A63 Castle Street Improvements, Hull
Environmental Statement

Volume 3 Appendix 11.5
ROAD DRAINAGE AND THE WATER ENVIRONMENT – PUMPING TEST REPORT

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A63 Castle Street Improvements, Hull

Environmental Statement

Appendix 11.5 Pumping test report

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1 Introduction

1.1 Background

This pumping test report supports the Groundwater Report (Document Number 1168-10-223-RE-001-PD1) and covers the construction of boreholes LDBH01 and LDBH02, and test pumping of LDBH01. Due to the expected sub-artesian conditions and high upwards pressures from the chalk groundwater, methods of construction for the proposed Mytongate Junction underpass during feasibility phases considered the possibility of dewatering the top of the chalk. There was insufficient existing information, however, to define parameters required to confirm the impacts of dewatering the junction. The conceptual earthworks design for Mytongate Junction is discussed in the Groundwater Report.

Therefore, Grontmij installed two new boreholes in the chalk and overlying drift close to Mytongate Junction, with a view to test-pumping both boreholes and analysing the results to determine aquifer parameters. Analysis of the test data and background monitoring would also provide an understanding of the interaction between chalk and drift aquifers and between the chalk aquifer and the Humber estuary.

1.2 Scope of This Report

This report details the following site investigation work:

- Construction of the chalk borehole LDBH01 and drift borehole LDBH02.
- LDBH01 pumping test, and analysis to derive aquifer properties.
- Water level observations during the pumping test monitoring period.
- Water quality sampling taken during the pumping test.
- Tidal Efficiency and lag time calculations for the chalk observation boreholes.

Analysis of the above investigations will help to further develop the conceptual model and inform the numerical modelling, and will further determine the degree of hydraulic continuity between the chalk and drift deposits.

2 Borehole Construction

2.1 Overview

Two new boreholes were drilled into the chalk (LDBH01) and overlying drift (LDBH02) specifically for the purpose of carrying out pumping tests. These are located approximately 30 m southeast of the Mytongate Junction. LDBH01 and LDBH02 are 2.5 m apart. The locations of the boreholes are shown on Figure 2.1.

2.2 Chalk Borehole LDBH01

LDBH01 was completed on 26 November 2013 to a depth of 50 m. It was constructed by means of auger drilling through drift deposits and rotary drilling through the chalk. Safety casing was used in case the Chalk proved artesian, although this was not the case.

The borehole was completed using 355 mm diameter steel casing to a depth of 32.7 m and is open hole (311 mm diameter) below this. Borehole completion was designed to target the effective chalk aquifer.

LDBH01 was developed upon completion, by means of airlift and pumping to remove accumulated sediment and to improve performance/reduce turbulent head loss. Water was discharged to tanker during development and was observed to be running clear after around 8 minutes of development.

The borehole headworks are located within a below-ground chamber, accessible via a 600 mm x 600 mm lockable manhole cover at ground level. The borehole is sealed with a 350 mm PN 16 flange and cover plate at 4.363 mAOD (0.311 mbgl). Dip access is via a 50 mm BSP socket within the cover plate, currently sealed.

Borehole records, as submitted to the British Geological Survey and including construction, geology and groundwater details are attached Annex 1 along with as built drawings, and a summary of the construction details are presented in Table 1. Photos of the borehole headworks and locations are also included in Annex 4.

2.3 Drift Borehole LDBH02

LDBH02 was completed on 13 November 2013 to a depth of 22m. It was constructed by means of augur drilling to a depth of 22 m.

The borehole was completed using 375 mm diameter uPVC casing to a depth of 14 m and 375 mm diameter uPVC screen from 14m to 18.5m. Borehole completion was designed to target sand layers observed when drilling, although these were minimal. No water strikes were observed during drilling, although a number of horizons were damp.

Further details are provided in Section 2.4.

LDBH02 was developed upon completion, by means of airlift and pumping to remove accumulated sediment and to improve performance/reduce turbulent head loss. Water levels were monitored within LDBH02 and the nearby ground investigation (GI) borehole BH25 during development pumping. Excessive drawdowns in LDBH02 were observed during development pumping at a rate of 0.3 l/s, resulting in the pump being switched off after only a matter of minutes (17 - 25 minutes). During these periods of development no drawdown was observed in BH25. This confirmed that it would not be possible to conduct a meaningful test on this borehole and the pumping test for LDBH02 was cancelled.

The borehole headworks are located within a below-ground chamber, accessible via a 600 mm x 600 mm lockable manhole cover at ground level. The borehole is sealed with a 150 mm PN 16 flange and cover plate at 4.281 mAOD (0.376 mbgl). Dip access is via a 50mm BSP socket within the cover plate, currently sealed.

Borehole records, as submitted to the British Geological Survey and including construction, geology and groundwater details are attached Annex 1 along with as built drawings, and a summary of the construction details are presented in Table 1. Photos of the borehole headworks and locations are also included in Annex 4.

Table 1: Borehole Construction Details

	LDBH01	LDBH02
Construction Date	26 Nov 13	14 Nov 13
Location (NGR)	509380.4 428334.3	509378.4 428332.9
Ground level (mAOD)	4.674	4.657
Total depth (mBGL)	50	22
Top and bottom of monitored aquifer unit (mbgl)	Chalk (28.6 – 50)	Glacial Till (11.3 - 19)
Monitored interval (mBGL)	32.7 – 50	14 – 18.5
Rest water level (mBGL)	1.48 – 6.31	0.34 – 0.95

2.4 Geological and Hydrogeological Observations

The Chalk is overlain by 28.6m of drift deposits at LDBH01 comprising cohesive alluvium, glacial till, glaciolacustrine and fluvio-glacial deposits. A summary of the geology encountered is provided in Table 2.

The granular alluvium, or any other significant sand layers, was found to be absent at the abstraction location despite having been identified in boreholes drilled to the north, east and south as part of GI (Geotechnics, 2013). It is possible that the abstraction boreholes have intersected the old channel of the River Hull, with the drift almost entirely comprising cohesive alluvium and glacial till.

Water-bearing sand bands were expected within the alluvium although these were not encountered by LDBH02. A number of damp horizons were encountered, however, within the cohesive alluvium and the glacial till.

Table 2: Summary of Geology Encountered

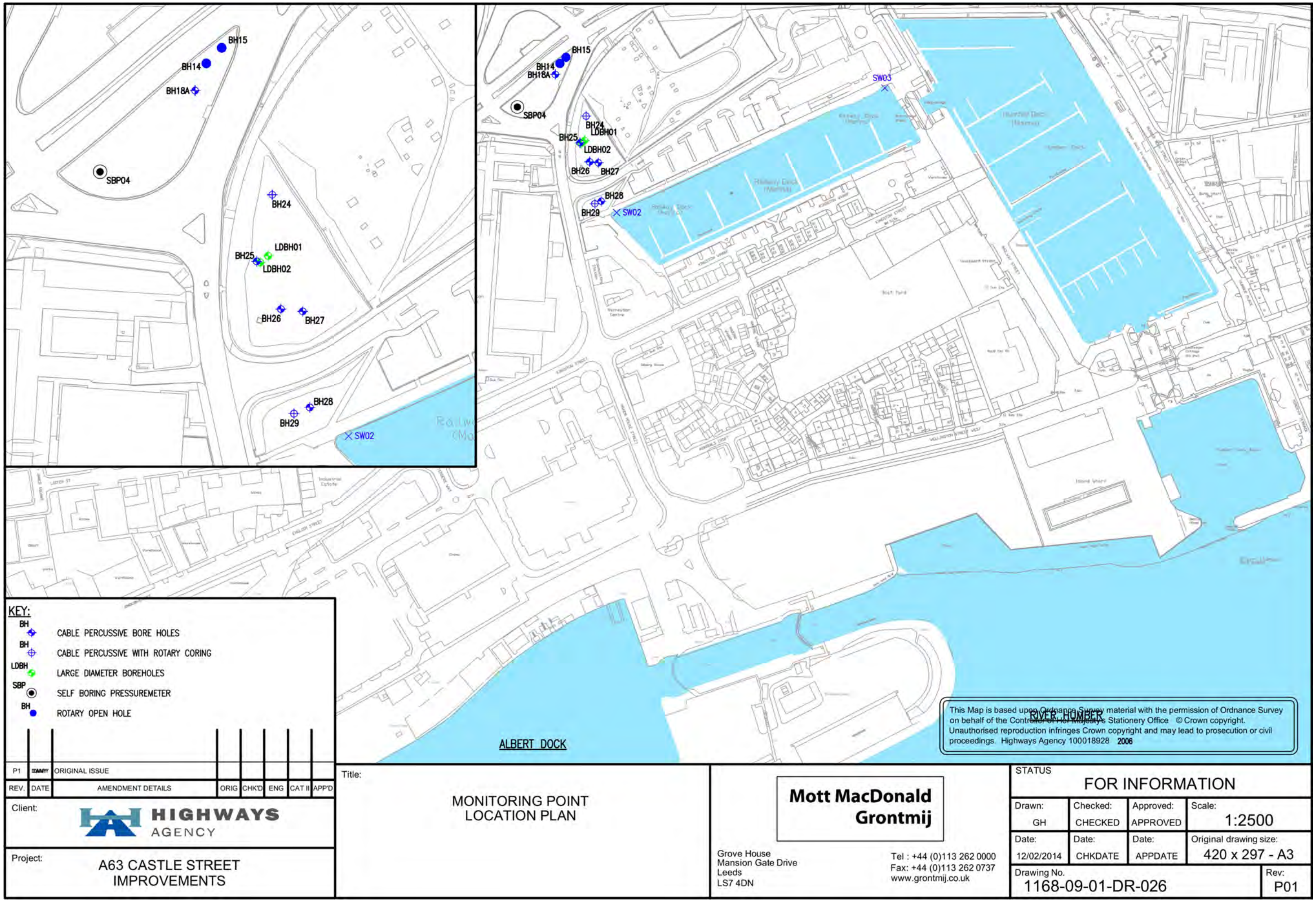
Table heading	Encountered Thickness	Description Summary
Made Ground	1.8 – 2.5m	Brown slightly silty, sandy, gravelly clay with bricks and rotten wood
Cohesive Alluvium	11 – 12.5m	Firm to soft dark brown and grey silts and silty clays, with sandy and silty peat lenses towards the base of the unit
Granular Alluvium	Absent	n/a
Glacial Till	3 – 5.5m	Firm to soft clay and sandy silty clay with chalk gravels
Glaciolacustrine deposits	6.7m	Stiff brown silty clays with fine sand laminations
Fluvio-Glacial deposits	4.3m (LDBH01 only)	Fine to course brown sand and sandy clay with flint and chalk gravel
Chalk	21m	Strong white chalk with large flint bands

The top 3 – 4 m of the chalk was found to be particularly unstable during drilling, with large amounts of infill occurring overnight.

Evidence of fracturing within the chalk was observed during drilling at a depth of 44 – 45 m, associated with a flint band.

Occasional water level measurements were taken during drilling of LDBH01. These indicated a large range in water levels within the chalk, coinciding with the tidal cycle. Further details of water level observations are provided in Section 4.

Figure 2.1 – Location Plan



3 Pumping Test

3.1 Overview

This section details the pumping test carried out at the chalk borehole LDBH01. As stated in Section 2.4, insufficient water was found in the drift horizons during drilling of the drift borehole LDBH02 and therefore a pumping test could not be undertaken from this borehole.

3.2 Programme of Works

The test programme for LDBH01 was as follows:

4 - 6 December 2013	Pre-test set up and calibration testing
9 – 12 December 2013	Pumping test
12 – 13 December 2013	Recovery test

Although the pumping test was originally planned to commence the day following calibration tests, a tidal surge occurred on the eve of 5 December 2013 resulting in much flooding of the surrounding area (see Annex 4). The pumping test was therefore delayed until all personnel could safely reach the site.

No other abstraction was occurring in the area before or at the time of the pumping test.

3.3 Test Design

The pumping test was designed based on a review of existing pumping tests conducted in the area. The nearest historic pumping test was at Hull Truck Theatre, approximately 0.9 km north of the site in 2005.

The review indicated that a pumping rate of around 1 Ml/d (11 l/s) was likely to be the maximum sustainable rate from the chalk, but an allowance was made for pumping between 0.5 Ml/d and 2 Ml/d (6 l/s and 23 l/s). The test was designed so that the abstraction borehole would be pumped at a constant rate for at least 3 days, or until a quasi-steady state condition (no significant increase in drawdown) had been achieved.

Whilst new boreholes were drilled specifically for the purpose of the pumping test, boreholes drilled as part of the Ground Investigation were utilised to monitor both the Chalk and overlying drift strata during the pumping test.

Grontmij's specialist subcontractor, PR Marriott Drilling Ltd, supplied the pumping test equipment including the pump, temporary pipework, electromagnetic flow meter and data loggers for the abstraction borehole and observation boreholes.

Regulator / Landowner Liaison

Approval from the following regulators and landowners was sought for the pumping tests:

- Environment Agency (Rich Greenley, Environment Management Team Leader, Land and Water: Derwent, Esk & Hull and Jane Gray, Environment Officer).
- Natural England – responsible for the adjacent Humber Estuary SAC.
- British Waterways Marines Ltd (BWML; Adam Nickerson, Buildings Surveyor) - landowners of Railway Dock (the proposed discharge point).
- Hull City Council – landowners of the site.
- Holiday Inn (Graham Fryer, Operations Manager) – lease holders of the site.

It was agreed with the Environment Agency that the pumping tests would comply with the Guidelines for Temporary Water Discharges from Excavations, with the following additional constraints:

- Water to be discharged to tanker until water runs clear and water quality proven to be comparable to routine groundwater samples taken in the Hull area, and with approval from the EA prior to commencement with the test.
- Water quality sampling at the point of abstraction and within the dock (at existing GI sampling points SW2 and SW3), including ammonium, selected metals and selected organics during the pumping tests
- Allowance for intermittent spot tests on ammoniacal nitrogen levels.
- The test is to be suspended should there be any significant deterioration in ammoniacal nitrogen, visual quality or odour of the discharge water.
- BWML to be informed of start and finish times of the test at allow for monitoring of dock levels.

Consultation with the following stakeholders was also undertaken:

- W&C
- Berth holders
- Dock masters

3.4 Pre-Test Set Up

Calibration

The pump was installed as deep as possible in the borehole with a stilling tube installed alongside to provide access for monitoring equipment.

Calibration tests were carried out at rates of 16 l/s and 23 l/s to confirm achievable pumping rates. These tests suggested that a pumping rate at 23 l/s would not be sustainable, however, and the test was therefore planned at a pumping rate of 18 l/s.

Water quality samples were taken during development and calibration to confirm water quality prior to discharge into Railway Dock during the pumping test.

After test set-up, water levels were left to recover for three days due to the flooding associated with the tidal surge.

Monitoring Equipment

Water level monitoring instruments (data loggers) were provided by PR Marriott Drilling Ltd and were installed in LDBH01 and additional boreholes in the surrounding area. Details of the measuring equipment are provided in Table 3.

Table 3: Instrumentation and Measuring Equipment

Instrument	Measurement	Further Details	Borehole
Druck PTX 1830	Borehole water level (automatic)	100m range	LDBH01
Druck PTX 1830	Borehole water level (automatic)	20m range	BH25
Druck PTX 1830	Borehole water level (automatic)	35m range	BH26 / BH24
Solinst Junior Levelogger Edge	Borehole water level (automatic)	10m range	BH18A, BH25, BH26, BH28, BH29
MicroDiver	Borehole water level (automatic)	50 mH_20 range	BH27
Electromagnetic flowmeter	Discharge rate (l/s)		LDBH01

Discharge Arrangements

Test water was discharged into Railway Dock, approximately 55 m to the south of LDBH01 (see Figure 2.1), via temporary pipework. This was considered to be sufficiently far away so as to prevent recirculation of groundwater. The discharge pipework was fitted with an anti-scouring skirt and extended to below the water level at Railway Dock. To further prevent scouring the pipework was kept away from the dock wall at the discharge point. Photos of the discharge point are included in Annex 4.

Permission to discharge to the Railway Dock was granted by the EA and BWML on 4 December 2013, following the return and screening of water quality results.

3.5 Water Level Monitoring Schedule and Water Level Datum

Electronic water level monitoring commenced at least six days prior to the test date and continued until after the test was complete (16 Dec 2013). Observation borehole details, including datums, monitoring horizon and monitoring period, are provided in Table 4. Locations of the observation boreholes are shown in Figure 2.1.

