAL data and confirmed by reference to the hotel websites. The hotel accommodation is listed in Table 3.4.3

Table 3.4.3: Hotels Discharging to Airport Foul System

Hotel	Type	Location	No. of Rooms
Sofitel Hotel	Full service	North Terminal	518
Old Premier Inn	Budget	North Terminal	100
New Premier Inn	Budget	North Terminal	1000
Marriott Hotel	Full service	South Terminal	223
Hilton Hotel	Full service	South Terminal	821
Bloc Hotel	Budget	South Terminal	245
Hampton Hilton Hotel	Full service	North Terminal	200
YotelAir	Budget	South Terminal	63

3.4.17 The published flow per room is given as 350l/day and 120l/day for full service and budget hotels respectively. However, the flows were found to be too high during the calibration and were reduced to 180l/day and 100l/day. The actual occupancy is unknown, so the seasonal profile has been used which applies an 80% factor for low periods and 100% for high periods, based on the seasonal passenger flow data.

Fast Food Restaurant Flows

3.4.18 There are two fast food restaurants (McDonalds and KFC) located east of the South Terminal, which are not counted in the South Terminal wastewater flows. Although there is some published guidance on flow rates in the USA, without numbers of patrons this information is of limited use. In the absence of any firm data it was assumed that there were 40 patrons per hour on average. The consumption flow of 12 l/patron was estimated and the calibration achieved using these figures was reasonable as it contributed to a good calibration result.

Fuel Service Station and Railway Station Flows

3.4.19 The toilet facilities at fuel service stations and the railway station generate some foul flow although there are no available figures to suggest the daily total and there is no published guidance. A flow of 2000 litres per day has been assumed for the fuel stations. For the railway station, the passenger dwell times are very short and the flow from the passengers (estimated as 48,000 passengers per day by Network Rail) is estimated to be nominal and a value of 1000 litres per day has been assumed.

Non-Domestic Flows

3.4.20 Tenant Trade Effluent Consents data together with metered flows for the period 2013 to 2018 allows the non-domestic discharges to be estimated. Pre-2020 data is being used for this assessment as pre-COVID years provide a more robust representation of flows for future years than more recent data. Most of the discharges are trivial as less than 10m^3 per day for the peak month so are too small to sensibly model, but several flows are more significant and have been added to the model as follows:

- Avis/Budget Carwash and Hertz Hire Car Centre 0.5l/s over 16 hour period
- Europcar/National Hire Car Centre 0.7l/s over 16 hour period
- Aircraft washing facility 2.0l/s over a 4 hour period
- Fire Training Centre 0.46l/s over 3 hour period
- Chiller Station 0.6l/s over 24 hour period

3.5 Base Infiltration Flows

3.5.1 Most sewer systems experience ingress of groundwater and the age and consideration of the sewers together with the water table level are the prime considerations in determining the volume of ingress. The soil types at the airport are slow-draining clayey soils that often result in high water tables, so some ingress can be expected. The flow survey night-time data has been used to determine the rate of ingress in each pumping station catchment and at the gravity pipe flow monitors, although this is complicated by the relatively high foul flow to the system compared to a residential area at this time of the daily cycle. Nevertheless, an estimate of the near constant base infiltration flows has been undertaken during the calibration and the modelled allowances are given in Table 3.5.1.

Table 3.5.1: Modelled Infiltration Base Flow

Catchment	Flow rate (l/s)
PS8	4.0
PS7	1.5
FM01	0.2
PS23	1.0
PS19	1.0
PS3	0.75
PS2	0.2
PS40	0.25

3.6 Known Sources of Storm Flows

3.6.1 The airport surface water drainage system is nominally separately drained, with storm flows discharged to a surface water network ultimately draining to local watercourses. However, there is highway drainage connected to the North Terminal foul system in Timberham Farm Road which discharges to Pumping Station 7. The exact extent of the contributing area is unconfirmed but is believed to be approximately 1.5ha , based on the gap in the coverage of the drainage system hydraulic model.

3.6.2 In addition, there are various paved areas that are connected in the vicinity of Pumping Station 3 of approximately 0.5 ha.

3.6.3 The impermeable and permeable areas were measured from the mapping and inserted in the model using appropriate runoff factors, which were subsequently revised during the wet day calibration (see Section 5.4).

4 Preparation of flow survey data

4.1 Introduction and Survey Details

4.1.1 In order to provide data for the model calibration to supplement that available from the Andover system, a short-term flow monitoring survey was undertaken of the foul system. As the system is foul only, there was no need for a protracted survey to capture storm flows, so a period of one week was chosen which provides data over a peak weekend and typical weekdays, with the latter offering alternative data for calibration should the data quality not be sufficient for calibration at any particular location over the weekend.

4.1.2 A total of three standard flow monitors, two depth monitors and one pump run time monitor were installed in the catchment. No rain gauges were required as the system is predominantly foul and any locations affected by storm runoff were calibrated separately using rainfall data from the Environment Agency gauge at Burstow 2km east of the South Terminal. Installation was completed by 30th May 2019 and the equipment was removed by 12th June 2019. 2019 data is being used for this assessment as a pre-COVID year provides a more robust representation of flows for future years than more recent data.

4.1.3 A pre-inspection survey was undertaken which followed a desktop study undertaken prior to the survey to choose potential sites that would provide the best data at the locations of interest. All but one of the monitors were installed in their intended locations with the exception of the depth monitor at PS8 which could not be installed in the temporary wet well due to a lack of access, instead the monitor was installed in a manhole on the incoming pipe which provided adequate data as the pump operation range was above the chamber invert level.

4.2 Flow Monitor Sites and Types

4.2.1 Standard depth and velocity data monitors were used at all sites.

4.2.2 Table 4.2.1 lists all the flow monitor locations (manhole references from Unistride manhole survey data).

Table 4.2.1: Flow Monitor Locations

Site Number	Manhole Number	Location	Size (mm)	Shape	Comments
FM01	FW28415312	Service road under South Terminal	300	Circular	The proposed preferred site was selected.
FM02	FW28411803	Police Station Compound	400	Circular	The proposed preferred site was selected.

Site Number	Manhole Number	Location	Size (mm)	Shape	Comments
FM03 (Gatwick ref. FM40X)	MH_P143	Airside service road at South Terminal	150	Circular	The proposed preferred site was selected.

4.2.3 Table 4.2.2 provides the details of the depth monitor locations.

Table 4.2.2: Depth Monitors

Site Number	Manhole Number	Location
DM8	FW27414610	Manhole upstream of temporary PS8 wet well
DM19	FW28417201	PS 19 wet well

4.2.4 The locations are shown on **ES Appendix 11.9.7 Figure 4.2.1** (Doc Ref. 5.3).

4.3 Suitability of Flow Survey Data

4.3.1 The suitability for calibration of the flow survey data for each of the flow and depth monitors was assessed.

FM01 (Service road under South Terminal)

4.3.2 FM01 (Figure 4.3.1) was located on a 300mm foul sewer on the inlet to manhole FW28415312 on the service road under the South Terminal. It records flows from the gravity system in the southeast parts of the South Terminal, including from Concorde House, and the southern part of the estate east of the railway which discharges to PS19. The flow was sluggish as the sewer has a flat gradient (so no theoretical depth/velocity curve). Good data was recorded with only occasional drop-out due to ragging. A wide scattergraph was obtained, with a wide variation of depths for the velocity data, but this reflected the pumping operation of PS19 for which frequent operation of the pumps was observed. The data is adequate for calibration.

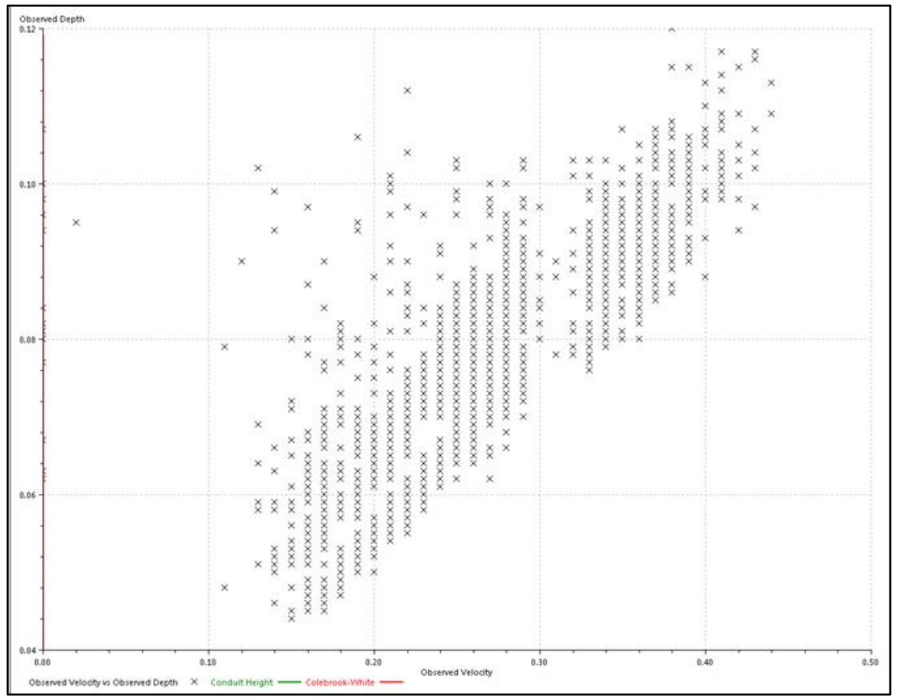


Figure 4.3.1: FM01 Scattergraph (Service road under South Terminal)