Table 4: Observation Borehole Details

Borehole	Location (NGR)	Dip Datum	Dip Datum Elevation (mAOD)	Monitoring Horizon (mbgl)	Monitoring Method	Monitoring Start	Monitoring End
BH14	509361 428391	Top edge of cover (~ground level), taken from edge across from hinge	3.6	10.5 – 12.3 (Cohesive Alluvium)	Manual	09/12/2013 10:26	13/12/2013 10:34
BH15	509363 428393	Top edge of cover (~ground level), taken from edge across from hinge	3.55	13 – 16 (Glacial Till)	Manual & automatic	27/11/2013 13:45	16/12/2013 12:46
BH18A	509357 428383	Top edge of cover (~ground level), taken from edge across from hinge	3.52	27 – 40 (Chalk)	Manual & automatic	27/11/2013 13:45	16/12/2013 12:49
BH24	509380 428352	Top edge of cover (~ground level), taken from edge across from hinge	4.896	34.5 – 47.2 (Chalk)	Manual & automatic *	27/11/2013 10:32	13/12/2013 10:45
BH25	509376 428332	Top edge of cover (~ground level), taken from edge across from hinge	4.647	14.5 – 17.2 (Glacial Till)	Manual	09/12/2013 08:41	13/12/2013 10:47

Borehole	Location (NGR)	Dip Datum	Dip Datum Elevation (mAOD)	Monitoring Horizon (mbgl)	Monitoring Method	Monitoring Start	Monitoring End
BH26	509383 428318	Top edge of cover (~ground level), taken from edge across from hinge	4.543	13.5 – 15.5 (Glacial Till	Manual	27/11/2013 08:00	13/12/2013 10:48
BH27	509389 428317	Top edge of cover (~ground level), taken from edge across from hinge	4.398	6.2 – 10.2 (Cohesive Alluvium)	Manual & automatic	27/11/2013 12:45	16/12/2013 12:28
BH28	509391 428289	Ground level (straight edge across top of borehole)	4.476	13.5 – 17 (Glacial Till)	Manual & automatic	09/12/2013 10:10	13/12/2013 10:41
BH29	509387 428287	Top edge of cover (~ground level), taken from edge across from hinge	4.5	36 – 50 (Chalk)	Manual & automatic	27/11/2013 14:15	16/12/2013 12:23
SBP04	509330 428359	Top edge of cover (~ground level), taken from edge across from hinge	3.49	4 – 7 (Cohesive Alluvium)	Manual	09/12/2013 10:30	13/12/2013 10:37

The water level logger originally installed in BH24 was found to be faulty. This was replaced on 9 December 2013. An additional diver data logger was also installed between 5 and 9 December 2013. The water level record presented for this borehole, although not entirely complete, is considered representative.

LDBH01 was installed with a water level logger on 4 December 2013 11:50. This was left in place until 13 December 2013 14:52.

Water levels within Albert Dock, approximately 350m south of LDBH01 (see Figure 1), have also been provided by ABP-Humber Estuary Services (www.humber.com) for the monitoring period. These are considered to be representative of tidal levels in the Humber Estuary. The monitoring data contains a number of gaps, notably 1 December, 7 – 8 December and 14 – 15 December 2013.

During the test, manual water level and flow measurements were scheduled as given in Table 5 below.

Monitoring of LDBH01 was undertaken by PR Marriott Drilling Ltd (manual water level dip readings, data logger, and flow recordings). Monitoring of the observation boreholes was undertaken by Grontmij and PR Marriott Drilling Ltd.

Table 5: Schedule of water level and discharge measurements

Manual water level and flow meter readings	0 to 5 minutes	30 second intervals
	5 to 10 minutes	1 minute intervals
	10 to 20 minutes	2.5 minute intervals
	20 to 30 minutes	5 minute intervals
	30 to 100 minutes	10 minute intervals
	100 to 180 minutes	20 minute intervals
	180 to 360 minutes	30 minute intervals
	360 to 1080 minutes	1 hour intervals
Electronic Water Level	1 minute intervals throughout the test	

Manual water level measurements were made to the top of the borehole covers (dip datum). These had been surveyed prior to the test. Elevation details are provided in Table 4.

3.6 Water Quality Sampling Schedule

Water quality samples were taken from LDBH01, two surface water sampling points and two observation boreholes, as summarised in Table 6 and shown in Figure 2.1. Samples were taken on a total of nine days, as summarised in Table 7, for major ions, the nitrogen suite, major metals, PAHs, TPHs and other organics.

Water quality samples were analysed by SAL Laboratories.

In situ analysis for alkalinity, pH, electrical conductivity, dissolved oxygen (DO), oxidation reduction potential (ORP) and temperature were taken from LDBH01 during the pumping test by PR Marriott Drilling Ltd. Spot samples of ammoniacal nitrogen were also taken on site from LDBH01, SW2 and SW3 using colorimetric test strips. The results are given in Section 5 below.

Table 6: Water Quality Sampling Locations

Sample Point	Location Description	Sampling method	Location
LDBH01	Abstraction point	Sample tap installed on pump rising main / temporary submersible pump	509380 428334
SW2	Discharge point, Railway Dock	Grab sample	509403 428280
SW3	Downstream point of Railway Dock	Grab sample	509603 428373
BH24	17.8m NE of LDBH01	Temporary submersible pump	509380 428352
BH29	48.0m S of LDBH01	Temporary submersible pump	509387 428287

Table 7: Schedule of Water Quality Sampling

Date	Description	LDBH01	SW2	SW3	BH24	BH29
Aug 13 - Oct 13	Pre construction (taken as part of GI)		ü	ü	ü	ü
27 Nov 13	Development test at 12l/s	ü				
27 Nov 13	Development test at 23l/s	ü				
28 Nov 13	Background chalk water quality				ü	ü
4 Dec 13	Background Surface water quality		ü	ü		
6 Dec 13	Following Tidal Surge [1]	ü [1]				
9 Dec 13	Test, Day 1	ü	ü	ü		
10 Dec 13	Test, Day 2	ü				
11 Dec 13	Test, Day 3	ü	ü	ü		
17 - 18 Dec 13	Post pumping test	ü [2]	ü	ü	ü	ü

Notes:

[1] Reduced suite of major ions (including chloride) and nitrates only.

[2] Temporary submersible pump used to take samples – pumping test pump removed.

4 Background Water Level Observations

4.1 Overview

Water level observations from the start of the monitoring period (27 November 2013) to the end of recovery monitoring on 16 December 2013 are provided below to increase the understanding of external influences such as tides on the hydrogeology of the area. Figure 4.1 shows groundwater levels for all chalk observation boreholes and Figure 4.2 shows groundwater levels for drift observation boreholes. Tidal levels recorded at Albert Dock are also shown for reference.

4.2 Chalk

Figure 4.1 shows that throughout the monitoring period, chalk groundwater levels vary by the following amounts for each tidal cycle:

BH18A: 1.8 to 2.5 m

BH24: 1.5 – 3.1 m

BH29: 1.5 – 4.4 m

The tidal influence on chalk groundwater levels is greater than any other observed influence, although underlying variations due to recharge and other factors also occur.

The 12 hour moving average for water levels recorded at BH18A (see Figure 4.1) shows the background water level variations with the tidal influence removed. The 12 hour average was relatively stable prior to the tidal surge of 5 December 2013, and following this average groundwater levels fell steadily. This downwards trend continued into the pumping test, although average groundwater levels appeared to stabilise between 10 and 12 December. The average groundwater levels again rose on 13 December following the end of the pumping test.

Spring and neap tides affect the variation in levels seen within each tidal cycle. Neap tides occurred on 25 November and 9 December 2013, and the variation in groundwater levels immediately following these neap tides was much smaller than following the spring tide of 3 December 2013. Prior to the tidal surge on 5 December, fluctuations in groundwater levels were relatively stable, showing a steadily increasing range. Groundwater levels reached a peak in all observation boreholes on 5 December 2013, as the tidal surge occurred.

Following the tidal surge the variation in groundwater levels reduced gradually, with peak groundwater levels in each tidal cycle dropping more significantly than minimum groundwater levels. A step drop in peak groundwater levels can be seen coinciding with the start of the pumping test. Following the end of the pumping test, peak groundwater levels are seen to recover to levels observed before the start of the test. Minimum water levels appear to stabilise during the pumping test period, although a step increase can be seen following recovery.

The tidal efficiency, lag times and amplitude (half the range in water levels in each tidal cycle) have been measured from the hydrographs for BH18A, BH24 and BH29 using the Albert Dock water level (tidal stage) information. The tidal efficiency of a confined aquifer is defined as the ratio of change in hydraulic head to the change in tide stage, and the lag time is calculated as the difference in time between the peak tidal stage and peak groundwater level. The amplitude is half the range of water levels in a tidal cycle.

The average of measured tidal efficiencies and lag times for the monitoring period 27 November to 5 December 2013 are summarised in Table 8.

Table 8: Measured Tidal Influence Parameters for Chalk Observation Boreholes

Borehole	Tidal Efficiency (%)	Lag Time (hr:min:sec)	Amplitude (m)
BH18A	42	00:53:06	1.22
BH29	43	00:48:18	1.24
BH24	43 *	01:04:12 *	1.06 *

* Derived from limited data: 27 – 28 November and 4 – 6 December 2013.

4.3 Drift

Figure 4.2 shows that water levels in the cohesive alluvium at BH27 and the glacial till at BH15 generally follow a similar trend. Groundwater levels within the drift are thought to be largely influenced by recharge, with peaks occurring on 29 November, 5 December, 13 December, 14 December and 16 December 2013.

Some very slight semi-diurnal variation in levels can be seen in water levels at BH15 between 2 and 8 December 2013, suggesting tidal influence, although there is an obvious lag between peak levels in the tidal data and peak drift groundwater levels. Tidal influence parameters are summarised in Table 9.

Table 9: Measured Tidal Influence Parameters for Drift Observation Boreholes

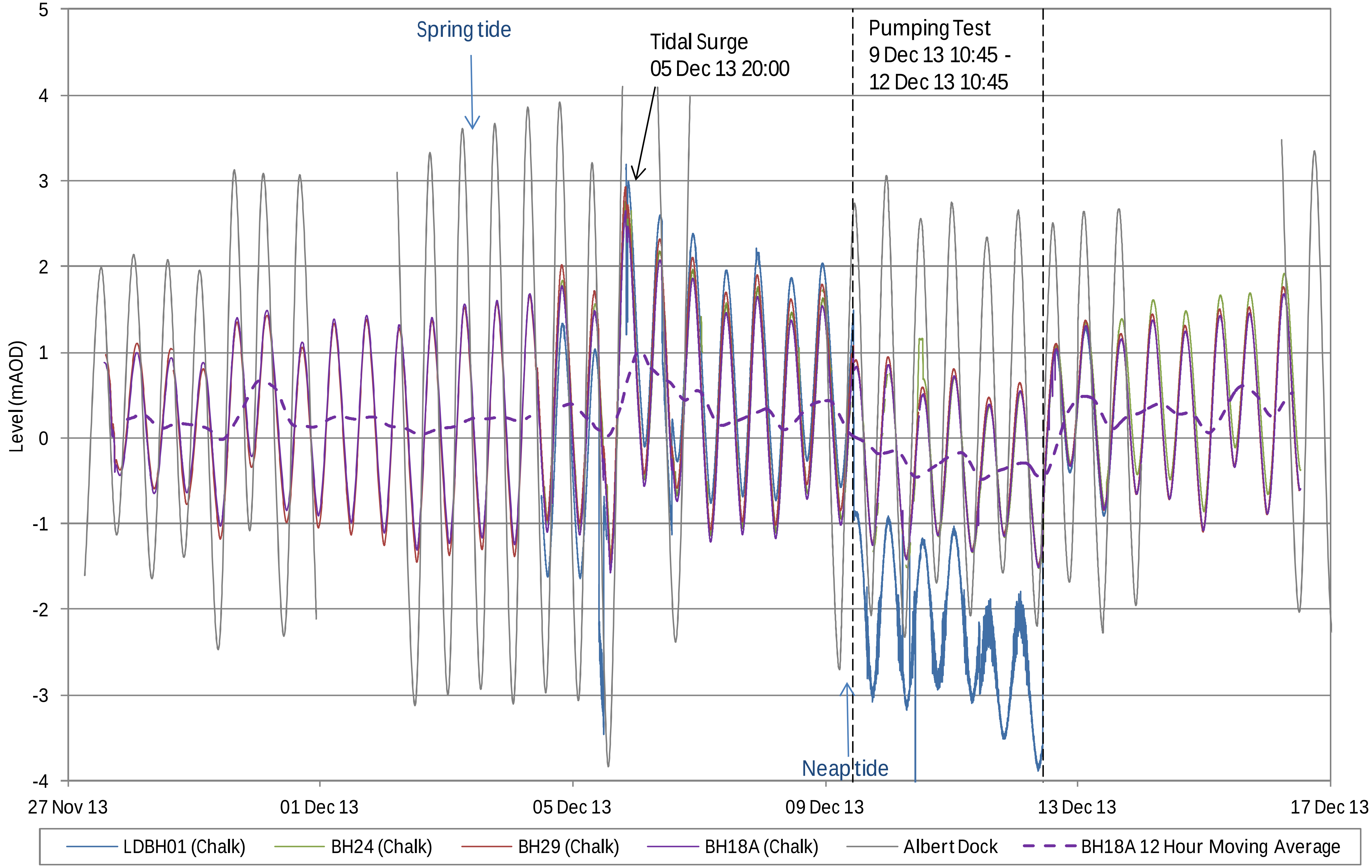
Borehole	Tidal Efficiency (%)	Lag Time (hr:min:sec)	Amplitude (m)
BH15	0.3	02:14:08	0.014

* Derived from limited data: 2 – 4 December 2013.

Following the tidal surge, groundwater levels in BH27 (cohesive alluvium) increase very steadily and continue to increase throughout the pumping test period. Groundwater levels in BH15 (glacial till) show a decreasing trend following a slight peak on 7 December and throughout the pumping test however, before levelling off towards the end of the pumping test.

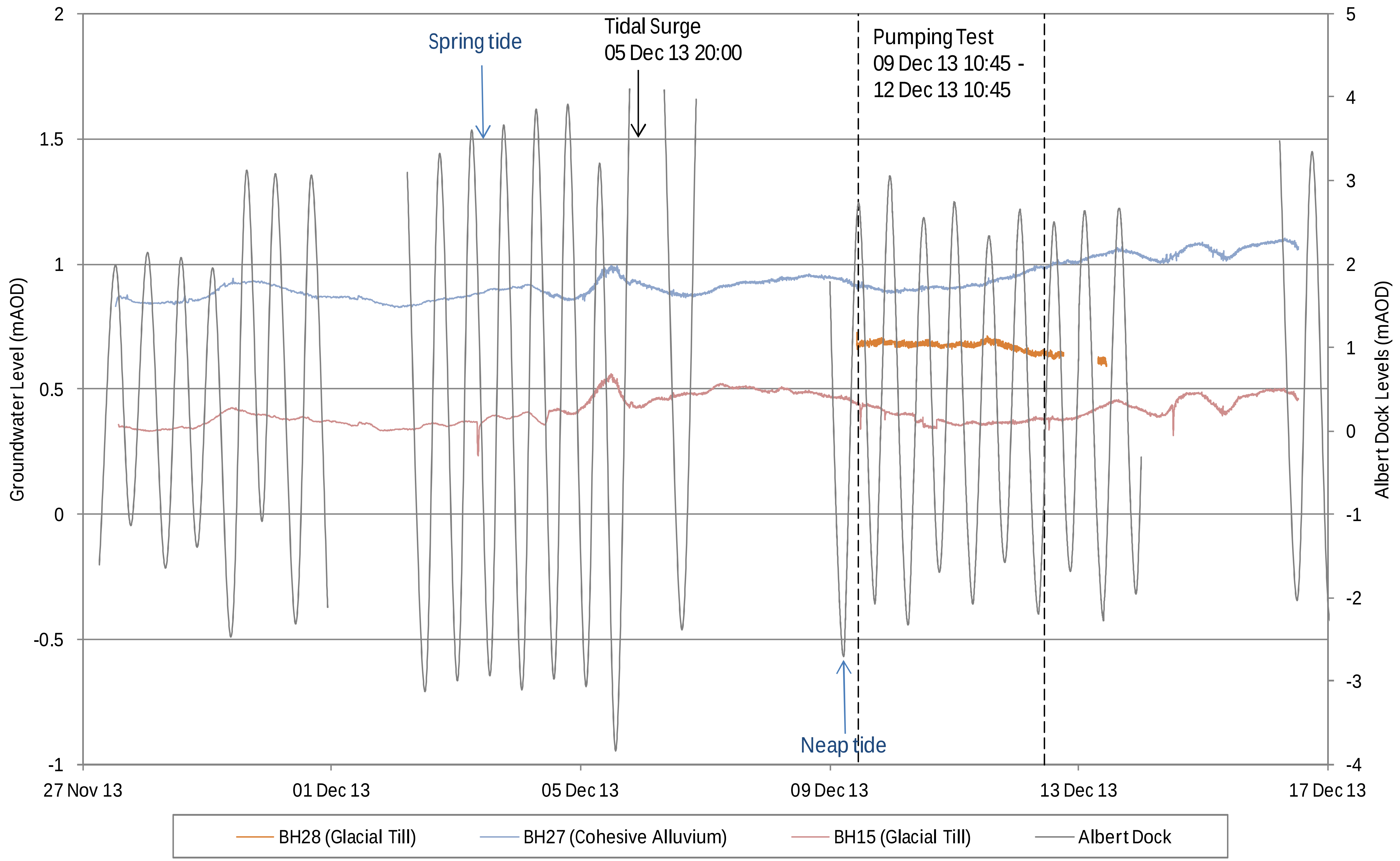
Water level data for BH28 (glacial till) is only available for the pumping test period and shows very little change, although levels do appear to fall towards the end of the pumping test (11 December 2013).

Figure 4.1 – Chalk Groundwater Levels, 27 November 2013 – 17 December 2013



NB. No Albert Dock data 01 Dec 13, 07 Dec 13 - 08 Dec 13, 14 Dec 13 - 15 Dec 13

Figure 4.2 – Drift Groundwater Levels, 27 November 2013 – 17 December 2013



NB. No Albert Dock data 01 Dec 13, 07 Dec 13 - 08 Dec 13, 14 Dec 13 - 15 Dec 13

5 Pumping Test Results

The pumping test was carried out between 9 and 12 December 2013, and recovery of water levels was monitored over 12 and 13 December 2013. Figures 5.1 and 5.2 provide a graphical representation of the manual water level and discharge data collected during the LDBH01 pumping test for chalk observation boreholes and drift observation boreholes respectively. The water levels have been corrected to mAOD based on the information in Table 4.

Manual dip and flow data is provided in Annex 2 and in Figures 4 and 5 for chalk observation boreholes and drift observation boreholes respectively. Logger data is supplied electronically separately from this document.

5.1 Abstraction Rates

The test was started at 10:45:00 and the desired pumping rate was achieved 5 minutes into the test. An average pumping rate of 18.2 l/s was maintained for the first two days of the test, although there were fluctuations of around 0.05 l/s and up to 1 l/s. These variations in abstraction rate were largely due to the tidal influence on pumping water levels and hence head against the control valve. Abstraction rates were controlled manually via the control valve during the pumping test resulting in a slow response to any changes in discharge. The average pumping rate over the first 300 minutes of the test was slightly lower at 18l/s.

The pumping rate was increased to 23 l/s on 11 December 2013 10:45, after 48 hours of pumping at the lower rate. This was to see if a response to pumping greater than the tidal influence could be created. An average pumping rate of 23.0 l/s was maintained with fluctuations of around 0.5 l/s and up to 1 l/s. Again, variations in abstraction rate were due to the tidal influence on pumping water levels.

5.2 Water Levels

Chalk

Figure 5.1 shows that a clear, albeit small response to abstraction can be seen in all chalk observation boreholes within the first 20 minutes of the test, with a maximum drawdown of 0.265 m observed at BH24. After 20 minutes tidal variations again become the primary influence on groundwater levels. After 60 minutes, chalk groundwater levels reach the tidal peak and following this they begin to fall in response to the falling tide.

Table 8 summarises the maximum observed drawdown values at each of the Chalk observation boreholes. BH18A and BH29 are at similar distances away from the pumping borehole, yet the drawdown is greater at BH18A, to the north of the pumping borehole and further away from the Humber Estuary.

Table 10: Maximum drawdown observed during first hour of pumping

Borehole	Distance from pumping borehole	Maximum Observed Drawdown (m)
LDBH01	-	1.987 (t = 12.5min)
BH24	17.8m NE	0.265 (t = 20min)
BH29	48.0m S	0.139 (t = 19min)
BH18A	54.1m NW	0.156 (t = 18min)

Figure 5.3 shows Chalk groundwater levels at the start of the test and in comparison to tidal levels at Albert Dock. This shows that the high tide peaked approximately 20 minutes into the test and after approximately 195 minutes the hydraulic gradient between the Humber Estuary and the underlying Chalk changes so that Chalk groundwater levels in the observation boreholes are higher beyond this point in the tidal cycle.

No further pumping influence can be identified during the pumping test, despite the increase in pumping rate on 11 December 2013 10:45 to 23 l/s.

It is not possible to comment on recovery, even within the first few minutes of the pumps being switched off, because groundwater levels were rising at the time due to the rising tide, as can be seen in Figure 4.

Drift

No tidal influence has been observed within the drift monitoring boreholes, as is apparent in Figure 5.2.

5.3 Water Quality

Results of the water quality samples, including in-situ analysis, are provided in Annex 3. The results are summarised in Table 10, which shows the range of values and mean obtained for major ions, nitrogen species, metals and organics. This highlights exceedances against Environmental Quality Standards (EQS). Note in particular the saline intrusion evidenced by high sodium and chloride concentrations.

Table 11: Summary of Water Quality

Substance	EQS [1]	Range of Values and Average Given in Brackets					
		Humber Estuary [2]	LDBH01	SW2	SW3	BH24	BH29
Number of samples		96	7	7	7	6	4
pH	6 - 9	6 – 8.3 (7.7)	7 – 7.2 (7.1)	7.7 – 7.8 (7.7)	7.7 – 8 (7.8)	7 – 7.6 (7.2)	7 – 7.6 (7.2)
Electrical Conductivity (mS/cm)	-		16,000 – 26,000 (23,571)	2,500 – 27,000 (19,071)	2,500 – 27,000 (18,786)	7,200 – 26,000 (19,550)	7,100 – 27,000 (21,275)
MAJOR IONS							
Calcium (mg/l)	250 (DWS)	103 – **308** (214)	170 – **270** (227)	140 – 230 (170)	140 – 220 (169)	130 – **260** (200)	100 – 240 (192.5)
Magnesium (mg/l)	-	50 – 600 (357)	94 – 610 (503)	360 – 590 (459)	350 – 610 (463)	99 – 590 (410)	52 – 570 (377)
Sodium (mg/l)	-	-	2,200 – 4,400 (3,457)	2,200 – 4,600 (3,286)	2,100 – 4,700 (3,143)	860 – 4,300 (3,060)	330 – 4,500 (2433)
Potassium (mg/l)	-	-	200 – 820 (313)	140 – 230 (183)	140 – 240 (184)	62 – 250 (1,732)	78 – 210 (143)
Alkalinity as $CaCO_3$ (mg/l)	-	-	160 – 350 (213)	110 – 160 (133)	120 – 160 (130)	180	220
Sulphate (mg/l)	400 (EQS, 2004)	-	180 – **1300 (1166)**	**850 – 1300 (1070)**	**720 – 1300 (1049)**	130 – **1,300 (843)**	320 – **1100 (878)**
Chloride (mg/l)	250 (EQS, 2004)	134 – **15,100 (6,937)**	**5500 – 10,000 (9,050)**	**6,000 – 9800 (7,743)**	**5300 – 10,000 (7,643)**	**1,700 – 11,000 (6,900)**	**3,200 – 8,800 (7,025)**
NITROGEN SPECIES							
Nitrate (mg/l)	50 (DWS)	0.2 – 6.6 (4.0)	<0.5 – 0.9	1.7 – 5.8 (4.2)	1.8 – 5.8 (4.4)	<0.5 – 5	<0.5 – 10

Substance	EQS 1	Range of Values and Average Given in Brackets					
		Humber Estuary [2]	LDBH01	SW2	SW3	BH24	BH29
Nitrite (mg/l)	0.5 (DWS)	0.004 – 0.087 (0.023)	<0.01 - <0.1	<0.1 – 0.2 (0.17)	<0.1 – 0.2 (0.17)	<0.1 – **5.2 (2.7)**	<0.1 – 0.8
Ammoniacal Nitrogen as N (mg/l N)	0.2	0.012 – **0.779** (0.095)	**0.32 – 23 (4.89)**	**0.31 – 0.69 (0.55)**	**0.31 – 1.6 (0.65)**	**0.96 – 14 (4.03)**	**1.5 – 18 (6.35)**
METALS							
Arsenic (mg/l)	50	5 – 48 (17)	**96 – 320 (193)**	32 – **270 (159)**	34 – **260 (159)**	29 – **170 (76)**	17 – **170 (75)**
Boron (mg/l)	2000 (EQS, 2004)	850 – **3,040** (1,743)	1,900 – **2100 (2000)**	1300 – 1900 (1,600)	1,200 – 1,900 (1,571)	330 – **2,000** (1,368)	63 – **2,000** (1,178)
Cadmium (mg/l)	0.15	-	0.03 – **0.89 (0.20)**	0.08 – **0.15** (0.12)	0.1 – **0.15** (0.12)	0.02 – 0.11 (0.05)	<0.02 – **0.15** (0.07)
Chromium (total) (mg/l)	-	2 – 60 (30)	1 – 23 (14)	2 – 18 (13)	2 – 17 (12)	2 – 19 (12)	3 – 51 (19)
Chromium (III) (mg/l)	4.7	-	<3 – **23 (16)**	<3 – **18 (14.5)**	<3 – **17 (13.5)**	<3 – **16 (12.5)**	3 – **51 (19)**
Copper (mg/l)	10	**17 – 127 (47)**	**49 – 200 (123)**	**27 – 170 (85)**	**29 – 250 (104)**	**12 – 610 (288)**	**35 – 250 (112)**
Lead (mg/l)	7.2	**38 – 248 (112)**	<0.3	<0.3 – 4.5 (2.0)	<0.3 – 2.5 (1.2)	<0.3 – **17** (6.2)	<0.3 – **7.9** (4.3)
Iron (dissolved) (mg/l)	1,000	<100	<10 – 230 (154)	76 – 220 (148)	84 – 200 (142)	150 – 310 (233)	95 – 210 (152)
Iron (total) (mg/l)	1,000	**5,310 – 31,600 (16,886)**	**4,200 – 9,700 (6,617)**	330 – 370 (350)	400 – 500 (450)	**4,400 – 24,000 (11,250)**	**5,500 – 11,000 (7,867)**
Manganese (dissolved) (mg/l)	50 (DWS)	-	**380 – 420 (342)**	49 – **54 (51.5)**	27 – **51** (39)	**440 – 470 (460)**	**500 – 510 (507)**
Manganese (total) (mg/l)	50 (DWS)	-	**110 – 440 (370)**	**400 – 500 (450)**	**66 – 72 (69)**	**460 – 510 (485)**	**510 – 580 (550)**

Substance	EQS 1	Range of Values and Average Given in Brackets					
		Humber Estuary [2]	LDBH01	SW2	SW3	BH24	BH29
Mercury (mg/l)	0.05	0.04 – **0.51 (0.17)**	<0.05 – **0.1 (0.09)**	<0.05	<0.05	<0.05 – **0.28 (0.18)**	<0.05 – **0.11 (0.11)**
Nickel (mg/l)	20	9 – **123 (37)**	**31 – 47 (32)**	6 – 18 (12)	6 – 16 (12)	14 – **57 (31)**	14 – **39 (30)**
Selenium (mg/l)	10 (DWS)	-	<0.5 – **210 (165)**	<0.5 – **170 (129)**	<0.5 – **160 (122)**	<0.5 **– 57 (56)**	<0.5 – **94 (51.4)**
Zinc (mg/l)	75	**75 – 720 (240)**	2 – **78** (42)	26 – 47 (36)	21 – 49 (35)	4 – 32 (14)	3 – 13 (8)
Cyanide (total) (mg/l)	1	-	<10 – **71 (42)**	<10 – **26 (18)**	<10	**17 – 26 (22)**	<10 **– 48 (32)**
Total TPH (mg/l)	10 (DWS)	-	<10	<10 – 60	<10 - 60	<10 – **220 (113)**	<10 – **30**
Total PAHs (mg/l)	-	-	<0.01 – 0.27 (0.15)	<0.01 – 0.36 (0.17)	<0.05 – 0.27 (0.13)	<0.01 – 0.62 (0.22)	0.02 – 0.64 (0.21)

Notes:

Ranges exclude spurious results (as highlighted in Annex ##).

Values in Bold exceed the EQS.

[1] EQS taken from the Water Framework Direction (2010). Where a value is not available, the 2004 EQS or Drinking Water Standard are provided.

[2] Environment Agency water quality data for the River Humber at Albert Dock, 29 Jan 2003 – 17 Oct 2012.

Concentrations of most substances generally remained steady with the development and continued pumping of LDBH01, although concentrations of selenium increased and cyanide decreased during the pumping test. Similarly, concentrations of most substances remained steady throughout the sampling period at SW2 and SW3, in Railway Dock.

As discussed in the Groundwater Report (Document Number 1168-10-223-RE-001-PD1), the main controls over the chalk groundwater quality in the project area are redox reactions occurring due to its confined nature and saline intrusion.

Low concentrations of dissolved oxygen, denitrification (i.e. low nitrate and nitrite concentrations and increased ammoniacal nitrogen) and increased concentrations of iron and manganese are indicative of a reducing environment. Although dissolved oxygen values obtained from in-situ testing range are not necessarily as low as would be expected for a reducing environment (ranging between 1.6 and 5.3 mg/l), nitrate and nitrite concentrations in LDBH01 are very low, with only one sample exceeding the detection limit for nitrate on 10 December 2012. In addition, ammoniacal nitrogen concentrations in samples from LDBH01 are up to two orders of magnitude greater than concentrations in surface water samples. Whilst dissolved iron in groundwater is not particularly high in comparison to surface water samples, a concentration of 200 mg/l is considered high in groundwater. In comparison, the average iron concentration in the Yorkshire unconfined chalk is 19 mg/l (Smedley et. al., 2004).

Elevated concentrations of arsenic and boron have also been reported across the confined Yorkshire chalk aquifer (Smedley et. al., 2004).

Concentrations of chloride, magnesium, sodium, sulphate and electrical conductivity are high in all groundwater samples and comparable to surface water samples, and are therefore indicative of saline intrusion in the chalk aquifer.

Concentrations of chromium and copper were high in all samples taken, with concentrations of nickel high in all groundwater samples, and concentrations of selenium and zinc high in samples taken from LDBH01 and the surface water samples. Elevated concentrations of these metals cannot be related to either the reducing environment or saline intrusion however, and may reflect local contamination.

A number of PAHs were detected on 27 November 2013 during development of LDBH01. The only detection of PAHs after more prolonged pumping however was of naphthalene on 10 December 2013, and this was below the EQS. Naphthalene was also detected in a number of samples taken from SW2 and SW3 (5 September, 4 & 17 December 2013). Again these samples were below the EQS.