FM02 (Police Station Compound)

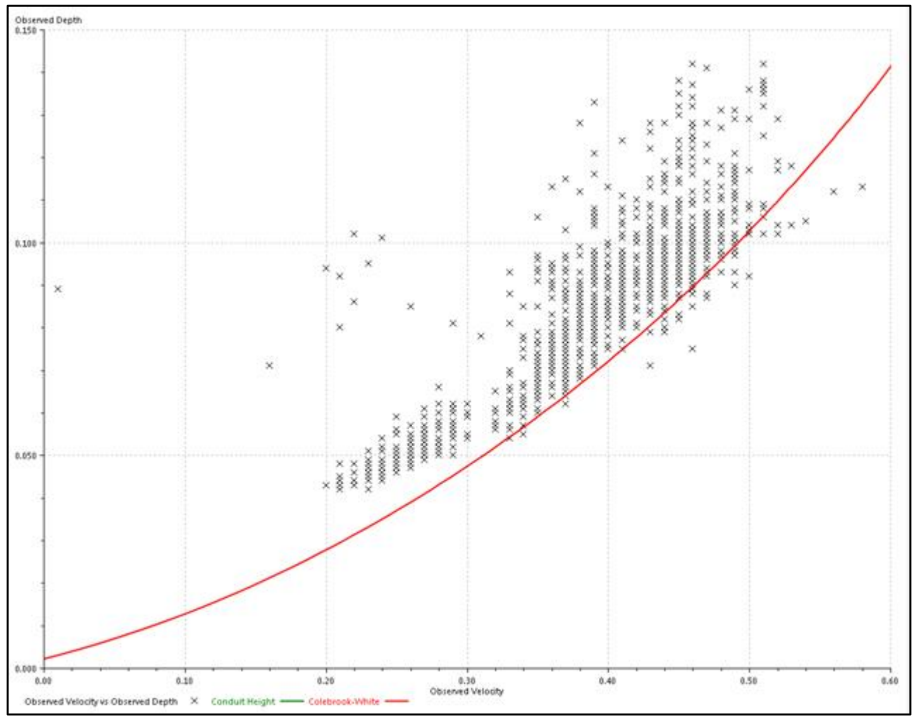


Figure 4.3.2: FM02 (Police Station Compound) Scattergraph

4.3.3 FM02 (Figure 4.3.2) was located on a 400mm foul sewer on the inlet to manhole FW28411803 within the police station dog exercise compound. It records all flows from the South Terminal area and the whole of the estate east of the railway via PS19 and PS23; only a small flow from the end of Pier 3 is not captured. Good data was recorded with only occasional drop-out due to ragging. A wide scattergraph was obtained, with a wide variation of depths for the velocity data, but this reflected the pumping operation of PS19 and PS23 for which frequent operation of the pumps was observed. The scatter curve follows the theoretical depth/velocity curve (red line) but at a higher depth which suggests that there may be sediment in the downstream sewer. The flow balance for this site was compared to the flows recorded at FM01 and FM03 upstream and the balance was good. The data is adequate for calibration.

FM03 (Airside service road at South Terminal)

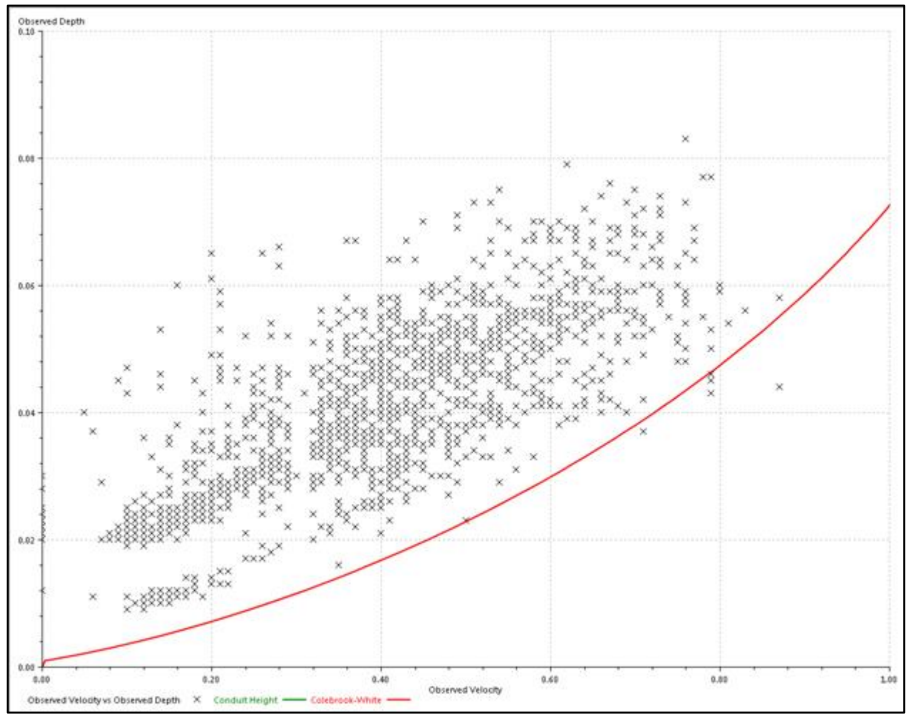


Figure 4.3.3: FM03 (Airside service road at South Terminal) Scattergraph

4.3.4 FM03 (Figure 4.3.3) was located on a 150mm foul sewer on the inlet to the manhole NO39116701 on an airside service road at the north end of the South Terminal. It records flows from the gravity system in the southwest parts of the South Terminal, and from Pier 2 which discharges to PS40. The flow was sluggish as the sewer has a flat gradient (so no theoretical depth/velocity curve). Good data was recorded with only occasional drop-out due to ragging. A wide scattergraph was obtained, with a wide variation of depths for the velocity data, but this reflected the pumping operation of PS40 for which frequent operation of the pumps was observed. The scatter curve follows the theoretical depth/velocity curve (red line) but at a higher depth which suggests that there may be sediment in the downstream sewer. The data is adequate for calibration.

DM8 (Airside service road at North Terminal)

4.3.5 DM8 was located on a 225mm foul sewer on the invert in manhole FW27414610 on an airside service road at the west end of the North Terminal. It records the variation in depth at the temporary PS8 wet well 29m downstream as the pump operation levels are high enough to be recorded here. The quality of the data was good.

DM19 (PS19 wet well)

4.3.6 DM8 was located within the wet well of PS19 and records the variation in depth due to the operation of the PS19 pumps. Current clamps were fitted to the Duty and Standby pump control panel and these recorded when the pumps operated. The quality of the data was good.

4.3.7 It was noted that the OnSite data used Greenwich Mean Time as a clock datum compared to British Summer Time in the GAL data, so the OnSite times were adjusted for parity.

4.3.8 As the system is a foul system, a minimal storm response was expected, but the flow monitors recorded quite strong responses, so rainfall data was required to correlate the response. There is a rain gauge on the airfield and initially this data was used, but it was found to grossly under-measure the depth, so the Environment Agency gauge at Burstow 2km east of the South Terminal was used.

4.4 Selection of Calibration Events

4.4.1 Ideally, the calibration needs to cover a period of high passenger numbers at the airport, as it is the peak operation that is of most interest. However, this has to be coupled with the recording of good data at all or most of the monitor locations, both from the flow survey and the Andover system. The Saturday and Sunday (1 and 2 of June 2019) passenger numbers were elevated as this weekend followed a Bank Holiday and also benefited from additional traffic from a sports event. However, FM02 suffered a loss of velocity data on the evening of the 1st and FM03 also suffered a similar loss at this time, so the following Monday (3rd June 2019) was chosen instead as this also had comparable passenger numbers to the Sunday. Friday 31st May 2019 also had high passenger numbers and reasonable data so this was also used as a second weekday, although FM03 data suffered some loss over the evening of 31st May 2019 and the morning of 1st June 2019, but the peaks were captured so this data was used. Sunday 9th June 2019 also had high passenger numbers so this too was selected.

4.4.2 Rainfall occurred on 4 June 2019 (4.2mm), intermittently over 7 and 8 June 2019 (17.4mm), on 10 June 2019 (48.6mm) and 11 June 2019 (1.2mm):). This provided an opportunity to determine the impact of rainfall on the network. A response to rainfall was detected at most locations on the South Terminal system, and also on the North Terminal system at PS3, PS7 and PS8.

4.4.3 The Andover data capture was comprehensive except that the data for the wet well depth at PS3 failed to record, although this is a minor issue as the pumped flow data was still available and the pump operating levels were derived from data recorded in February 2019.