Figure 5.1 – Manual Water Level & Discharge Rate During Pumping Test - Chalk

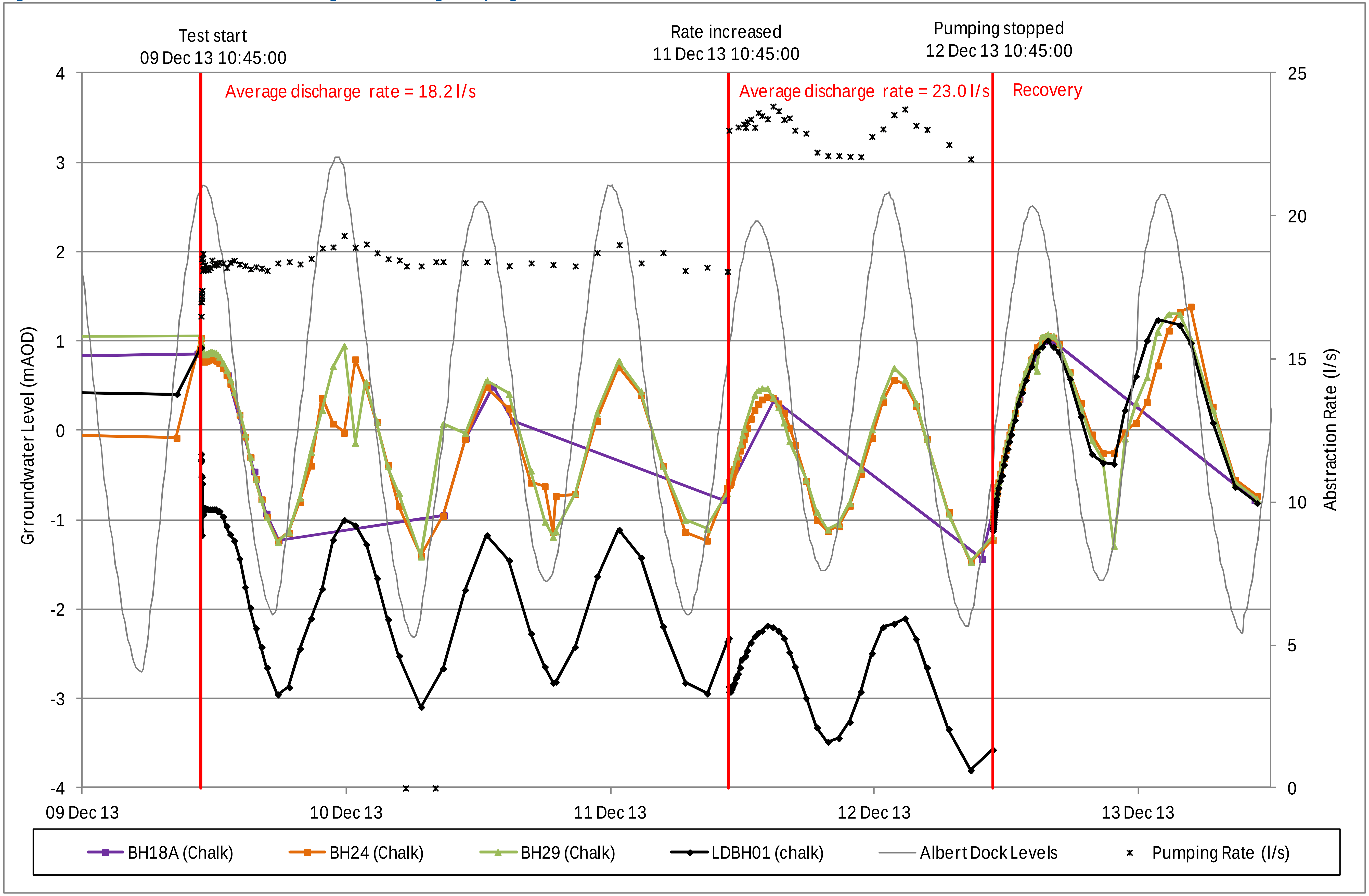


Figure 5.2 – Manual Water Level & Discharge Rate During Pumping Test - Drift

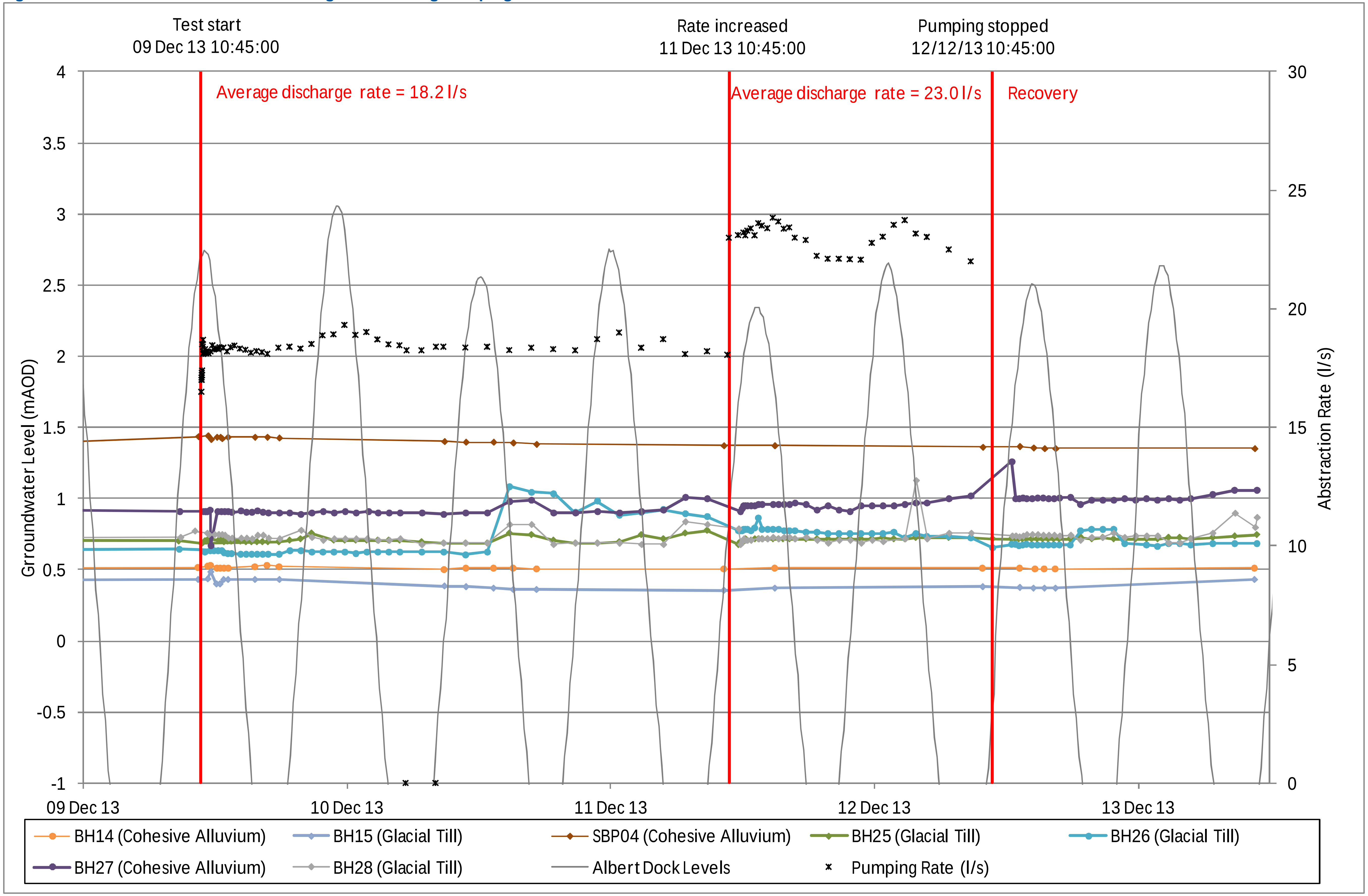
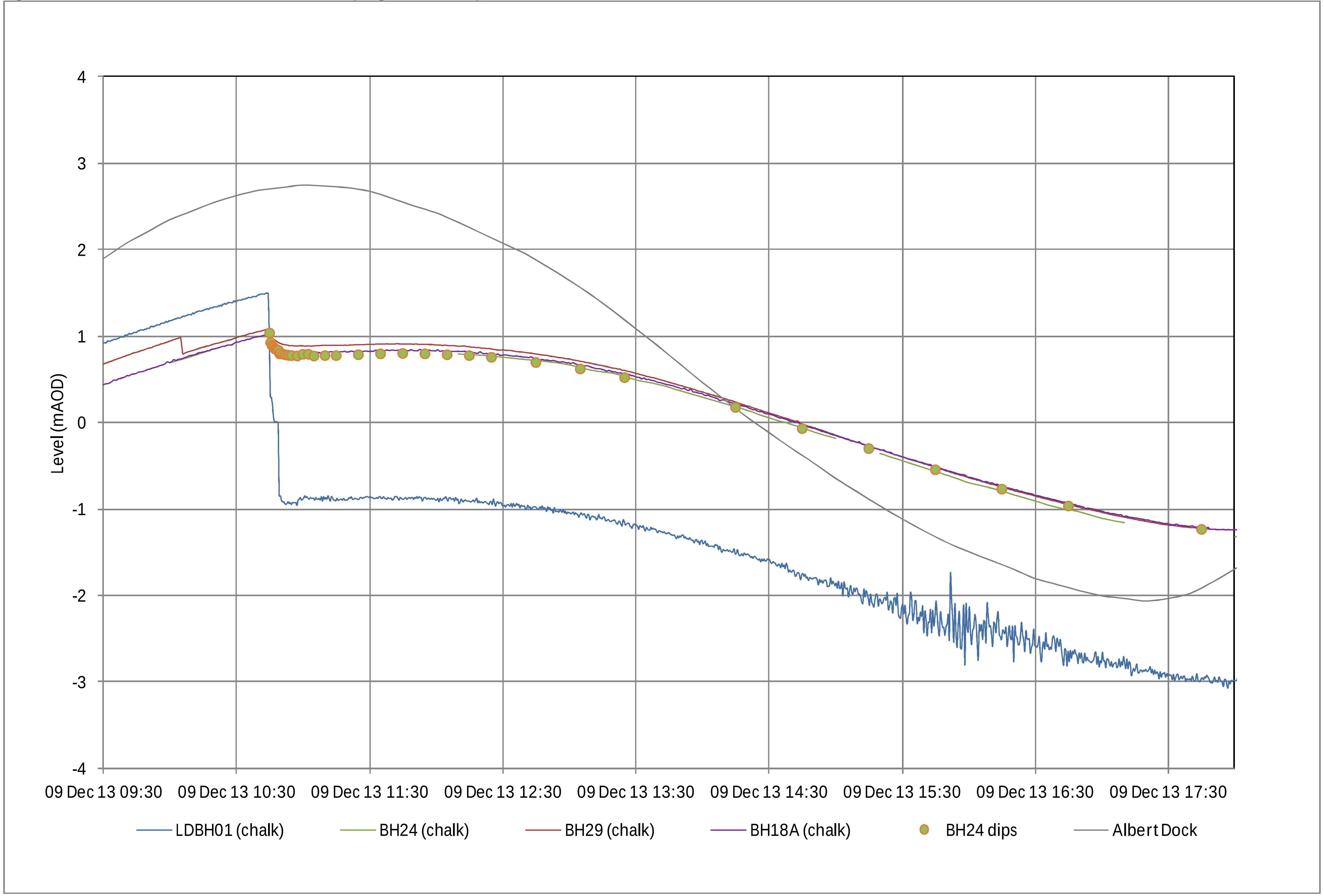


Figure 5.3 – Chalk Groundwater Levels at Pumping Test Startup



6 Analysis of the Results

6.1 Pumping Test Analysis Method

Overview

Drawdown data has been analysed using the Theis (1935) curve fitting and Cooper and Jacob (1946) straight line methods for unsteady-state flow in confined aquifers. These methods are considered appropriate to the LDBH01 pumping test, where chalk is overlain by around 29m of drift deposits (including the glaciolacustrine clays) and the rest water level in the chalk is above the top of the aquifer unit. The assumptions behind the two methods are described in Kruseman and de Ridder (1994).

Data Processing

Water levels in the abstraction and observation boreholes failed to approach a quasi-steady state condition during the pumping test due to the tidal influence on chalk groundwater levels.

Water level data from the chalk observation boreholes has therefore been corrected for tidal influence by super-position of non-pumping water levels from the preceding tidal oscillation. Figures 6.1 to 6.3 show curve matching of the preceding tidal oscillation and resulting drawdowns for BH18A, BH24 and BH29 respectively. Amplitude and duration of the adjacent tidal oscillations were considered sufficiently similar that this super-position approach is considered to provide representative drawdown data for the first 300 minutes of the test. 60 minutes into the pumping test, however, groundwater levels reached the peak in tidal fluctuations and beyond this there is increased uncertainty in the accuracy of the corrected data.

Input Parameters

The data from BH18A, BH24 and BH29 have been analysed using the commonly-used AquiferWin32 software, in which the time drawdown data are used to calculate transmissivity (T) and the aquifer storativity (S). Data up to 300 minutes has been used in curve and line fitting. Table 10 presents input parameters used in AquiferWin32.

Table 12: Input parameters

Parameter	Pumping Well	Monitoring Well		
		BH18A	BH24	BH29
Pumping rate	18 l/s *	N/A	N/A	N/A
Radial distance	N/A	54.1 m	17.8 m	48.0 m
Casing inner diameter	0.356 m	0.05 m	0.05 m	0.05 m
Diameter of drilled hole	0.311 m	0.14 m	0.14 m	0.14 m
Screen length	17.3 m	13 m	12.7 m	14 m
Screen top depth	0	0	0	0

Note: * Although the pumping rate varied over the course of the three day test, only early data is being analysed when the pumping rate was held constant at an average rate of 18l/s. Therefore the test was analysed assuming a constant pumping rate.

Transmissivity is the product of the average hydraulic conductivity (K) and the saturated thickness of the aquifer (d). From this, the average hydraulic conductivity has been calculated, using an effective aquifer thickness of 20m. A summary of the results is given in Section 6.3.

Recovery

It is not possible to quantify residual drawdown during the recovery phase as tidal variations during the pumping test do not allow the original rest water level to be used.

6.2 Pumping Test Analysis

Theis Analysis

The Theis analysis for boreholes BH18A, BH24 and BH29 are shown in Figures 6.4 to 6.6 respectively. A fairly good fit is achieved to the early data, up to around 60 minutes. Beyond this point data falls below the Theis curve at this point, as would be expected when leakage occurs or a recharge boundary is encountered.

Very early time data would usually be ignored to allow for well storage effects and time taken to establish a constant pumping rate. Due to the limited meaningful data available for the pumping test, early data has been relied upon for curve fitting. In the analysis for BH24 however, data up to 4 minutes does not fit the Theis curve particularly well, possibly suggesting some slight well storage effects.

Cooper and Jacob Analysis

The Cooper and Jacob analysis for boreholes BH18A, BH24 and BH29 are shown in Figures 6.7 to 6.9 respectively. As with the Theis analysis, a fairly good fit is achieved to the early data up to around 50 minutes (and between 50 and 60 minutes for BH24). Beyond this, data deviates from the straight line as would be expected when leakage occurs or a recharge boundary is encountered.

As with the Theis analysis, very early data (up to 4 minutes into the test) from BH24 does not fit the straight line particularly well, possibly suggesting well storage effects (see Figure 14).

6.3 Summary of Estimated Aquifer Properties

Table 13 shows the estimated aquifer properties derived from the above analyses. Hydraulic conductivity (K) has been calculated from the transmissivity (T) and saturated aquifer thickness (b) using the following equation: -

$$K = T/b$$

As the derived hydraulic conductivity is based on the pumping test analysis, it represents a bulk value. In reality, it is likely to be much more locally variable.

Table 13: Summary of Estimated Aquifer Properties

Borehole	Test	Results		
		T (m^2/d)	S	K (m/s)
BH18A	Theis	1394	0.00032	8.07×10^{-4}
	Cooper and Jacob	1379	0.00033	7.98×10^{-4}
BH24	Theis	1587	0.00126	9.18×10^{-4}
	Cooper and Jacob	1631	0.00126	9.44×10^{-4}
BH29	Theis	1504	0.00043	8.70×10^{-4}
	Cooper and Jacob	1515	0.00044	8.77×10^{-4}
Average		**1502**	**0.0007**	$\mathbf{8.69 \times 10^{-4}}$

Note: ‘b’ estimated to be 20m (i.e. the effective thickness of the chalk).

6.4 Radius of Influence

Jacob's approximation of the Theis equation and aquifer parameters derived from analyses for BH18A and BH29 has been used to calculate the pumping boreholes' radius of influence during the pumping test. After 50 minutes of pumping, the zone of influence extends to approximately 550m from the pumping borehole, which is approximately the distance from the pumping borehole to the Humber Estuary.

6.5 Tidal Efficiency and Lag Time

Tidal efficiency can be calculated from aquifer properties using the following equation (from Fetter, 2001) :

$$\text{Tidal efficiency} = \exp(-x\,\sqrt{((\pi\,S)/(t_0\,T))})$$

Where x is the distance from the coast, t_0 is the tidal period (i.e. time between minimum and maximum levels), S is the storage coefficient and T is transmissivity. Groundwater amplitude can be calculated using the tidal efficiency and tidal amplitude.

Lag times can also be calculated using the following equation:

$$\text{Lag time} = x\sqrt{(t_0/4\pi T)}$$

Table 14 summarises tidal efficiencies and lag times calculated for the observation boreholes using transmissivity and storativity derived from the pumping tests.

Table 14: Calculated Tidal Influence Parameters for Chalk Observation Boreholes

Borehole	Distance from Humber Estuary (m)	Tidal Efficiency	Lag Time	Amplitude (m)
BH18A	553	39%	55 minutes	1.08
BH29	457	42%	50 minutes	1.16
BH24	520	20%	94 minutes	0.55
LDBH01	504	40% *	53 minutes *	1.12

Notes: * calculated using average T and S values derived from BH18A and BH29.

The values derived for BH18A and BH29 agree well with measured values of tidal influence parameters, as presented in Section 4.2, confirming that T and S values obtained from the pumping test analysis are appropriate.

The S value derived for BH24 is an order of magnitude greater than that derived for other observation boreholes and this is reflected in the calculated tidal influence parameters. The greater deviation between measured and calculated tidal influence parameters for this borehole suggests that the S value is inaccurate.

6.6 Discussion of Results

Mean values of hydraulic conductivity (K) of 8.69×10^{-4} m/s and storativity (S) of 7×10^{-4} were derived from pumping test data analysis. The observation borehole test data gave very similar K values, which fall at the higher end of the range of values obtained by falling head tests within the GI and are two orders of magnitude greater than values obtained from packer tests.