4.4.4 The following dry days were selected for catchment calibration:

- Dry Day 1 – Friday 31 May 2019 00:00 to 00:00 1st June 2019
- Dry Day 2 – Sunday 2 June 2019 00:00 to 00:00 3rd June 2019
- Dry Day 3 – Monday 3 June 2019 00:00 to 00:00 4th June 2019
- Dry Day 4 – Sunday 9 June 2019 00:00 to 00:00 10th June 2019

4.4.5 Based on the rainfall data, it was calculated that the storm on 10th June 2019 had a 25% (1 in 4) Annual Exceedance Probability (AEP) and anecdotal evidence confirmed that the rainfall was very heavy on that day. The system response compared to the dry days at the beginning of the month are shown in Table 4.4.1.

4.4.6 An outline storm calibration was carried out with the available data. The greatest depth fell on 10 June 2019, so the full day was chosen as it rained shortly after midnight and continued to the early morning of the 11 June 2019. The model was run into the dry period before rainfall recommenced at 18:00 on 11 June 2019 to assess the slow response. It should be noted that the rainfall which was recorded is not considered suitable for a full calibration because the data was collected at a timestep of 60 minutes, compared to a normal timestep of two minutes, so the rainfall profile was considerably attenuated and the peaks and troughs – and indeed the times – of the rainfall are not replicated in the rainfall data applied. However, a degree of calibration is possible, particularly with respect to the infiltration response as this is more dependent on rainfall depth than intensities.

Table 4.4.1: Storm Response in Foul Network

Location	Average Dry Day for ~155,000 Passengers Volume (m^3)	10th June 2019 48.6mm Rain for 150,300 Passengers Volume (m^3)	Increase
FM01 (ST Service Road)	368	614	67%
FM02 (ST Police Station)	902	No data	
FM03 (ST downstream PS40)	134	200	49%
PS19 (ST)	137	332	142%
PS23 (ST)	234	387	65%
PS3 (NT)	146	506	245%
PS7 Pump 1 (NT)	479	834	74%
PS7 Pump 2 (NT)	631	941	49%
PS8 (NT)	869	1,119	29%

4.4.7 Wet Period – Sunday 10th June 00:00 to 18:00 11th June 2019: A total of 48.6mm of rainfall fell in this period. The simulation was started on 4th June 2019 which is when rain first fell during the flow survey in order that the model could set up the ground moisture conditions for the wet period. The Net Antecedent Precipitation Index (NAPI) was calculated for the start of this simulation and found to be zero, which indicates a very dry soil condition, and this reflects the lack of rainfall over the preceding 30 days.

4.4.8 Notwithstanding some of the limitations of the data quality noted above, the flow monitor data quality for the chosen calibration days is given in Table 4.4.2 and is shown to be generally be Good or Fair.

Table 4.4.2: OnSite Flow Monitor Data Quality

Day	FM01		FM02		FM03	
	Flow	Depth	Flow	Depth	Flow	Depth
Dry Day 1	Good	Good	Good	Good	Poor	Poor
Dry Day 2	Good	Good	Good	Good	Good	Good
Dry Day 3	Good	Good	Good	Good	Good	Good
Dry Day 4	Good	Good	Fair	Poor	Fair	Good
Wet Period	Good	Good	Fair	Fair	Poor	Poor

4.4.9 The depth monitor data from the OnSite survey was good for all four days, as was the Andover data with the exception of the depths for PS3 which failed to record, so no data was collected, although flow data was available.

5 Calibration

5.1 Introduction

5.1.1 In order to have confidence in the model predictions, a calibration must be carried out to review the performance of the model against observed data throughout the foul network. The flow survey provides some of the data, but this is supplemented by data from the Andover system where suitable pumping station wet well depth, pumping rate and/or pump operation data is available. This is not normally recorded at the frequency required for a calibration, but the system was set up to do this over the course of the flow survey period between 31 May 2019 and 12 June 2019. The locations where data was collected from the Andover system are listed in Table 5.1.1.

Table 5.1.1: Andover Data Calibration Locations

Location	Data Monitored
North Terminal Area	
PS2	Pump Operation (Start/Stop)
PS3	Pump Operation (Start/Stop) Magflow Pumped Flow Depth
PS7	Pump Operation (Start/Stop) Magflow Pumped Flow Depth
PS8	Magflow Pumped Flow
South Terminal Area	
PS19	Magflow Pumped Flow Pump Operation (Start/Stop)
PS23	Pump Operation (Start/Stop) Magflow Pumped Flow Depth
PS40	Pump Operation (Start/Stop) Depth

5.1.2 The locations are shown on **ES Appendix 11.9.7 Figure 4.2.1** (Doc Ref. 5.3).

5.1.3 Unlike a normal sewer catchment where the population is static and only differs between weekdays and weekends with respect to foul discharges, the airport has a fluid population of passengers which has an uncertain distribution and varying dwell times, and the use of drinking and catering facilities also varies which creates a significant difficulty in both modelling this variability and calibrating the model. The modelling software is not able to replicate changing populations easily, and certainly not on a sub-daily or daily basis, so multiple models have to be created with the passenger numbers and distribution within the terminals and piers incorporated in discrete models for specific days. Since the primary interest is in the peak flows, the 31 May (Friday) and 2 (Sunday), 3 (Monday) and 9 (Sunday) of June 2019 were chosen for the calibration as these days had the highest passenger numbers, all recording approximately 155,000 passengers). Where necessary the data from the other days was used to check model performance. Storm response was calibrated using data from 10 and 11 June 2019.

5.1.4 Since any changes to variables in the model have to be consistent, this can cause difficulties if a variable change is required for one day and not another. This mitigates against a close fit between observed and predicted data for all events and the intent of the calibration is to obtain a broad agreement between the two sets of data with respect to peak flows, peak depths, pumping and gravity flow patterns and flow volumes. Published guidance suggests that flow volumes should agree within +/-10%, but this is only true where the flow data is of high quality which is not the always case here. For storm events the standard is more relaxed and +/-20% has been adopted to account for the uncertainties. For depths a tolerance of +/-100mm has been applied which is the published standard. The guidance also suggests a +/-10% tolerance for peak flows, but in a system fed by multiple pumping stations this is difficult to achieve as the model can never match the exact operation of the real pumping stations with respect to timing of operation – particularly where multiple pump operations are recorded - and the software also does not accurately replicate the damping of the pumped flow once it discharges into a gravity sewer, so tends to over-predict the peaks from pumped sources. To account for the difficulties described, a tolerance of +25/-15% has been used for peak flows which is the standard tolerance for storm flows.

5.1.5 In all cases, adjustments were made to either the assumed number of non-passenger flow generators (e.g. airport workers, hotel guests, etc.) or the assumed per capita flow daily discharge rate. No changes were made to passenger numbers or the diurnal profiles. Where night-time flows were recorded, base infiltration flows were added where the flow was judged not to derive from nightshift workers or activities.

5.1.6 For the storm calibration, the model commenced with a small contributing area discharging to PS7, but an analysis of the system response to rainfall suggested that small contributing areas were present in most catchments, so these were added during the calibration. A strong slow response was also recorded which is due to groundwater infiltration into the pipes and this was modelled using the software facility for such infiltration flows. During the calibration two techniques were tested and the various parameters were adjusted to achieve the best fit. In the event, the use of the ground store model rather than the soil store model gave the best results.

5.1.7 The storm model assessment was run as a continuous simulation from 4 June 2019 when the first recorded rainfall fell, through to the 12 June 2019 in order that the model could account for the rainfall cycle from runoff to infiltration over the full period. This gives the most accurate representation of groundwater infiltration. The foul components were left as those calibrated from early June, although in practice the passenger numbers fell slightly, but this has a minimal impact on the assessment.

5.2 Comparisons between Predicted and Recorded Flows

5.2.1 The location of all monitors are shown in **ES Appendix 11.9.7 Figure 4.2.1** (Doc Ref. 5.3).

5.3 Flow Survey Locations

FM01 (Service road under South Terminal)

5.3.1 The recorded data for all three dry days were generally good, although the high frequency of pumped flows affects the quality, but not significantly. The calibrated model replicates the diurnal profile of the flow reasonably well, although the exact pattern of pumped flows from PS19 could not be matched. The model broadly meets the criteria for matching flow volume, but there were some transient spikes in the observed data that skewed the peaks which were more than predicted by the model, although this is really a data issue than model inaccuracy. The depths all met the criteria. Overall, the model has achieved a good level of calibration for the dry days.

5.3.2 The data was good for the duration of the wet period. The observed data suggested that the storm response was a combination of rapid response due to runoff and a slower response due to groundwater infiltration. A contributing area of 0.1ha was added to the model to replicate the fast response and the ICM groundwater infiltration routine was used to generate the slow response from the upstream catchment which was approximately 8.5ha in extent. A good correlation was achieved in the flow and depth profiles and the model predictions met the peak flow, volume and depth criteria. The model achieved a reasonable wet period calibration here.

FM02 (Police Station Compound)

5.3.3 The recorded data suffered from some minor dropping out of data on the morning of 2 June 2019 but this has only a minor impact on volume. For the 9h June 2019 there was a partial blockage which resulted in sustained high depths which reduced the velocities to levels at the boundary of the equipment accuracy, so the depth data for this day is not representative and has not been used for the assessment. The calibrated model replicates the diurnal profile of the flow reasonably well, although the exact pattern of pumped flows from PS23 could not be matched. The model meets the criteria for achieving a +/-10% match for flow volume for most events, and the peak flows also generally met this criteria once the transient spikes in the observed data were accounted for. The model did not match the velocity and depth trends with respect to the values recorded, with the observed data having consistently higher depths and lower velocities (although with the model still meeting the depth criteria). This is believed to be caused by the sensor itself which is observed on the installation photograph to cause a wave across the sensor, so the measured depths are actually slightly higher than they should be. Given that the inlet and outlet pipes have been surveyed, there is a high confidence in the pipe gradients that would not justify any changes to achieve a better match and the model data is considered to be reasonably accurate. Overall the model has achieved a good level of calibration for the dry days.

5.3.4 On the last dry day there was a partial blockage in the sewer which resulted in elevated depths and intermittent velocities, so no usable data was recorded. The elevated depths are suggestive of backing up from the gravity sewer downstream and a depth of nearly 850mm was reached (Figure 5.4.1)

5.3.5 A test was performed to determine if this was due to genuine flow in the pipe and it was proven not to be, so there was either a partial blockage in the pipe downstream or backing up from a lack of capacity somewhere in the downstream network. No changes were made to the model at this location, which is affected by the changes made upstream. No conclusions as to the degree of calibration achieved can be made.

5.3.6 No storm response was identified at FM02.

FM03 (Airside service road at South Terminal)

5.3.7 The recorded data for all three dry days were generally only fair, with loss of data on the late evening on the 2 June 2019 and also on the morning of 9 June 2019. The observed volumes had a broad range which was not possible to match for all events, so a match was made against the highest to be conservative. The model follows the trend of the observed diurnal profile, although it has not attenuated the pump cycles from PS40 as much as the network achieves in reality, but the peak flows generally match. The model also meets the criteria for achieving a +/-10% match for depths except for the 9 June 2019 where the observed data gave a spurious high transient peak. Overall the model has achieved a good level of calibration for the dry days.

5.3.8 The data was good for the duration of the wet period. The observed data suggested that the storm response was a combination of rapid response due to runoff and a slower response due to groundwater infiltration. A contributing area of 0.1ha was added to the model to replicate the fast response and the ICM groundwater infiltration routine was used to generate the slow response from the upstream catchment which was approximately 1.7ha in extent. A good correlation was achieved in the flow and depth profiles and the model predictions met the peak flow, volume and depth criteria. The model achieved a reasonable wet period calibration here.

DM8 (Airside service road at North Terminal)

5.3.9 The recorded data for all three dry days was good with fairly consistent pump operation evident, although there is a flushing cycle every two days which affects the data for the 31 May 2019 and 2 June 2019 and the observed high and low depths when this occurs have been excluded for the calibration. . The model gives a reasonable match with the observed number of day- and night-time cycles, although the exact pattern of cycles could not be completely matched due to the variability in passenger numbers and consumption during the day. The flow volume calibration results are given under PS8. Overall the model has achieved a good level of calibration for the dry days.

5.3.10 The data was good although there was a pump test on the morning of the 10 June 2019 which the model does not seek to match. The onset of rainfall shows as increased cycles and more rapid filling of the wet well which is replicated in the model: the flows are discussed under FMPS8. The model met the depth criteria.

DM19 (PS19 wet well)

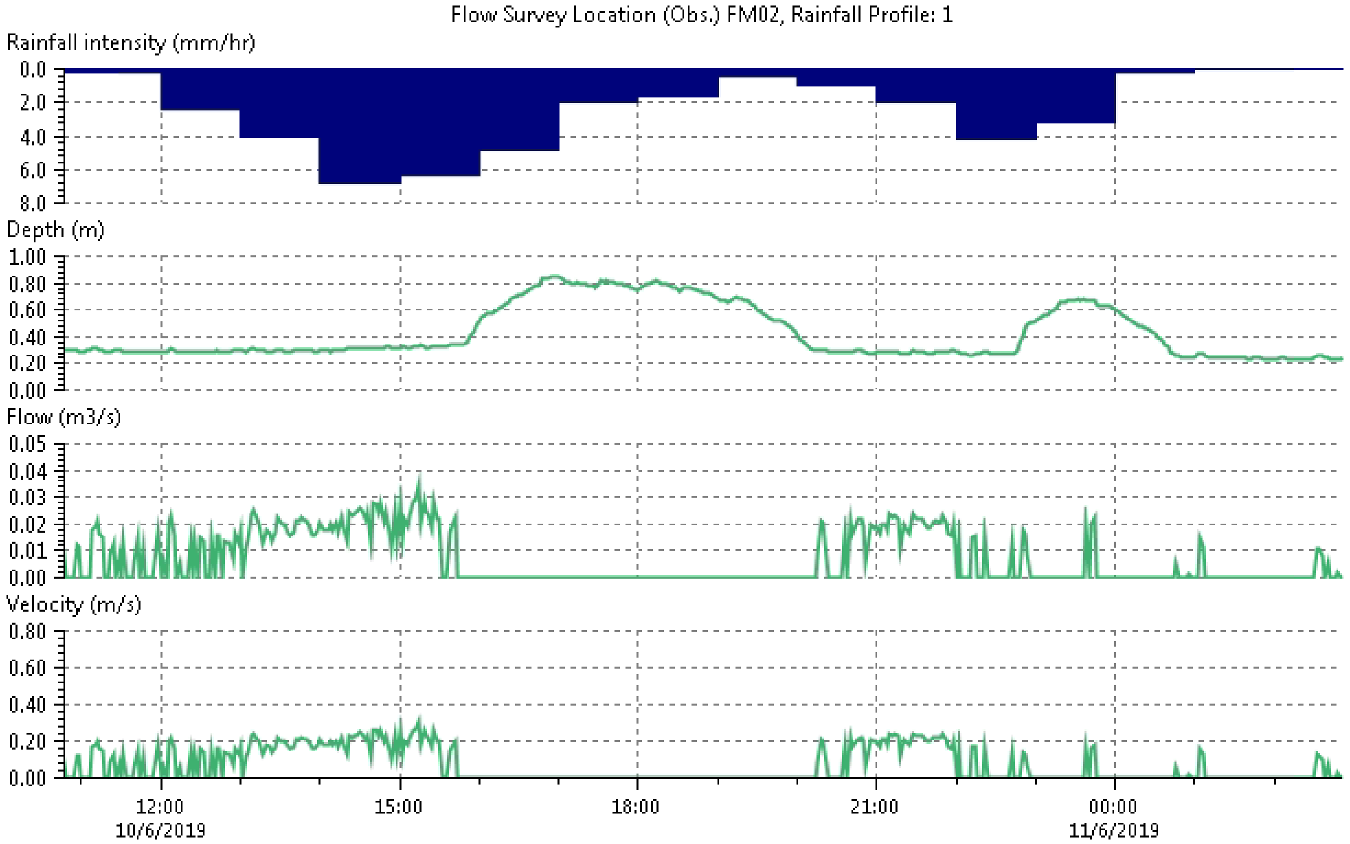
5.3.11 The recorded data for all three dry days was good with fairly consistent pump operation evident. The model gives a reasonable match with the observed number of day- and night-time cycles, although the exact pattern of cycles could not be completely matched due to the variability in passenger numbers and consumption during the day. The flow volume calibration results are given under PS19. Overall the model has achieved a good level of calibration for the dry days.

5.3.12 The data was good although it was noted that the pump operating levels varied more than in the dry days, with frequent depths above and below the nominal settings. The onset of rainfall shows as increased cycles and more rapid filling of the wet well which is reasonably well replicated in the model: this is discussed under FMPS19. The model met the depth criteria.

5.4 Andover Data Locations

DM3 (PS3 Wet Well)

This depth monitor was intended to record depths in the wet well. However, due to a fault, no data was collected. The pump cycles here are heavily influenced by the operation of PS2 and to a lesser extent PS4 and 5. Data was available for a dry period in February 2019 and a sample shown in Figure 5.4.2 shows that the model has a reasonable match with the observed depth trends.



	Rainfall			Depth		Flow			Velocity	
	Depth (mm)	Peak (mm/hr)	Average (mm/hr)	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Rain	39.243	6.800	2.450							
Observed				0.229	0.847	0.000	0.035	382.269	0.000	0.310