The K values are much higher than those obtained from the Hull Truck Theatre pumping test in 2005 (Environment Agency, 2005), where a transmssivity value of $45m^2/d$ was obtained. Despite its close proximity to LDBH01 (850m to the north), chalk transmissivity is to a large extent dependant on fissuring within the chalk, and can therefore be highly variable over short distances. Evidence of fissuring was indeed noted at a depth of 44 – 45 mbGL during construction of LDBH01. It has also been reported that zones of enhanced transmissivities occur in dry valleys, where chalk 'bearings' or broken chalk occur at rock-head due to periglacial processes. The top 3 – 4m of the chalk was indeed reported to be particularly weak during drilling of LDBH01, possibly suggesting that LDBH01 is located within such a zone of enhanced transmissivity. Observations of the spatial extents of drift deposits also suggests that the site is possibly located within the old channel of the River Hull.

Storativity values ranged from 3.2×10^{-4} to 1.3×10^{-3}. This range is at the upper end of values expected for a confined aquifer. The higher values were obtained from the analysis of BH24 data, but are not considered accurate as these were not verified by tidal influence parameter calculations.

Maximum observed (i.e. not corrected for tidal influences) drawdown occurred 20 minutes into the test. This coincides with high tide, as observed at Albert Dock. Beyond 20 minutes, observed drawdown falls as tidal influence becomes the overriding control over groundwater levels. Meaningful drawdown data was obtained, however, by superposition of non-pumping water levels from the preceding tidal oscillation. This removed the tidal influence from the drawdown data.

The corrected data indicates that drawdown stabilised in observation boreholes at around 50 minutes. As discussed in Sections 5.2 and 5.4 this is most likely associated with a recharge boundary, namely the Humber Estuary. Although the corrected drawdown data was assumed to be reasonable, there is some uncertainty of its accuracy beyond 60 minutes. However, radius of influence calculations confirm that the cone of depression reached the Humber Estuary 50 minutes into the pumping test.

Although the hydraulic gradient between water levels at Albert Dock and groundwater levels reverses 195 minutes into the test, there is no observable impact in the drawdown data.

Analysis of water quality results also suggests an estuarine influence within the chalk, with a strong saline signature observed in water from both the pumping and observation boreholes (i.e. elevated concentrations of chloride, magnesium, sodium and sulphate).

No drawdown was observed in any of the drift observation boreholes during the test, confirming that there is no induced leakage from the overlying drift into the chalk during the test. This suggests that hydraulic continuity between the drift and the chalk is limited, and that the glaciolacustrine deposits act as an effective aquitard. Although tidal influences were observed within the drift at observation borehole BH15 (see Section 4.3), it is unlikely that this tidal signal has been projected through the chalk. It is more likely due to a direct connection between the glacial till and the Humber Estuary. This does not preclude the possibility of localised areas of coarse granular alluvium in hydraulic continuity with the chalk aquifer.

Figure 6.1 – Drawdown Corrections – BH 18A

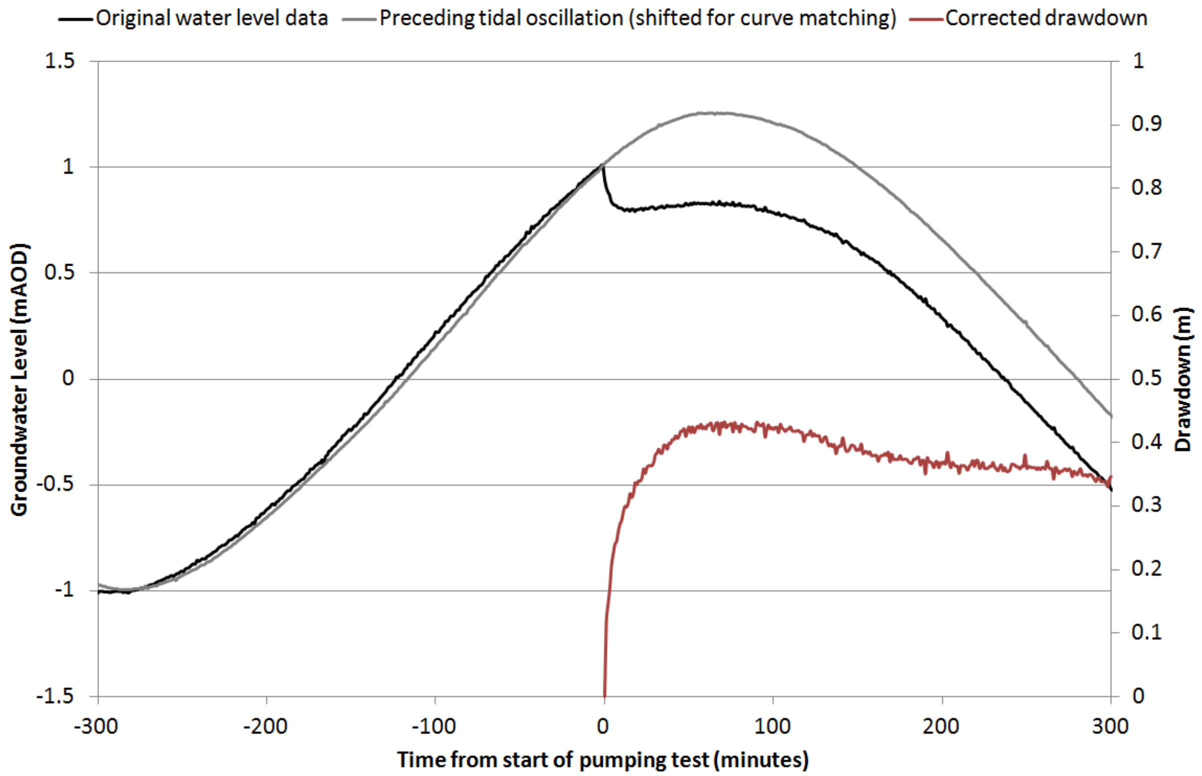


Figure 6.2 –Drawdown Corrections – BH24

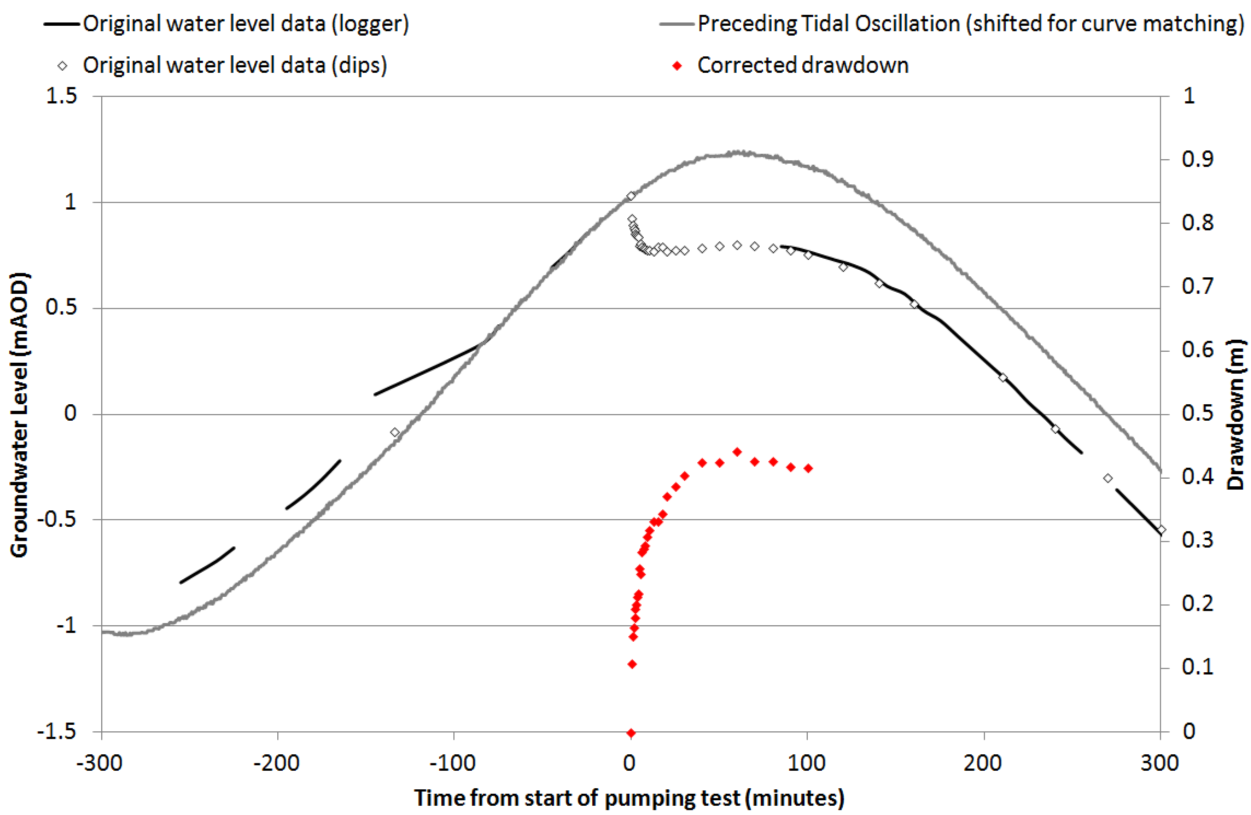


Figure 6.3 –Drawdown Corrections – BH29

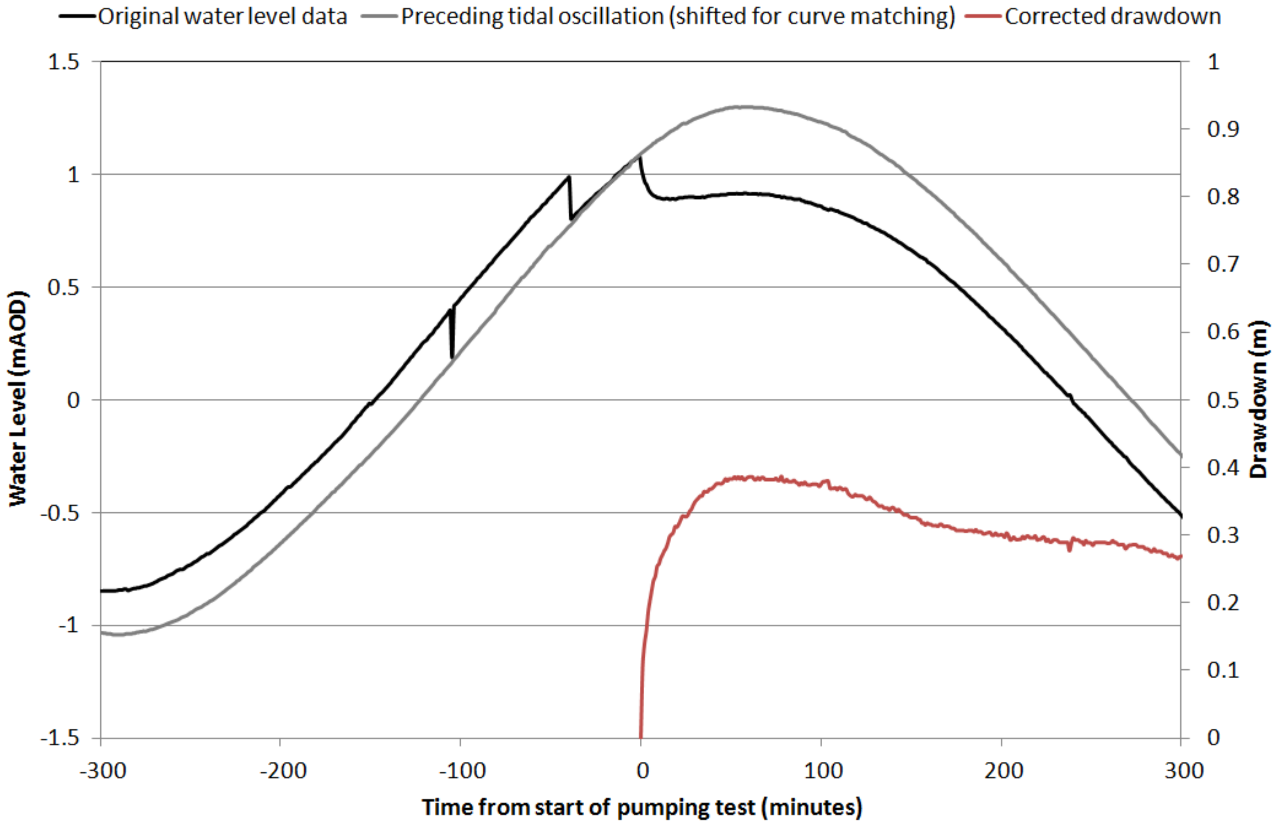
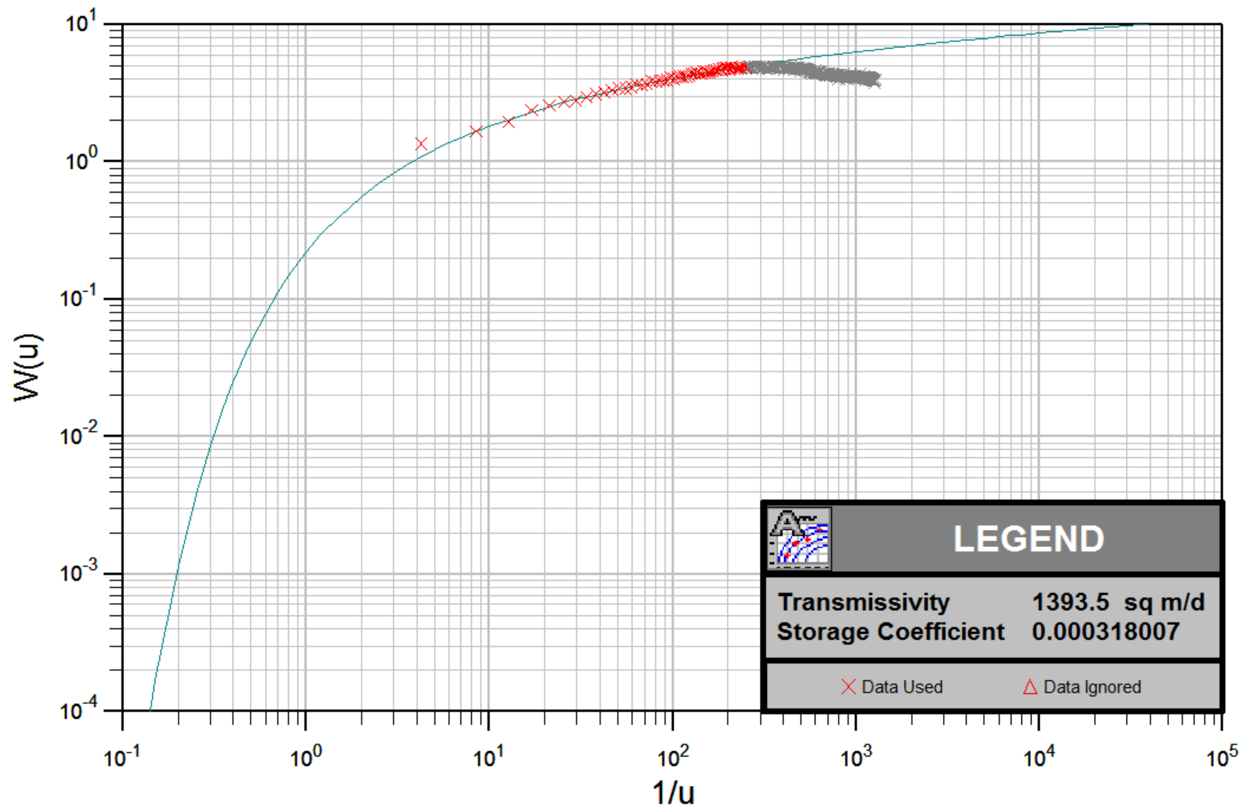


Figure 6.4 –Theis Analysis – BH18A



Note: Low confidence data (t >60 minutes) shown in grey.

Figure 6.5 – Theis Analysis – BH24

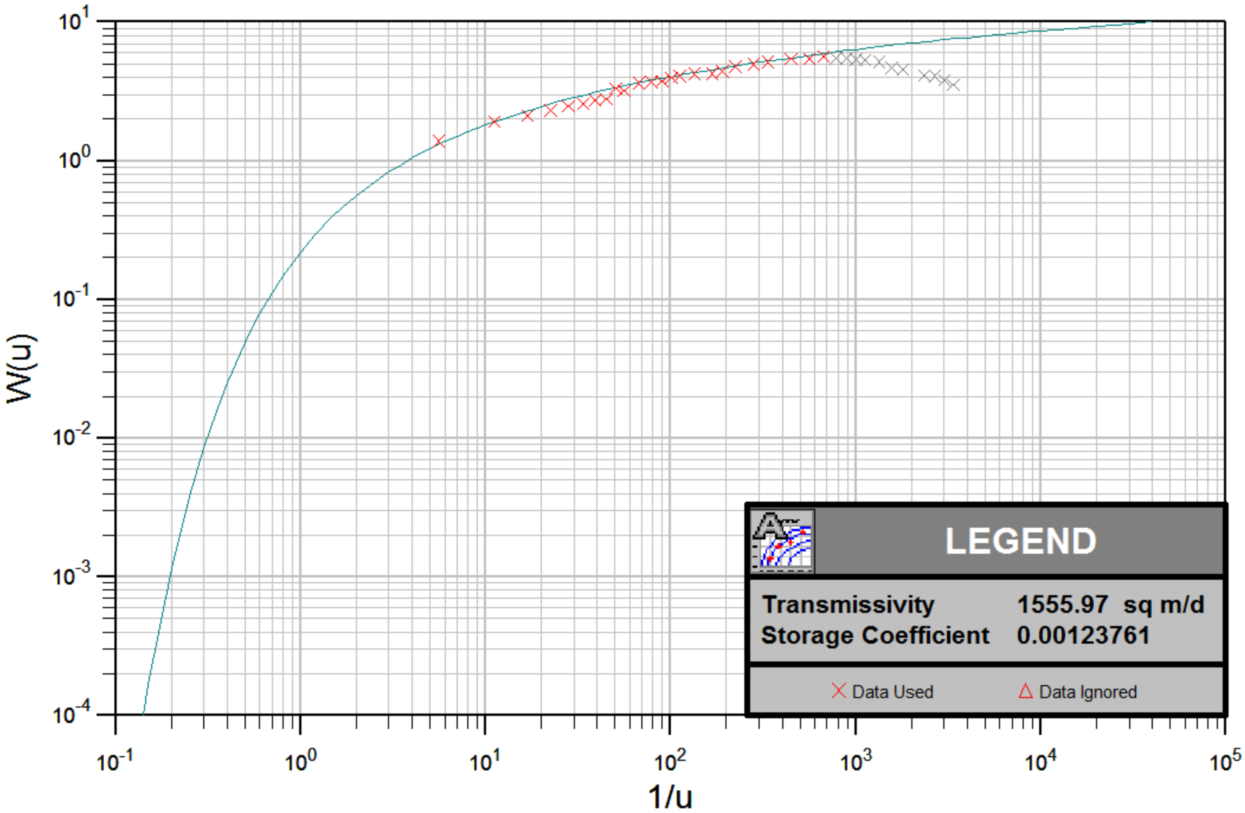
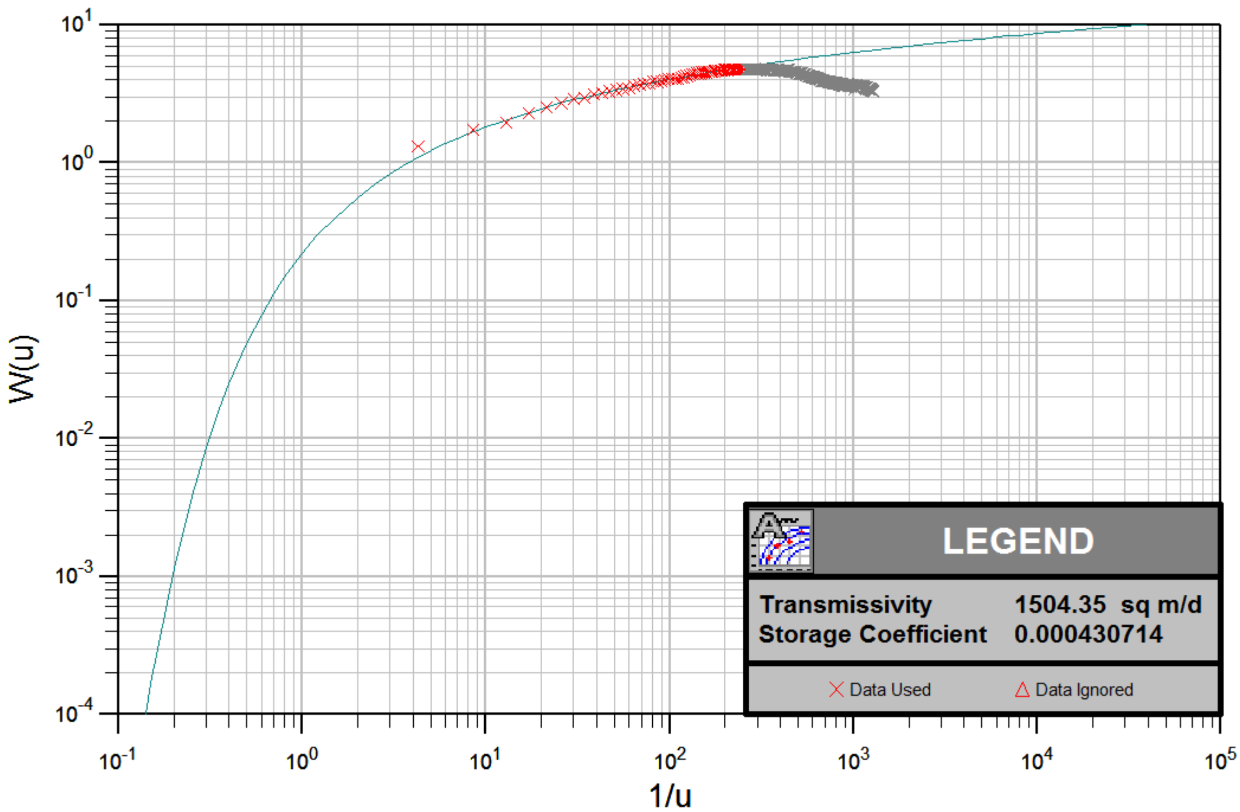


Figure 6.6 – Theis Analysis – BH29



Note: Low confidence data (t >60 minutes) shown in grey.

Figure 6.7 –Cooper and Jacob Analysis – BH18A

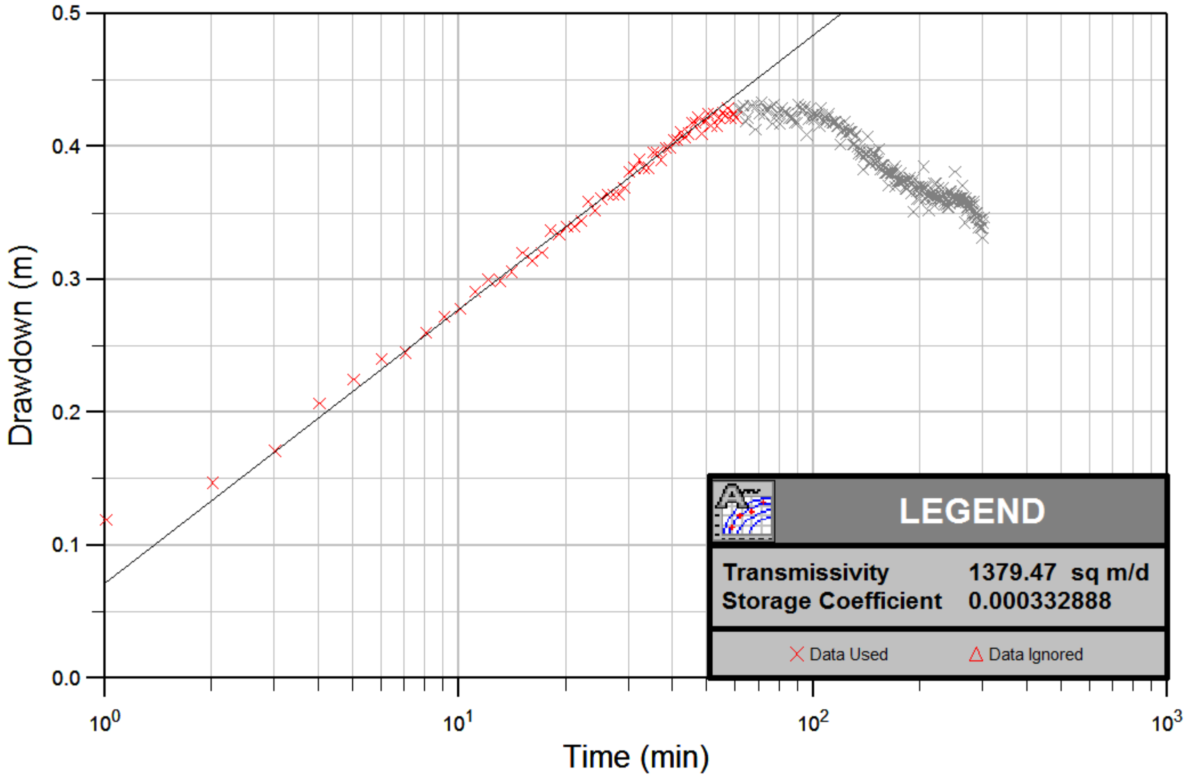
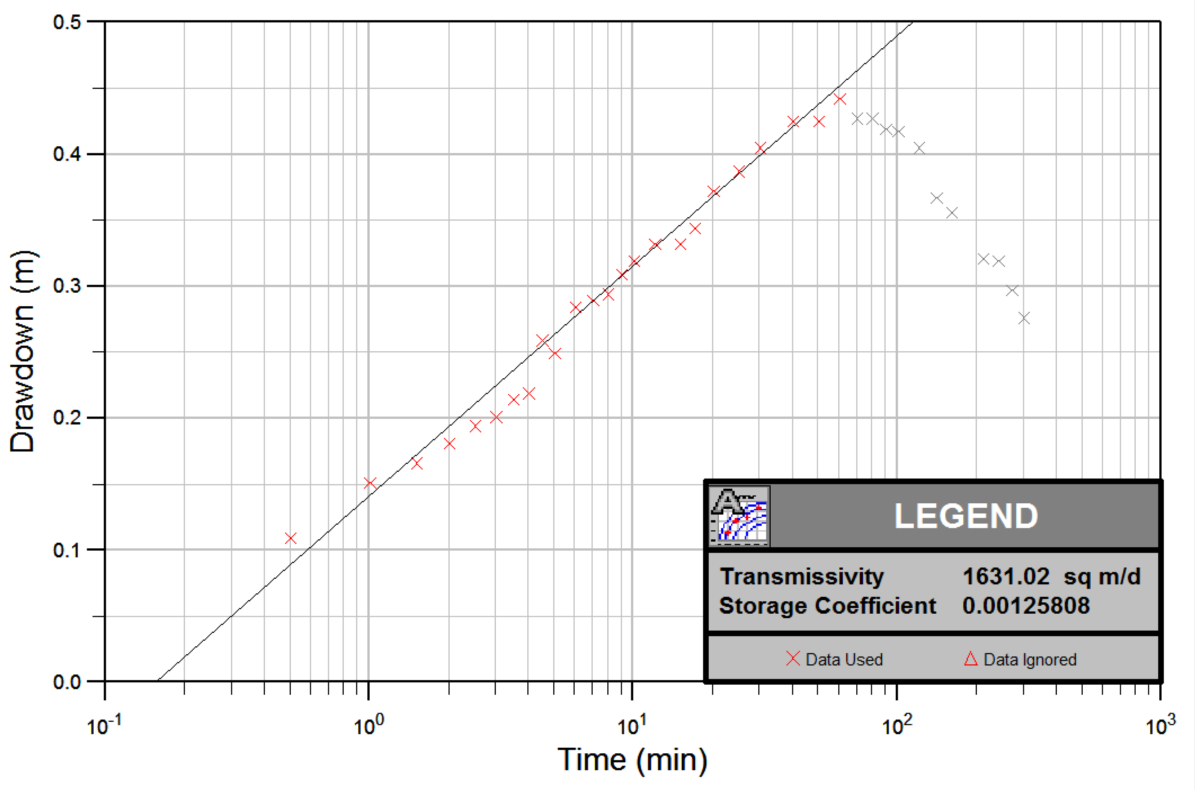
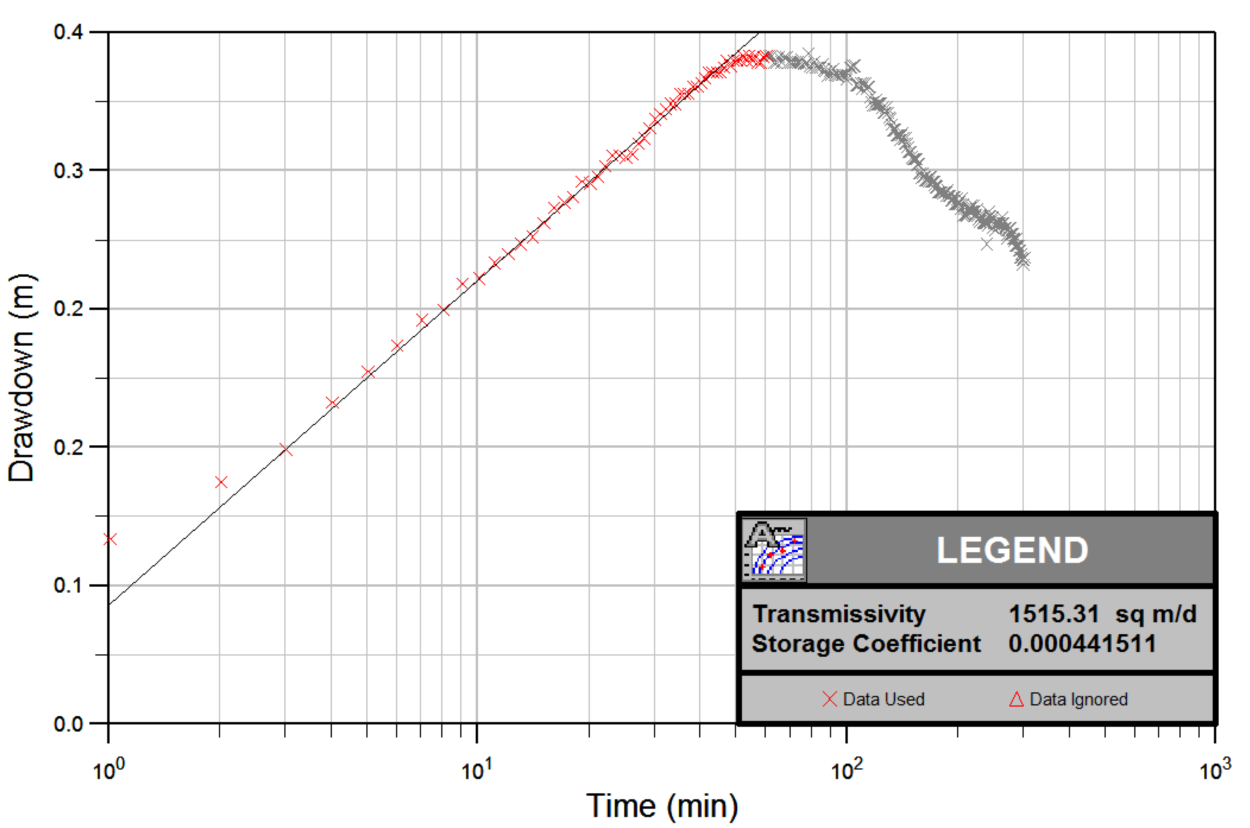


Figure 6.8 – Cooper and Jacob Analysis – BH24



Note: Low confidence data (t >60 minutes) shown in grey.

Figure 6.9 – Cooper and Jacob Analysis – BH29



Note: Low confidence data (t >60 minutes) shown in grey.

7 Conclusions

- Chalk and drift boreholes were successfully constructed close to Mytongate Junction in November 2013.

- The boreholes did not intercept water-bearing sand horizons within the drift, and it is possible that the boreholes intercept an old river channel of the River Humber.

- Background water level monitoring shows that chalk water levels closely follow tidal variations, with a lag time of between 48 and 64 minutes at BH18A, BH24 and BH29. The observation boreholes have a tidal efficiency of between 42 and 43 %, i.e. the groundwater level variation is only 43 % of tidal water level variations observed at Albert Dock.

- Groundwater levels were relatively stable prior to the tidal surge of 5 December 2013, and gradually decreasing following this and into the pumping test period.

- Drift groundwater levels primarily respond to rainfall, although a tidal response was observed in BH15. The lag time was much greater than observed in the chalk however, at 2 hours 15 minutes, and the tidal efficiency was also much smaller at 0.3 %.

- A pumping test was successfully completed in the chalk (LDBH01) at a rate of 18l/s for two days and at an increased rate of 23 l/s on the final day. Drawdown was only observed in the first 20 minutes in the chalk observation boreholes, with no further observed drawdown due to the rate increase.

- There was no observable drawdown in the drift observation boreholes, suggesting that there was no induced leakage from the drift during the tests and that there is limited hydraulic continuity between the chalk and the overlying drift deposits.

- The pumping test was successfully analysed using data super-position (to remove tidal effects), and the Theis curve fitting and Copper and Jacob straight line methods for unsteady-state flow in confined aquifers. Mean values of hydraulic conductivity of 8.31×10^{-4} m/s, transmissivity of 1436 m^2/s and storativity of 7×10^{-4} were derived. Transmissivity and storativity values derived from the pumping tests give tidal efficiency and lag times in agreement with those measured from water level monitoring, and thus confirming the accuracy of the aquifer parameters.