Figure 5.4.1 Depths on a dry day

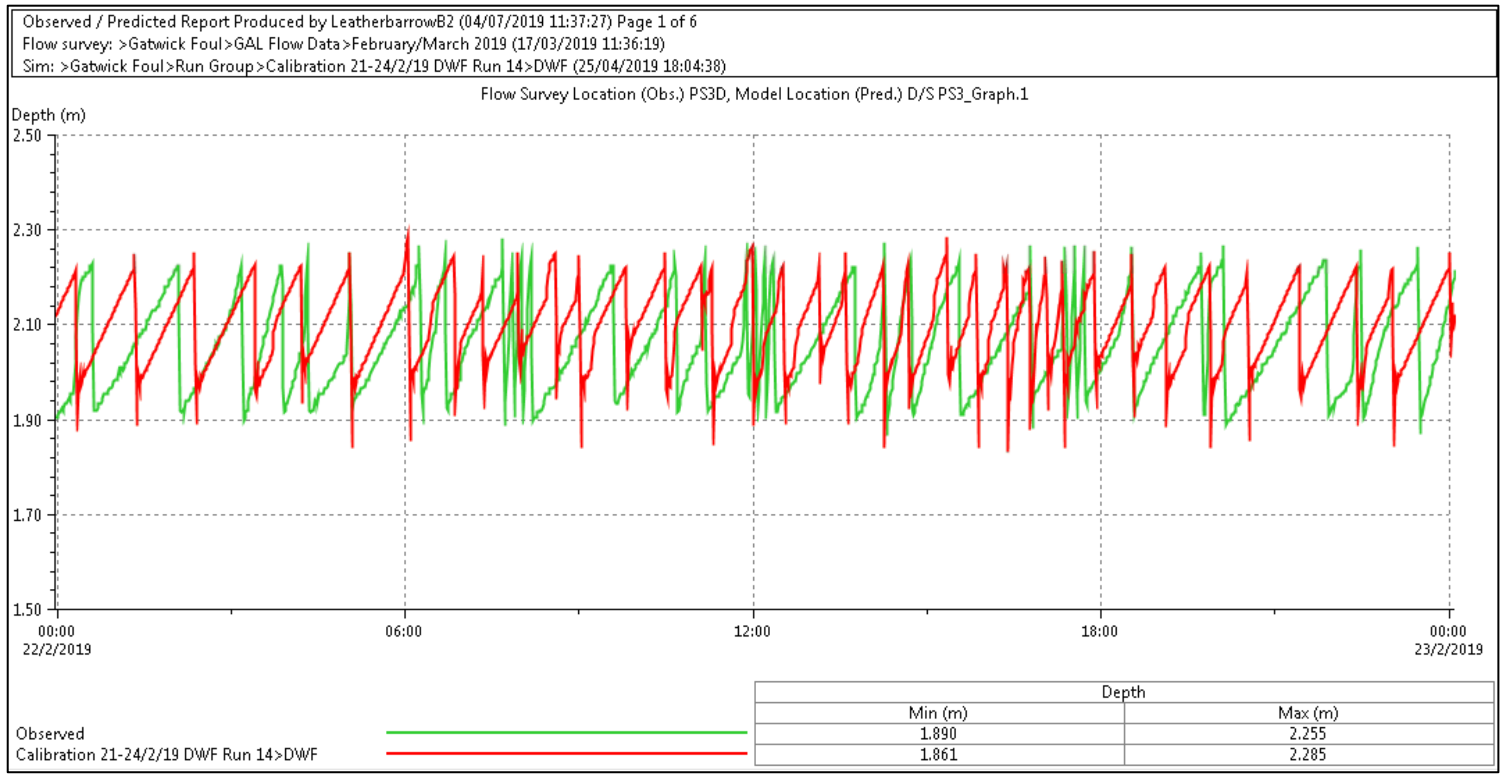


Figure 5.4.2: DM3 (PS3 Wet Well)

FMPS3 (PS3 Magflow Meter)

5.4.1 This flow monitor recorded the pumped flow at one minute intervals and was supplemented by pump operation data. The recorded data for this monitor was good although there was some variability in the flow rates due to the cycling of the three pumps, so the modelled flow rate used the average and this gave a reasonable match. The observed data records groups of pump cycles rather than more consistent timings and this is thought to be due to the operation of PS2 which must store flow and then pump to PS3 only four times per day. A further complication is that PS3 is inhibited from operating when PS7 is operating, and the model includes control logic to replicate this. The model achieves this to some extent, but the cycle pattern could not be matched exactly. The volume criteria was achieved for two of the four events, with 2 June 2019 only failing by 1% and on 9 June 2019 the observed data recorded twice the volume of all other dry days which suggests that there was still some post-rainfall infiltration in the network after the rainfall on the 8 June 2019. Overall the model has achieved a reasonable level of calibration for the dry days.

5.4.2 The data was good for the wet period, and the pump operation recorded by the Andover system indicated that the PS7 operating inhibit was over-ridden at times of high flow, so the model control logic was modified to replicate this. There was a considerable storm response here, with the largest increases in flow relative to average dry weather flow being recorded. The pattern of the response suggested that some was from PS2 as well as local to the pumping station. The observed data suggested that the storm response was a combination of rapid response due to runoff and a slower response due to groundwater infiltration. A contributing area of 0.2ha was added to the model at PS2 and 0.55ha at PS3 to replicate the fast response and the ICM groundwater infiltration routine was used to generate the slow response from the upstream catchment which was approximately 10ha in extent at PS2 and 16ha for PS3. A reasonable correlation was achieved in the flow profiles (no depth data available) and the model predictions met the peak flow and volume. The model achieved a reasonable wet period calibration here.

DM7 (PS7 Wet Well)

5.4.3 This depth monitor recorded depths in the wet well of PS7. The recorded data for this monitor was good for all events and the pump cycles were fairly consistent over the dry days. The pump cycles are heavily influenced by the operation of PS8 and to a lesser extent PS16, so the shapes of the cycles cannot be matched by the model due to the differing cycle times. A good match with the pattern of cycles and the depths was achieved and the depth criteria was met for all events. Overall the model achieved a reasonable level of calibration for the dry days.

5.4.4 The data was good. The onset of rainfall shows as increased cycles and more rapid filling of the wet well which is reasonably well replicated in the model, although the modelled pump tended to run for longer periods in each cycle compared to the observed data which shows rapid cycles, but this may be a function of the disparate data collection and modelling timesteps: this is discussed under FMPS7. The model met the depth criteria.

FMPS7 (PS7 Magflow Meter for 2 pumps)

5.4.5 This flow monitor recorded the pumped flow from both Duty pumps at one minute intervals and was supplemented by pump operation data. Calibration was performed in conjunction with the depth data in the wet well (DM7). The recorded data for this monitor was good and it was noted that Pump 1 generally ran at a flow rate of 26l/s (with some occasional higher rates), whereas Pump 2 ran at 31l/s (also with some higher peaks). A good match with the pattern of cycles and the flows was achieved and the volume criteria was met for both pumps for all but Pump 1 for one event where the failure was only by 1%. Overall the model has achieved a good level of calibration for the dry days.

5.4.6 The data was good for the duration of the wet period, although the Pump 1 rate was often higher than observed during the dry days (32l/s vs 26l/s). The observed data suggested that the storm response was a combination of rapid response due to local runoff and a slower response due to groundwater infiltration and contributions from PS8. The model already included a contributing area of 0.45ha for the highway drainage connected at Timberham Farm Road, but this was insufficient to replicate the fast storm response so this was increased to 1.25ha. The ICM groundwater infiltration routine was used to generate the slow response from the upstream catchment which was approximately 23ha in extent. A good correlation was achieved in the flow and depth profiles and the model predictions met the flow volume and depth criteria, but not the peak flow for Pump 1 which was retained at the rate observed during dry weather. The model achieved a reasonable wet period calibration here.

FMPS8 (PS8 Magflow Meter)

5.4.7 This flow monitor recorded the pumped flow at one minute intervals and was supplemented by pump operation data. Calibration was performed in conjunction with the depth data in the wet well (OnSite DM8). The recorded data for this monitor was good and the pattern of cycles reflected the passenger throughput except that every two days there was a flushing test which inhibited the pump for a short period and which then ran to pump the stored flow and drew down the wet well to a lower level than normal. This cycle could not be replicated in the model and it was not taken into account in the assessment. A good match with the normal pattern of cycles and the flows was achieved and the volume criteria was met for all events, as was the flow criteria except during the flushing cycles. Overall the model has achieved a good level of calibration for the dry days.

5.4.8 The data was good for the duration of the wet period, although the pump rate was often lower than observed during the dry days (28l/s vs 33l/s). The observed data suggested that the storm response was a combination of rapid response due to local runoff and a slower response due to groundwater infiltration. A contributing area of 0.4ha was added to the model to replicate the fast response. The ICM groundwater infiltration routine was used to generate the slow response from the upstream catchment which was approximately 31ha in extent. A good correlation was achieved in the flow profile and the model predictions met the peak flow and flow volume criteria. The model achieved a reasonable wet period calibration here.

FMPS19 (PS19 Magflow Meter)

5.4.9 This flow monitor recorded the pumped flow at one minute intervals, and was supplemented by pump operation data. Calibration was performed in conjunction with the depth data in the wet well (OnSite DM19). The recorded data for this monitor was good but the pumping rate was quite variable due to cycling between the installed pumps which had different ratings, so the model used the average rate. The model was calibrated against the peak passenger day of 2June 2019 and a good match for volume was achieved. Overall the model has achieved a good level of calibration for the dry days.

5.4.10 The data was good for the duration of the wet period, although the pump rate was often higher than observed during the dry days (21l/s vs 19l/s). The observed data suggested that the storm response was a combination of rapid response due to local runoff and a slower response due to groundwater infiltration. A contributing area of 0.3ha was added to the model to replicate the fast response. The ICM groundwater infiltration routine was used to generate the slow response from the upstream catchment which was approximately 12ha in extent. A good correlation was achieved in the flow profile and the model predictions met the flow volume criteria but not for peak flow as the model retained the predominant pump rate from the dry weather flow period. The model achieved a reasonable wet period calibration here.

DM23 (PS23 Wet Well)

5.4.11 This depth monitor recorded depths in the wet well. The recorded data for this monitor was good for all events and the pump cycles were fairly consistent over the dry days. A good match with the pattern of cycles and the depths was achieved and the depth criteria was met for all events. Overall the model has achieved a good level of calibration for the dry days.

5.4.12 The data was good although it was noted that the pump operating levels varied more than in the dry days, with frequent depths above and below the nominal settings. The onset of rainfall shows as increased cycles and more rapid filling of the wet well which is reasonably well replicated in the model: this is discussed under FMPS23. The model met the depth criteria.

FMPS23 (PS23 Magflow Meter)

5.4.13 This flow monitor recorded the pumped flow at 1 minute intervals and was supplemented by pump operation data. Calibration was performed in conjunction with the depth data in the wet well (DM23). The recorded data for this monitor was good but the pumping rate was quite variable due to cycling between the installed pumps which had different ratings, so the model used the average rate. As a result, the model failed to replicate the peak flow criteria which is assessed against the few higher recorded peaks, but overall the model matched for many cycles. Despite this, the model matched the assessment criteria for peak flow for all events and also matched the volume criteria for all but one event where the failure was by only 1%. Overall the model has achieved a good level of calibration for the dry days.

5.4.14 The data was good for the duration of the wet period. The observed data suggested that the storm response was a combination of rapid response due to local runoff and a slower response due to groundwater infiltration. A contributing area of 0.12ha was added to the model to replicate the fast response. The ICM groundwater infiltration routine was used to generate the slow response from the upstream catchment which was approximately 19ha in extent. A good correlation was achieved in the flow profile and the model predictions met the flow volume and peak flow criteria. The model achieved a reasonable wet period calibration.

DM40 (PS40 Wet Well)

5.4.15 This depth monitor recorded depths in the wet well and was supplemented by pump operation data. Calibration was performed in conjunction with the flow data in the downstream sewer (OnSite FM3). The recorded data for this monitor was good but showed that there were issues with the pumps, as the number of pump starts recorded exceeded the number of pump cycles recorded by the level monitor and on 4and 9 June 2019 the levels rose higher than usual, particularly for the latter where the pumps appear to have failed for over 2 hours. The pump operation is also over a very short operating depth of 120mm which results in a very high frequency of cycles. Notwithstanding this, the model gave a reasonable match to the observed depth data and for normal operating conditions the model met the criteria. Pumped flows are covered under FM03. Overall the model has achieved a good level of calibration for the dry days.

5.4.16 The data was good for the duration of the wet period. The observed data was inconclusive regarding a storm response, as the number of cycles increased compared to the dry days, but it was noted that the depth range for each storm period cycle was shorter which suggested that a different Duty pump was operating. As there was no definite storm response, the model was not amended.

5.5 Model Calibration Summary

5.5.1 In summary a good dry day calibration was achieved, with the critical volume, peak flows and peak depths being well replicated for most of the events at all locations. The model is considered to be suitable for assessing the growth of dry weather flows in the foul sewer network.

5.5.2 The storm calibration was challenging due to the lack of detail in the rainfall data, the small rainfall intensities recorded and the operation of the pumps which made interpretation difficult. There was a demonstrable response to rainfall at virtually all locations and it was evidently due to a combination of fast and slow responses, although the relative contribution cannot be confirmed due to the lack of temporal granularity in the rainfall data. Nevertheless, the assumptions made on the sources of storm response generated a reasonable match with the observed data and the model is considered to be suitable for assessing the impact of storm flows in the foul sewer network, although the confidence in this aspect of the model is lower than for the dry weather flow.

6 Strategies to Meet Future Baseline and Project Demand

6.1 Introduction

6.1.1 In order to test the capacity of the wastewater system, models were created with the projected busy day passenger data for the future baseline and Project scenarios for various timeframes. All foul flows which are impacted by increased passenger numbers (for example, hotel occupancy and worker numbers) were increased in line with the passenger number increase.

6.1.2 Since the calibration process indicated that in most areas the network received rainfall runoff, an allowance was required within the model to account for this. Given that the rainfall during the flow survey period has been calculated as a 25% (1 in 4) AEP event, the peak observed storm response has been derived for each of the calibration locations during that event and the flows have been extrapolated to the equivalent of a 3.33% (1 in 30) AEP event using the hydrological characteristics of the Gatwick area. The 3.33% (1 in 30) AEP event is typical of the maximum magnitude of storm that a sewer system can reasonably be expected to cope with without flooding.

6.1.3 The peak inflow representing the rainfall runoff has been applied in the model as a constant inflow over the full 24 hour period so that all peak dry weather flows are coincident with peak storm inflows. This is considered to be a reasonable approach to determine the peak response of the network as it is conservative estimate of storm flow volumes as they will be over-predicted.

6.1.4 Mitigations will be secured as a requirement in Schedule 2 of the DCO (Doc Ref. 2.1) .

6.2 Future Baseline

6.2.1 The model was run with the additional future flows that would result from future developments at Gatwick outside the Project. The findings of the assessment on the existing foul sewer system were as follows.

- There are plans to replace the pumps in PS40, but the proposed 26l/s pumping capacity exceeds the capacity of the 150mm diameter gravity pipe to which the pump currently discharges, so there is a need for a new rising main discharging to the 375mm diameter trunk sewer.
- The future baseline flows to PS3 are not predicted to increase much as the only significant change would be

increased passengers from Pier 6 which does not generate a large increase in flows. On this basis there is no requirement to uprate the pumps.
- There is no intention to address gravity pipes with reverse or slack gradients as these do not impact the ability of the network to convey flows, although they do result in minor surcharging at peak times in wet weather so could have an operational impact such as sedimentation due to low velocities.

PS40 – Provision of New Pumps and Rising Main

6.2.2 The uprating of the two PS40 pumps within the existing wet well to 26l/s as Duty/Standby would require a new rising main as the flow rate is too high for the existing gravity branch sewer downstream, so the main could be extended to manhole FW28414403 on the 375mm diameter trunk sewer. This is shorter than the current pumping main/gravity pipe combination as it can connect into the trunk sewer upstream of the current connection as it does not need a positive gradient. The main would be a pipe 228m long, 100mm diameter laid at an average depth of 1.12m or lower if greater protection or avoidance of existing services is required. Given the high velocity a hard-wearing high-density polyethylene material would be required which is also flexible enough for threading through existing services. The route of the proposed main is shown in Figure 6.3.1

6.3 With-Project case:

For the Project case, there are several adjustments to the foul water system that would be required, which are summarised as follows:

- Diversion of flows from PS7 to a new PS, PS7A: This would require consideration of the pumping rate to be installed, the impact on the existing pumping mains and changes to the gravity system to divert the flows to the new wet well.
- Replacement of PS3 with PS2A: This would require consideration of the pumping capacity. It has been assumed that the rising main would connect into the existing rising main currently serving PS3.
- As for the future baseline case, the upgrade to PS40 would require a new extended rising main, as described above, and indicated in Figure 6.3.1
- Upgrade of PS6: This would require either a new extended rising main, or the existing gravity sewer downstream to be upgraded Diversion of the flows from east of the railway line at the South Terminal to Crawley STW and construction of a new pumping station; the proposed East pumping station.
- These adjustments to the system could be made, as outlined above, both to relieve the current assets and also to limit the impacts on TW assets. The model has been used to test and develop technically feasible sewer and pumping station upgrade options that achieve target headroom/capacity for the future baseline and the Project scenarios. The detail of each of the proposed system upgrades is described below.

Proposed Pumping Station 7A

6.3.1 The proposed pumping station would be located on an area of grass verge on the northwest corner of the junction between Cargo Forecourt Road and the airside access road from the fuel farm. In order to divert the flows to the new wet well, the existing inlet pipes at PS7 from the west, south and east would require diverting via new pipes running along the north side of Cargo Forecourt Road and an additional connection would be required from the 300mm diameter sewer running down the fuel farm access road. In total the diversion pipes total approximately 110m in length, mostly at 300mm diameter and an average depth of 3.2m. In addition, 3 no. new 1.2m diameter manholes would be required also at an average depth of 3.2m. The route of the proposed diversions is shown on Figure 6.3.2.

6.3.2 The optimum pump has been determined as a single variable speed unit rated at 76l/s discharging into a valve chamber downstream of which the flows split to the two existing pumping mains. This includes an allowance of flow from PS2A of 17l/s injected into the west main. To meet the GAL resilience specification, 4 no. pumps are proposed as Duty/Standby/Standby/Standby on a rotating duty, with fixed speed backup. The wet well is proposed as 5m diameter to accommodate the pumps, with a sump level 5.5m below ground level.

Proposed Pumping Station 2A

6.3.3 Since most of the storm inflow and a proportion of the foul inflow to the existing PS3 would either be abandoned or diverted (Virgin Hangar area), the capacity of the replacement PS2A would be much reduced and a pumping rate of 17l/s is adequate as it is only receiving flows from PS2 and PS45. The rising main only needs to be 100mm internal diameter and this would connect into the existing main serving PS3 at the valve chamber which can be retained. There would be low velocities in the existing main due to the much lower pump rate.

Proposed Upgrade PS40

6.3.4 The proposals set out for the uprated PS40 and mains under the future baseline solution (paragraph 6.2.2) apply equally to the Project.