- Corrected drawdown data (i.e. with tidal effects removed) was seen to stabilise after 50 minutes, suggesting that a recharge boundary was encountered. This was confirmed to be the Humber Estuary by radius of influence calculations.

- The transmissivity value is much higher than values derived from other pumping tests in the immediate area, suggesting that the area lies within a zone of enhanced transmissivity either due to the presence of glacial outwash channels or fracturing within the chalk. This highlights local variability in transmissivity, and subsequently hydraulic conductivity.

- Water quality sampling confirmed that the discharge of chalk water during the pumping test had no adverse impact on water quality within the dock, and hence the estuary.

- Water quality sampling also confirmed that the Chalk water quality is heavily influenced by saline intrusion.

8 References

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The River Basin Districts Typology, Standards and Groundwater threshold values (Water Framework Directive) (England and Wales) Directions 2010

Annex 1 – Borehole Logs

Borehole record form

British Geological Survey
NATURAL ENVIRONMENT RESEARCH COUNCIL

Environment Agency

Water Resources Act 1991 (as amended by the Water Act 2003)

A Site details

Borehole drilled for: Highways Agency : Chalk Test De-Watering Well Designated: LDBH01

Location: Commercial Road, Hull HU1 2SG Grass Bank North Side of Holiday Inn Access Road

NGR (ten digits): TA 09380 28334 Please attach site plan

Ground level (if known): 4.674 metres Above Ordnance Datum

Drilling company: P R Marriott Drilling Ltd

Date drilling commenced: 14/11/2013 (DD/MM/YYYY) Completed: 26/11/2013 (DD/MM/YYYY)

B Construction details

Borehole datum (if not ground level) ______ metres (m). Please tick if this is above ☐ or below ☐ ground level.
(point from which all measurements of depth are taken, for example, flange, edge of chamber)

Borehole drilled diameter

Diameter (mm)	mm from	to (m/depth)
610	0	17
455	17	32.7
311	32.7	50

Casing material and type (for example, if plain steel, plastic slotted). Please record permanent casing details, not temporary casing.

Casing material	diameter (mm)	mm from	to (m/depth)
Steel Casing BS 879	610	0.6	17
Steel Casing BS 879	355	0.3	32.7

Grouting details: Neat cement grout in annulus 0 to 17m for 610mm casing and 0 to 32.7m for 455mm casing

Water struck at 1. 15.5 m (depth below datum – mbd) 2. ______ m (mbd)
3. ______ m (mbd) 4. ______ m (mbd)

C Test pumping summary (Please supply full details on form WR39)

Test pumping datum (if different from borehole datum) ______ m. Please tick if this is above ☐ or below ☐ ground level.

Pump suction depth: 18 mbd

Water level (start of test): 2.6 mbd

Water level (end of test): 5.6 mbd

Type of test (for example, bailer, step, constant rate): Constant Rate

Pumping rate: 23 m³/hour ☑ or litres/second ☐. Please tick as appropriate.
for 3 days, ______ hours, ______ mins

Recovery to (from end of pumping): 3.5 mbd in 1 days, ______ hours, ______ mins

Date(s) of measurements
Pump started: 09/12/2013 (DD/MM/YYYY)
Pump stopped: 12/12/2013 (DD/MM/YYYY)

Please supply chemical analysis if available. If you have included this please tick this box ☐

D Strata log

Geological classification (BGS only)	Description of strata	Thickness m	Depth (to base of strata) m
	FINE BROWN SANDY TOPSOIL	0.1	0.1
	FINE DARK BROWN SANDY SOIL WITH BRICKS, ASH AND WOOD	0.5	0.6
	FIRM BROWN VERY SILTY CLAYWITH ORGANICS	1.2	1.8
	FIRM GREY SILT	0.4	2.2
	FIRM DARK BROWN SANDY SILT	0.7	2.9
	SOFT TO FIRM BROWN SILTY CLAY. 3.5 BECAME DAMP	2	4.9
	SOFT GREY ORGANIC SILTY CLAY	1.1	6
	SOFT GREY SILT	3	9
	SOFT GREY SILTY CLAY	0.5	9.5
	SOFT TO FIRM GREY SILTY CLAY WITH ORGANIC & GREY SANDY BANDS	1.5	11
	SOFT DARK BROWN PEATY SILT	0.4	14
	SOFT BROWN SANDY PEAT	0.3	14.3
	SOFT GREY SANDY CLAY (DAMP)	1.2	15.5
	ORGANIC WOOD (TREE)	0.2	15.7
	FIRM /STIFF BROWN SANDY CLAY SMALL/MEDIUM GRAVELS,WATER 15.5m	1.6	17.3
	FIRM TO STIFF BROWN SANDY CLAY WITH SILT BANDS	2.4	20
	SOFT BROWN SILTY CLAY WITH SAND BANDS	4.3	24.3
	FINE BROWN SAND WITH SMALL CHALK GRAVELS	0.2	24.5
	COURSE BROWN SAND WITH FLINT AND CHALK GRAVELS	0.5	25
	FIRM BROWN VERY SANDY CLAY WITH FLINT AND CHALK GRAVELS	3.6	28.6
	WEAK CHALK WITH FLINTS	0.4	29
	MODERATELY STRONG WHITE CHALK WITH OCCASIONAL FLINTS	3	32
	MODERATELY STRONG WHITE CHALK WITH LARGE FLINTS	1.5	33.5
	MODERATELY STRONG WHITE CHALK WITH OCCASIONAL FLINTS	10	43.5
	STRONG WHITE CHALK	0.5	44
	STRONG YELLOW BROWN CHALK. WATER BECAME STRONGER	1	45
	STRONG GREY FLINTS	0.6	45.6
	STRONG WHITE CHALK	4.4	50
	(continue on separate page if necessary)		

Other comments (for example, gas encountered, saline water intercepted)

Chalk aquifer water saline and tidal. Pumping test figures are the upper reading at high tide. The test was run at 18 l/sec for 2 days and at 23 l/sec for a further 1 day. The test was carried out just after the surge tide of December when the area was flooded. See attached graph.
The well head was completed with a 600mm x 600mm cover in the grass verge

E Completing this form

How long did it take you to fill in this form?

For Official use only

Date received (DD/MM/YYYY) File Consent number BGS reference number

Accession number Wellmaster number SOBI number NGR

LIC NO Purpose EA reference number

Copy number Entered by

F The Data Protection Act 1998

The Environment Agency will process the information you provide so that we can:

- deal with your application;
- make sure you keep to the conditions of any consent; and
- process renewals.

The Environment Agency will pass the information provided on this form to the British Geological Survey, in accordance with Section 198 of the Water Resources Act 1991, which states that any person drilling a well or borehole more than fifty feet below the surface, shall notify the British Geological Survey of this and provide them with the information as requested on this form.

We may also process or release the information to:

- offer you documents or services relating to environmental matters;
- consult the public, public organisations and other organisations (for example, the Health and Safety Executive, local authorities, the emergency services, the Department for Environment, Food and Rural Affairs) on environmental issues;
- carry out research and development work on environmental issues;
- prevent anyone from breaking environmental law, investigate cases where environmental law may have been broken, and take any action that is needed;
- assess whether customers are satisfied with our service, and to improve our service; and
- respond to requests for information under the Freedom of Information Act 2000 and the Environmental Information Regulations 2004 (if the Data Protection Act allows).

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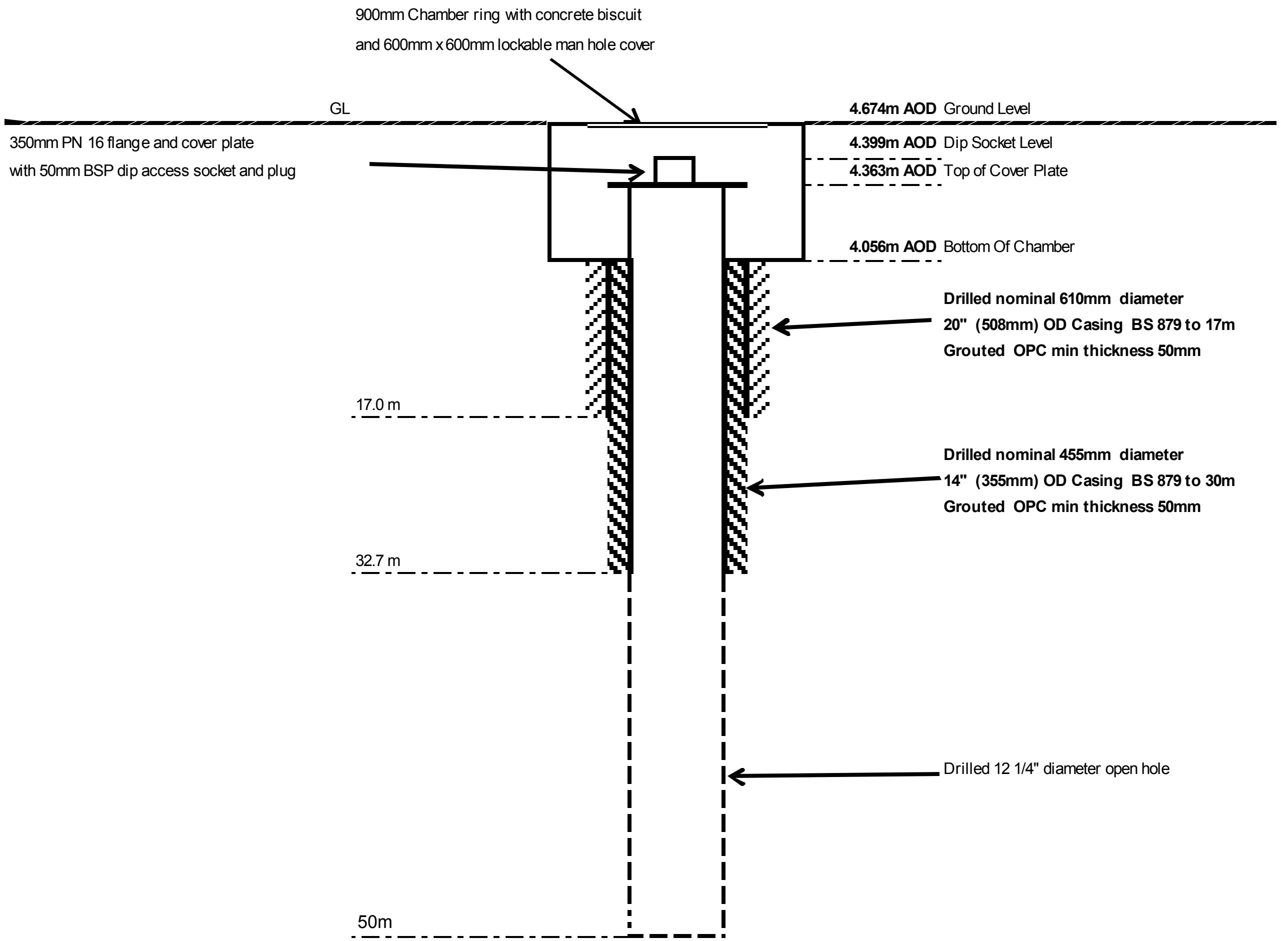
- We will also publish the information on our website; and
- provide the technical details of the borehole (for example, the depth, geology and water levels) to others. This will not include information about ownership of the borehole.

Contract: Hull A63
Well: LDBH01 Chalk Test Pumping Well
Location Coordinates: 509380.421, 428334.269



Borehole record form

British Geological Survey
NATURAL ENVIRONMENT RESEARCH COUNCIL

Environment Agency

Water Resources Act 1991 (as amended by the Water Act 2003)

A Site details

Borehole drilled for: Highways Agency : Drift Test De-Watering Well Designated: LDBH02

Location: Commercial Road, Hull HU1 2SG Grass Bank North Side of Holiday Inn Access Road

NGR (ten digits): TA 09378 28332 Please attach site plan

Ground level (if known): 4.657 metres Above Ordnance Datum

Drilling company: P R Marriott Drilling Ltd

Date drilling commenced: 09/11/2013 (DD/MM/YYYY) Completed: 14/11/2013 (DD/MM/YYYY)

B Construction details

Borehole datum (if not ground level) ______ metres (m). Please tick if this is above ☐ or below ☐ ground level.
(point from which all measurements of depth are taken, for example, flange, edge of chamber)

Borehole drilled diameter

mm	from (m)	to (m/depth)
508	0	10.5
375	10.5	22

Casing material and type (for example, if plain steel, plastic slotted). Please record permanent casing details, not temporary casing.

Casing material	diameter (mm)	from (m)	to (m/depth)
Steel Casing BS 879	406	0.68	10.5
uPVC casing	125	0.3	14
uPVC Screen 1mm slots	125	14	18.5

Grouting details: Neat cement grout in annulus 0 to 10.5m 406mm casing and 0 to 9.5m for 125mm casing, screen 2-3mm gravel

Water struck at 1. ______ m (depth below datum – mbd) 2. ______ m (mbd)
3. ______ m (mbd) 4. ______ m (mbd)

C Test pumping summary (Please supply full details on form WR39)

Test pumping datum (if different from borehole datum) ______ m. Please tick if this is above ☐ or below ☐ ground level.

Pump suction depth ______ mbd

Water level (start of test) 4.45 mbd

Water level (end of test) ______ mbd

Type of test (for example, bailer, step, constant rate)
Excessive draw down even at 1/sec test abandoned

Pumping rate ______ m^3/hour ☐ or litres/second ☐. Please tick as appropriate.
for ______ days, ______ hours, ______ mins

Recovery to ______ mbd in ______ days, ______ hours, ______ mins
(from end of pumping)

Date(s) of measurements Pump started ______ (DD/MM/YYYY)
Pump stopped ______ (DD/MM/YYYY)

Please supply chemical analysis if available. If you have included this please tick this box ☐

D Strata log

Geological classification (BGS only)	Description of strata	Thickness m	Depth (to base of strata) m
	TRIAL PIT MADE UP OF GROUND BRICK ETC	1.2	1.2
	MADE UP OF GROUND SANDY CLAYS, BRICK ETC	1.3	2.5
	BROWN SILTY CLAYS, DRY	1.5	4
	YELLOW SILTY CLAYS, DRY	0.5	4.5
	DARK GREY SILTY CLAYS, DRY	1.5	6
	DARK GREY SILTY CLAYS, WET	0.5	6.5
	SILTY DARK GREY CLAYS, WET, SOFT	4	10.5
	GREY CLAYS	0.8	11.3
	DARK SANDY PEATS, DRY	1.2	12.5
	PEAT WITH MUD, DRY	1	13.5
	GREY DAMP CLAY,SILTY,DAMP	3.5	17
	BROWN SILTY CLAYS, DAMP	1	18
	GREY BROWN SILTY CLAYS WITH CHALK SHOWS	1	19
	STIFF GREY CLAY	0.5	19.5
	SANDY CLAYS	0.5	20
	STIFF CLAY,DAMP	1	21
	STIFF CLAY WITH SAND STRINGERS,DAMP	1	22
	(continue on separate page if necessary)		

Other comments (for example, gas encountered, saline water intercepted)

Borehole completed as a monitor well to record levels in drift. Well head completed with 600mm x 600mm cover in grass bank.

E Completing this form

How long did it take you to fill in this form?