PS6 – Provision of New Pumps and Rising Main

6.3.5 The uprating of the pumps utilising the existing wet well and valve chambers to 10.5l/s would require a new rising main but the flow rate is too high for the existing 100mm diameter gravity branch sewer downstream, so the main could be extended to the inlet to PS7 in the short term, or to the diversion manhole to be constructed taking flow from the PS7 inlet to PS7A (paragraph 6.3.1). The main would be a pipe 340m long, 80mm diameter laid at an average depth of 1.6m and the route along Cargo Forecourt Road as shown in Figure 6.3.3.

Proposed East Pumping Station

6.3.6 A new pumping station to the east of the railway would be provided to decouple the existing sewerage network east of the railway and remove its load from the South Terminal sewerage system. This would include a new underground pipeline connection between the new pumping station and the Crawley STW. The indicative corridor of the pipeline route has been designed to avoid the ancient woodland and make use of existing tracks. It would be up to 1,270 metres in length and require a construction corridor of up to 10m wide to install. The detail of the routing would be subject to LPA approval.

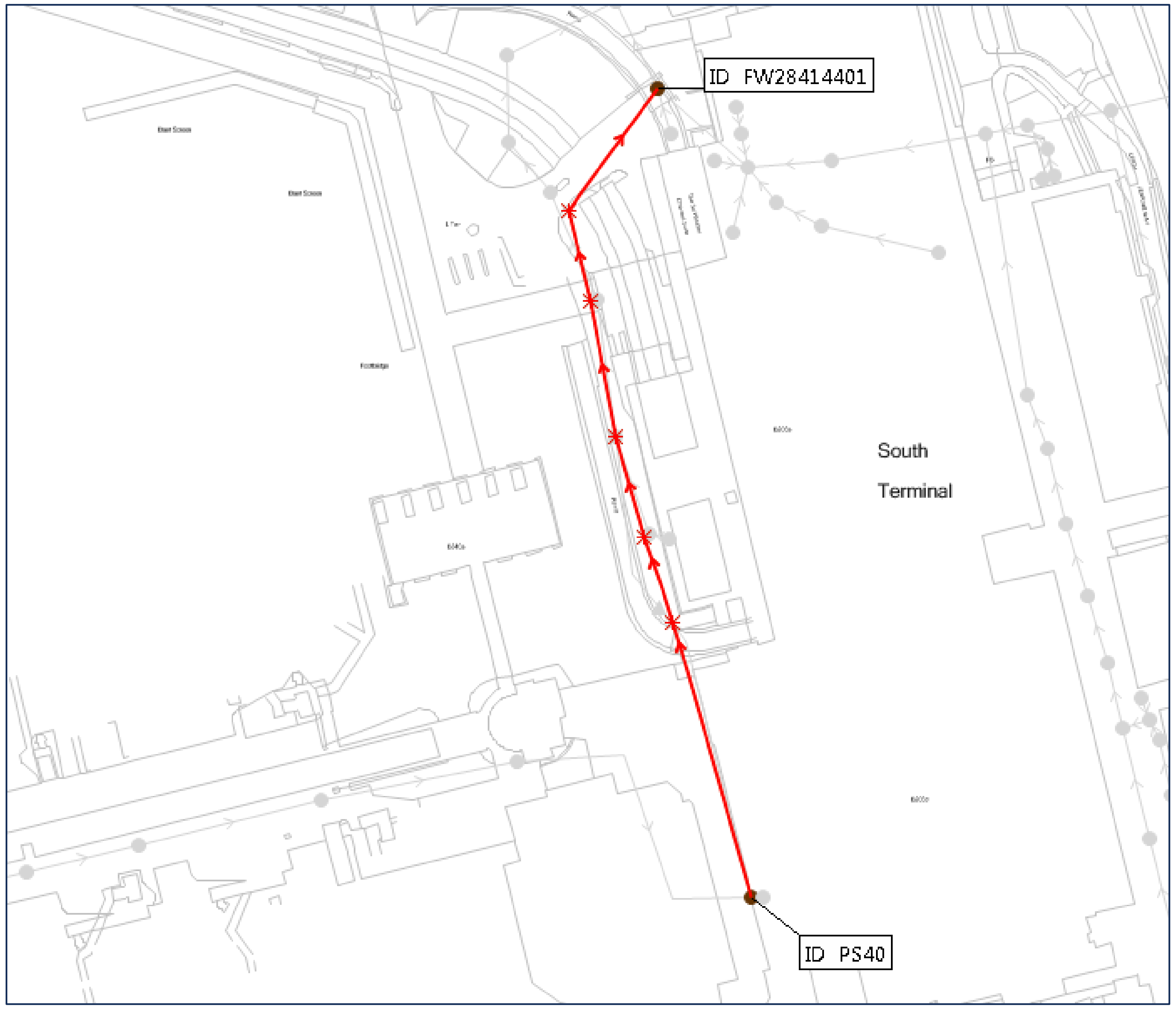


Figure 6.3.1: Proposed route of extended pumping main from PS40

Figure 6.3.2: Proposed sewer diversions to PS7A

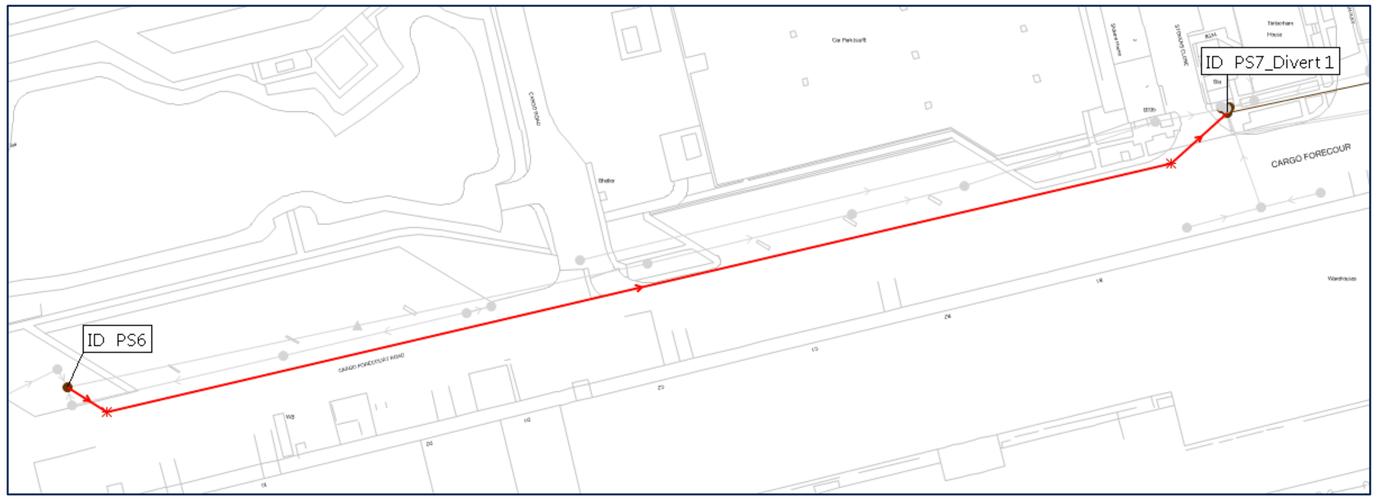


Figure 6.3.3: Proposed route of new pumping main from PS6

7 Summary, Conclusions and Recommendations

7.1.1 A hydraulic model has been constructed in InfoWorks ICM software to represent the foul sewer system serving the Gatwick estate. The model was built primarily from the GIS and old survey data supplemented by more recent survey data in limited locations and pumping station data taken from the Andover SCADA system or from GAL operational teams. All sources of foul discharges have been incorporated into the model as discrete discharges to an appropriate level of detail. The model is necessarily simplified as the network is complex, especially in the South Terminal area, but all the main gravity sewers and pumping stations have been modelled.

7.1.2 A flow survey was conducted over a week long period in May/June 2019 and a calibration exercise was completed that yielded reasonable results that met industry standards in most locations. A storm response was also recorded in most locations monitored by the flow survey. A 25% (1 in 4) AEP storm event occurred at the end of the survey period which enabled a basic calibration exercise of these storm inflows, although the lack of detailed rainfall data prevented a more thorough assessment. Nevertheless, the calibration met the industry standard for most locations, although there is a lower confidence in the model predictions for storm flows. Notwithstanding this, there is sufficient confidence in the storm event modelled flows to inform the extent of system upgrades that would be required.

7.1.3 The model was used to predict the system response to peak passenger flows recorded on 24th August 2018 and the predictions showed very little stress in the system in dry weather. In order to account for any storm response, a parallel model was run with a constant discharge, where this was observed during the flow survey undertaken in May/June 2019. This indicated that the system was stressed at PS7 and PS3 where the storm responses are strong. Total inflows exceed the pumping capacity during the peak, but no flooding is predicted. Some of the pumping stations exhibit higher hourly pump starts than would be expected, notably at PS40. However, this is not a concern as this was a deliberate operational response in order to keep flow moving and prevent deposition of fats and greases.

7.1.4 Overall, the model development, calibration, and testing, provides a sufficient level of certainty to understand the future constraints on the system, both for the future baseline scenario and the Project scenario. System upgrades have been identified to support the ES as summarised in Section 8.

8 Environmental Statement Assessment

8.1 Introduction

8.1.1 For the assessment presented in **ES Chapter 11: Water Environment** (Doc Ref. 5.1), the model as described in Sections 1-7 of this appendix has been updated to reflect the projected increases in discharges during the various stages of the Project. The impacts have been assessed in terms of exceedance of available capacity and consequent flooding compared to the future baseline case, taking account of the proposed mitigation works to be implemented as part of the Project (see Section 11.8 of the **ES Chapter 11: Water Environment** (Doc Ref. 5.1)).

8.1.2 Wastewater improvements to the foul sewer system as part of the Project would include the following:

- Construction of new pumping station 7A (PS7A) to replace existing facility PS (PS7) to provide additional capacity;
- Construction of new pumping station 2A (PS2A) to replace existing facility PS (PS3) to provide reduced capacity for smaller drainage area; PS40 upgrades – uprated pumps at existing PS40, with new rising main and gravity sewer;
- Replacement of pumps and pumping main at pumping station PS6 to provide additional capacity; and
- Construction of a new pumping station on the east side of the Brighton-London mainline railway to convey all foul flows from this area to Crawley STW to relieve the gravity outfall pipe discharging to TW Horley STW sewer network.

8.1.3 The potential impact on the foul sewer system is flooding arising from increased flows in the network exceeding the available capacity. This could disrupt airport operations, particularly in and around the terminal buildings.

8.1.4 The assessment of potential effects is limited to the supporting infrastructure at Gatwick.

8.1.5 The capacity of the public sewer network to which the private Gatwick wastewater system discharges and the downstream STW is the responsibility of TW under the terms of its license as the statutory authority. Discussions with TW are ongoing to agree the quantity and distribution of discharges from the airport in the future.

8.1.6 The local sewerage undertaker: TW, as part of their long-term planning, will undertake an assessment of the impact of wider projected growth in the local area on their sewage treatment works at Horley and Crawley, which would include the impact of the Project. If capacity issues are identified, TW would be responsible for reinforcing their network to support development and they would recoup their costs through infrastructure charges to Gatwick.

8.2 Passenger numbers

Four design horizons were simulated for both the future baseline and the Project (see Table 8.2.1). These passenger numbers are based on the assumption that there is no new runway at Heathrow and a busy day as a conservative approach.

Table 8.2.1 Busy Day Passenger numbers

Busy Day PAX	2029	2032	2038	2047
Future baseline	178,262	182,056	187,055	193,855
With Project	189,569	217,265	226,919	236,056
% difference	6.3%	19.3%	21.3%	21.8%

8.3 Assessment of Impact

Climate Change

8.3.1 Climate change has the potential to cause rainfall of increased depth, frequency and intensity to occur compared to the present rainfall patterns. As a result, storm runoff from the small contributing areas discharging to the foul sewer system would increase the flows in the network and potentially exceed the capacity of the gravity sewers or pumping stations.

8.3.2 The potential impact of climate change was tested using the 2047 flows for the future baseline and the Project scenarios. This provides the worst-case combination of passenger flows and climate change. The Environment Agency predicts a central potential increase in precipitation of 20 per cent for the 2050's epoch. An increase of 25% in rainfall intensity has been adopted as a conservative estimate of the predicted impact of climate change. Therefore, the storm flows were increased by this percentage and the performance of the system was compared to the equivalent future baseline. The absolute impact was also assessed.

8.3.3 The climate change increase to the storm flows increases the peak flows in the foul sewer system by approximately 11 per cent: for the Project scenario compared to the Project without climate change. As a result, there are some minor increases to surcharging of the gravity pipes, and the pumps have to run for longer in order to deal with the flow, but there is no predicted flooding or significant detriment to the operation of the network. Compared to the 2047 future baseline (i.e. without the Project) with the same rainfall uplift applied, the total flows are 5 per cent lower in the Project scenario and the predicted stress on the network is considerably less due to the proposed mitigation works and changes in land use associated with the Project which would divert storm flow out of the foul system.

8.3.4 The impact on the foul sewer system would be minor adverse as there is no predicted risk of flooding in the Project scenario, but the system would experience higher degrees of surcharge. As these factors are taken into account in the assessment process, no additional changes to the assessment are anticipated as a result of climate change.

Peak construction (2026)

8.3.5 Discharges to the wastewater network by construction workers and construction activities are estimated to increase the peak system loading by 1 per cent.

8.3.6 The magnitude of impact of the construction on the Gatwick wastewater network has been assessed as negligible adverse with an effect of negligible adverse and would not be significant.

First Full Year of Opening: 2029 (up to 2032)

8.3.7 The first full year of opening in 2029 would see peak daily passenger numbers increase by approximately 6 per cent in the Project scenario compared to the 2029 future baseline and 14 per cent compared to the 2018 baseline. The increase in foul water flows would add to the foul system loading throughout the network so would have a potential long-term impact on the foul drainage system.

8.3.8 Compared to the 2029 future baseline, the Project flows (including storm flows) would be 7 per cent lower due to the proposed mitigation works and changes in land use associated with the Project which would divert storm flow out of the foul system. The maximum flows are shown in Table 8.3.1.

8.3.9 No flooding is predicted in the 2029 simulations.

8.3.10 The magnitude of impact on the Gatwick wastewater infrastructure network would be negligible adverse resulting in a negligible adverse effect, which is not considered to be significant. This is due to the wastewater network having adequate capacity to accommodate the increase in flows as a result of additional passengers and the demand from construction workers.

Table 8.3.1 Maximum flows for future baseline and Project scenarios

Outflow (m^3/s)	2018 Baseline	2029	2032	2038	2047	2047 + Climate Change
Future baseline	0.131	0.155	0.155	0.156	0.157	0.176
With Project	0.131	0.144	0.148	0.150	0.152	0.168

Interim Assessment Year: 2032 (up to 2037)

8.3.11 The interim assessment year 2032 would see peak daily passenger numbers increase by approximately 19 per cent compared to the 2032 future baseline and 31 per cent compared to the 2018 baseline. The increase in foul water flows would add to the foul system loading throughout the network so would have a potential low long-term impact on the foul drainage system.

8.3.12 Compared to the 2032 future baseline, the Project flows (including storm flows) would be 4 per cent lower due to the proposed mitigation works and changes in land use associated with the Project which would divert storm flow out of the foul system. The maximum flows are shown in Table 8.3.1.

8.3.13 No flooding is predicted in the 2032 simulations.

8.3.14 The foul sewer system has adequate capacity to accommodate the increase in flows. Therefore, the magnitude of impact on the Gatwick wastewater infrastructure network would be negligible adverse resulting in a negligible adverse effect, which is not considered to be significant. This is due to the wastewater network having adequate capacity to accommodate the increase in flows as a result of additional passengers and the demand from construction workers.

Design year: 2038

8.3.15 2038 would see peak daily passenger numbers increase by approximately 21 per cent compared to the 2038 future baseline and 37 per cent compared to the 2018 baseline.

8.3.16 Compared to the 2038 future baseline, the Project flows (including storm flows) would be 4 per cent lower due to the proposed mitigation works and changes in land use associated with the Project which would divert storm flow out of the foul system. The maximum flows are shown in Table 8.3.1.

8.3.17 No flooding is predicted in the 2038 simulations.

8.3.18 The modelling results show that the proposed infrastructure is of sufficient capacity for the projected flows, so it is considered that the magnitude of impact would be negligible adverse resulting in a negligible adverse effect, which is not considered to be significant.

Highways opening +15 years (2047)

8.3.19 2047 represents the long term forecast year and to meet a specific requirement of guidance in the Design Manual for Roads and Bridges to assess impacts 15 years after the last of the key highways works associated with the Project are due to be completed.

8.3.20 2047 would see peak daily passenger numbers increase by approximately 22 per cent compared to the 2047 future baseline and 42 per cent compared to the 2018 baseline.

8.3.21 Compared to the future baseline for 2047, the Project flows (including storm flows) would be 3 per cent lower for the wet weather cases due to the proposed mitigation works and changes in land use associated with the Project which would divert storm flow out of the foul system. The maximum flows are shown in Table 8.3.1.

8.3.22 No flooding is predicted in the 2047 simulations.

8.3.23 The modelling results show that the proposed infrastructure is of sufficient capacity for the projected flows, so it is considered that the magnitude of impact is negligible adverse resulting in a negligible adverse effect, which is not considered to be significant.

8.4 Conclusion

8.4.1 In all the modelled timeframes and with climate change taken into account the magnitude of impact on the Gatwick wastewater infrastructure network would be negligible adverse resulting in a negligible to minor adverse effect, which would consequently not be significant. This is due to the wastewater network having adequate capacity to accommodate the increase in flows resulting from additional passengers and the demand from construction workers and staff, taking account of the proposed mitigation works to be implemented as part of the Project.

9 Glossary

9.1 Glossary of terms

Table 9.1.1 Glossary of terms

Term	Description
AEP	Annual Exceedance Probability
AOD	Above Ordnance Datum
DM	Depth Monitor
FM	Flow Monitor
GAL	Gatwick Airport Limited
Gatwick	Gatwick Airport
GIS	Geographical Information Systems
l/s	Litres per second
Magflow	Magentic Flow Meter
NAPI	Net Antecedent Precipitation Index
NPI	New Premier Inn
PS	Pumping Station
STW	Sewage Treatment Works
TW	Thames Water