For Official use only

Date received (DD/MM/YYYY)

File

Consent number

BGS reference number

Accession number

Wellmaster number

SOBI number

NGR

LIC NO

Purpose

EA reference number

Copy number

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F The Data Protection Act 1998

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- carry out research and development work on environmental issues;
- prevent anyone from breaking environmental law, investigate cases where environmental law may have been broken, and take any action that is needed;
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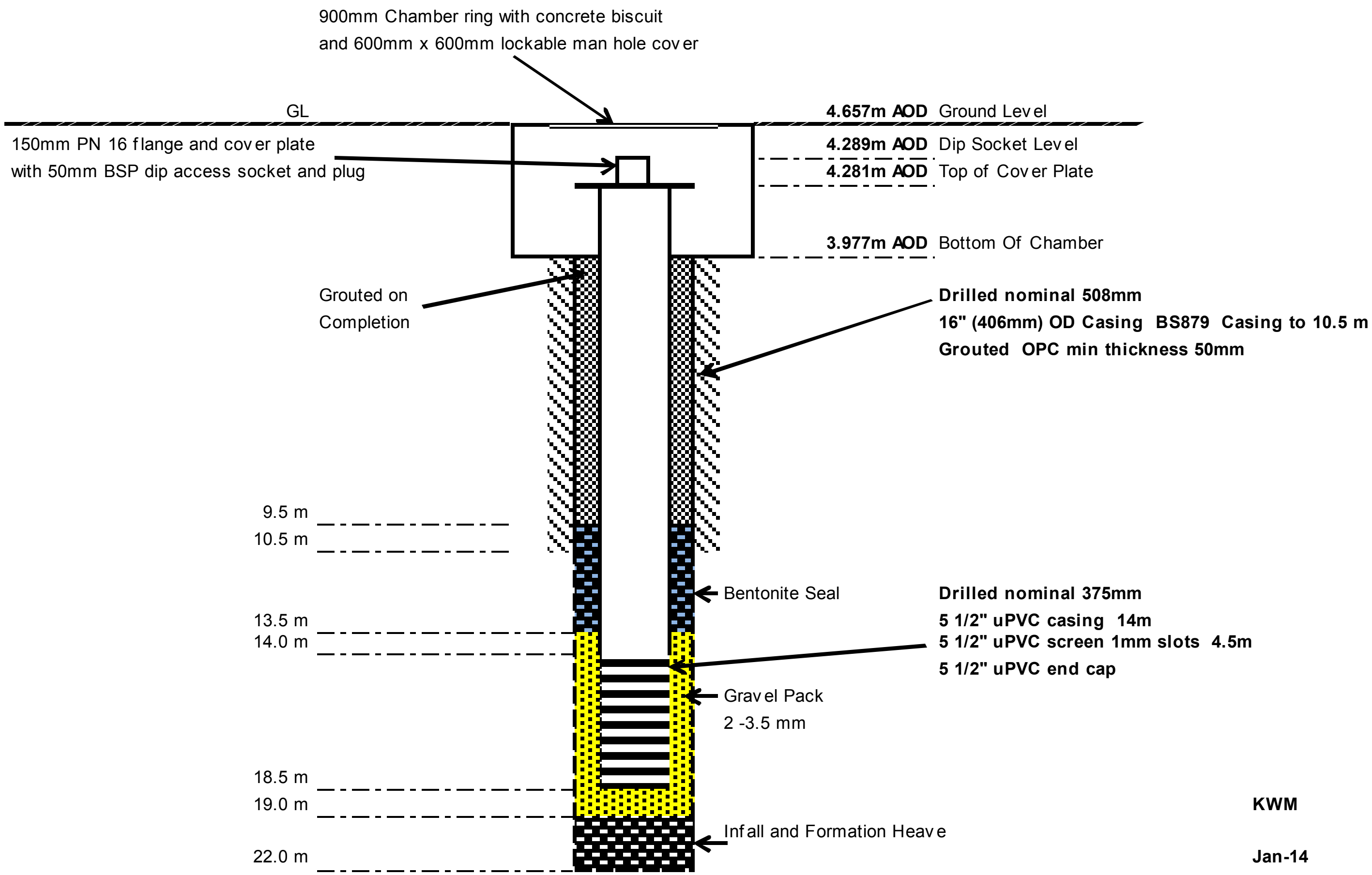
Contract **Hull A63**

Well: **LDBH02 Drift Test Pumping Well**

Location Coordinates: **509378.390, 428332.877**



KWM

Jan-14

Annex 2 – Pumping Test Manual Dip and Flow Data

Pumping Test Manual Dip and Flow Data

LDBH01 (Chalk)

Datums:	5.036 mAOD	Top edge of casing (pre pump test)
	4.677 mAOD	Top of dip tube / ground level (5 Dec 2013 onwards)

Date, time	Dip (mbd)	Dip (mAOD)	Flow Rate (l/s)	Comments
26/11/2013 13:29:00	4.42	0.616	0	
05/12/2013 09:45:00	4.08	0.597	0	
09/12/2013 08:34:00	4.27	0.407	0	
09/12/2013 10:45:00	3.75	0.927	16.5	
09/12/2013 10:45:30	5	-0.323	17	
09/12/2013 10:46:00	4.94	-0.263	17	
09/12/2013 10:46:30	5.02	-0.343	17.1	
09/12/2013 10:47:00	5.18	-0.503	17.3	
09/12/2013 10:47:30	5.19	-0.513	17.2	
09/12/2013 10:48:00	5.2	-0.523	17	
09/12/2013 10:48:30	5.2	-0.523	17.2	
09/12/2013 10:49:00	5.2	-0.523	17.4	
09/12/2013 10:49:30	5.85	-1.173	17.4	
09/12/2013 10:50:00	5.58	-0.903	18.5	
09/12/2013 10:51:00	5.27	-0.593	18.5	
09/12/2013 10:52:00	5.61	-0.933	18.5	
09/12/2013 10:53:00	5.61	-0.933	18.4	
09/12/2013 10:54:00	5.6	-0.923	18.69	
09/12/2013 10:55:00	5.61	-0.933	18.09	
09/12/2013 10:57:30	5.62	-0.943	18.13	
09/12/2013 11:00:00	5.58	-0.903	18.17	
09/12/2013 11:02:30	5.56	-0.883	18.17	
09/12/2013 11:05:00	5.54	-0.863	18.29	
09/12/2013 11:10:00	5.55	-0.873	18.12	
09/12/2013 11:15:00	5.55	-0.873	18.2	
09/12/2013 11:25:00	5.56	-0.883	18.12	
09/12/2013 11:35:00	5.56	-0.883	18.2	
09/12/2013 11:45:00	5.56	-0.883	18.46	
09/12/2013 11:55:00	5.56	-0.883	18.28	
09/12/2013 12:05:00	5.56	-0.883	18.34	
09/12/2013 12:15:00	5.58	-0.903	18.3	
09/12/2013 12:25:00	5.58	-0.903	18.38	
09/12/2013 12:45:00	5.64	-0.963	18.36	
09/12/2013 13:05:00	5.75	-1.073	18.2	
09/12/2013 13:25:00	5.84	-1.163	18.38	
09/12/2013 13:45:00	5.91	-1.233	18.45	
09/12/2013 14:15:00	6.11	-1.433	18.32	
09/12/2013 14:45:00	6.43	-1.753	18.27	
09/12/2013 15:15:00	6.66	-1.983	18.15	
09/12/2013 15:45:00	6.89	-2.213	18.22	
09/12/2013 16:15:00	7.1	-2.423	18.17	
09/12/2013 16:45:00	7.33	-2.653	18.1	
09/12/2013 17:45:00	7.63	-2.953	18.36	
09/12/2013 18:46:00	7.55	-2.873	18.4	
09/12/2013 19:46:00	7.12	-2.443	18.32	
09/12/2013 20:46:00	6.78	-2.103	18.52	
09/12/2013 21:46:00	6.45	-1.773	18.88	

Date, time	Dip (mbd)	Dip (mAOD)	Flow Rate (l/s)	Comments
09/12/2013 22:46:00	5.9	-1.223	18.92	
09/12/2013 23:46:00	5.68	-1.003	19.32	
10/12/2013 00:46:00	5.74	-1.063	18.9	
10/12/2013 01:46:00	5.95	-1.273	19.02	
10/12/2013 02:46:00	6.33	-1.653	18.71	
10/12/2013 03:46:00	6.79	-2.113	18.5	
10/12/2013 04:46:00	7.2	-2.523	18.46	
10/12/2013 05:20:00			0	Pump failed between 05:20 and 05:25
10/12/2013 05:25:00			18.25	
10/12/2013 06:46:00	7.77	-3.093	18.25	
10/12/2013 08:03:00			0	Pump failed between 08:03 and 08:06. Fuel filter changed and no further problems.
10/12/2013 08:06:00			18.4	
10/12/2013 08:46:00	7.34	-2.663	18.4	
10/12/2013 10:46:00	6.46	-1.783	18.37	
10/12/2013 12:45:00	5.85	-1.173	18.4	
10/12/2013 14:46:00	6.13	-1.453	18.26	
10/12/2013 16:46:00	6.95	-2.273	18.36	
10/12/2013 18:00:00	7.32	-2.643		
10/12/2013 18:46:00	7.5	-2.823	18.3	
10/12/2013 19:00:00	7.49	-2.813		
10/12/2013 20:46:00	7.1	-2.423	18.25	
10/12/2013 22:46:00	6.31	-1.633	18.72	
11/12/2013 00:46:00	5.79	-1.113	19	
11/12/2013 02:46:00	6.1	-1.423	18.36	
11/12/2013 04:46:00	6.87	-2.193	18.72	
11/12/2013 06:46:00	7.5	-2.823	18.09	
11/12/2013 08:46:00	7.62	-2.943	18.21	
11/12/2013 10:37:00	7.04	-2.363	18.06	
11/12/2013 10:45:00	7	-2.323	23	Rate increased to 23l/s after 48hrs testing completed at 18 l/s
11/12/2013 10:45:30	7.54	-2.863		
11/12/2013 10:46:00	7.6	-2.923		
11/12/2013 10:46:30	7.6	-2.923		
11/12/2013 10:47:00	7.6	-2.923		
11/12/2013 10:47:30	7.6	-2.923		
11/12/2013 10:48:00	7.6	-2.923		
11/12/2013 10:48:30	7.6	-2.923		
11/12/2013 10:49:00	7.6	-2.923		
11/12/2013 10:49:30	7.6	-2.923		
11/12/2013 10:50:00	7.6	-2.923		
11/12/2013 10:51:00	7.6	-2.923		
11/12/2013 10:52:00	7.59	-2.913		
11/12/2013 10:53:00	7.6	-2.923		
11/12/2013 10:54:00	7.6	-2.923		
11/12/2013 10:55:00	7.58	-2.903		
11/12/2013 10:57:30	7.57	-2.893		
11/12/2013 11:00:00	7.57	-2.893		

Date, time	Dip (mbd)	Dip (mAOD)	Flow Rate (l/s)	Comments
11/12/2013 11:02:30	7.55	-2.873		
11/12/2013 11:05:00	7.54	-2.863		
11/12/2013 11:10:00	7.52	-2.843		
11/12/2013 11:15:00	7.5	-2.823		
11/12/2013 11:25:00	7.44	-2.763		
11/12/2013 11:35:00	7.4	-2.723	23.11	
11/12/2013 11:45:00	7.33	-2.653		
11/12/2013 11:55:00	7.24	-2.563		
11/12/2013 12:05:00	7.22	-2.543	23.22	
11/12/2013 12:15:00	7.2	-2.523	23.1	
11/12/2013 12:25:00	7.14	-2.463	23.3	
11/12/2013 12:45:00	7.05	-2.373	23.39	
11/12/2013 13:05:00	6.98	-2.303	23.1	
11/12/2013 13:25:00	6.94	-2.263	23.61	
11/12/2013 13:45:00	6.92	-2.243	23.51	
11/12/2013 14:15:00	6.86	-2.183	23.4	
11/12/2013 14:45:00	6.88	-2.203	23.84	
11/12/2013 15:15:00	6.92	-2.243	23.68	
11/12/2013 15:45:00	7	-2.323	23.38	
11/12/2013 16:15:00	7.16	-2.483	23.43	
11/12/2013 16:45:00	7.32	-2.643	23	
11/12/2013 17:45:00	7.67	-2.993	22.9	
11/12/2013 18:45:00	8	-3.323	22.23	
11/12/2013 19:45:00	8.16	-3.483	22.11	
11/12/2013 20:45:00	8.12	-3.443	22.11	
11/12/2013 21:45:00	7.94	-3.263	22.09	
11/12/2013 22:45:00	7.6	-2.923	22.07	
11/12/2013 23:45:00	7.17	-2.493	22.78	
12/12/2013 00:45:00	6.88	-2.203	23.04	
12/12/2013 01:45:00	6.84	-2.163	23.54	
12/12/2013 02:45:00	6.78	-2.103	23.74	
12/12/2013 03:45:00	7.01	-2.333	23.17	
12/12/2013 04:45:00	7.33	-2.653	23.03	
12/12/2013 06:45:00	8.02	-3.343	22.5	
12/12/2013 08:45:00	8.48	-3.803	22	
12/12/2013 10:45:00	8.25	-3.573	0	Pump switched off at 12/12/13 10:45
12/12/2013 10:45:30	5.8	-1.123		
12/12/2013 10:46:00	5.76	-1.083		
12/12/2013 10:46:30	5.76	-1.083		
12/12/2013 10:47:00	5.78	-1.103		
12/12/2013 10:47:30	5.74	-1.063		
12/12/2013 10:48:00	5.73	-1.053		
12/12/2013 10:48:30	5.72	-1.043		
12/12/2013 10:49:00	5.7	-1.023		
12/12/2013 10:49:30	5.67	-0.993		
12/12/2013 10:50:00	5.67	-0.993		
12/12/2013 10:51:00	5.64	-0.963		
12/12/2013 10:52:00	5.62	-0.943		
12/12/2013 10:53:00	5.6	-0.923		
12/12/2013 10:54:00	5.59	-0.913		
12/12/2013 10:55:00	5.57	-0.893		
12/12/2013 10:57:30	5.53	-0.853		

Date, time	Dip (mbd)	Dip (mAOD)	Flow Rate (l/s)	Comments
12/12/2013 11:00:00	5.51	-0.833		
12/12/2013 11:02:30	5.47	-0.793		
12/12/2013 11:05:00	5.44	-0.763		
12/12/2013 11:10:00	5.38	-0.703		
12/12/2013 11:15:00	5.32	-0.643		
12/12/2013 11:25:00	5.24	-0.563		
12/12/2013 11:35:00	5.18	-0.503		
12/12/2013 11:45:00	5.06	-0.383		
12/12/2013 11:55:00	4.97	-0.293		
12/12/2013 12:05:00	4.88	-0.203		
12/12/2013 12:15:00	4.8	-0.123		
12/12/2013 12:25:00	4.72	-0.043		
12/12/2013 12:45:00	4.56	0.117		
12/12/2013 13:05:00	4.38	0.297		
12/12/2013 13:25:00	4.25	0.427		
12/12/2013 13:45:00	4.11	0.567		
12/12/2013 14:15:00	3.96	0.717		
12/12/2013 14:45:00	3.8	0.877		
12/12/2013 15:15:00	3.74	0.937		
12/12/2013 15:45:00	3.67	1.007		
12/12/2013 16:15:00	3.74	0.937		
12/12/2013 16:45:00	3.8	0.877		
12/12/2013 17:45:00	4.1	0.577		
12/12/2013 18:45:00	4.52	0.157		
12/12/2013 19:45:00	4.94	-0.263		
12/12/2013 20:45:00	5.04	-0.363		
12/12/2013 21:45:00	5.05	-0.373		
12/12/2013 22:45:00	4.45	0.227		
12/12/2013 23:45:00	4.07	0.607		
13/12/2013 00:45:00	3.67	1.007		
13/12/2013 01:45:00	3.44	1.237		
13/12/2013 03:45:00	3.5	1.177		
13/12/2013 04:45:00	3.7	0.977		
13/12/2013 06:45:00	4.59	0.087		
13/12/2013 08:46:00	5.31	-0.633		
13/12/2013 10:46:00	5.49	-0.813		

Pumping Test Observation Boreholes Manual Dip Data

BH18A (Chalk)

Datum: 3.52 mAOD Top edge of cover (ground level)
- taken from the edge across from the hinge

Date, time	Dip (mbd)	Dip (mAOD)
11/11/2013 15:50	3.18	0.34
12/11/2013 14:04	2.445	1.075
13/11/2013 08:57	4.215	-0.695
14/11/2013 15:24	2.185	1.335
21/11/2013 15:28	4.13	-0.61
22/11/2013 11:48	2.835	0.685
25/11/2013 16:07	3.87	-0.35
26/11/2013 15:16	3.25	0.27
09/12/2013 10:25	2.66	0.86
09/12/2013 11:18	2.735	0.785
09/12/2013 11:32	2.72	0.8
09/12/2013 12:05	2.71	0.81
09/12/2013 13:10	2.9	0.62
09/12/2013 15:36	3.98	-0.46
09/12/2013 16:42	4.45	-0.93
09/12/2013 17:48	4.75	-1.23
10/12/2013 08:49	4.47	-0.95
10/12/2013 10:48	3.61	-0.09
10/12/2013 13:19	3.03	0.49
10/12/2013 15:06	3.41	0.11
11/12/2013 10:16	4.295	-0.775
11/12/2013 14:54	3.18	0.34
12/12/2013 09:46	4.96	-1.44
12/12/2013 13:11	3.17	0.35
12/12/2013 14:25	2.71	0.81
12/12/2013 15:25	2.52	1
12/12/2013 16:25	2.535	0.985
13/12/2013 10:33	4.302	-0.782

BH24 (Chalk)

Datum: 4.896 mAOD Top edge of cover (ground level)
- taken from the edge across from the hinge

Date, time	Dip (mbd)	Dip (mAOD)
06/11/2013 12:08	4.5	0.396
07/11/2013 12:28	4.336	0.56
08/11/2013 16:11	5.56	-0.664
11/11/2013 15:20	4.36	0.536
12/11/2013 14:45	3.68	1.216
14/11/2013 07:56	4.89	0.006
15/11/2013 08:46	4.97	-0.074
15/11/2013 09:54	5.5	-0.604
18/11/2013 16:02	4.795	0.101
19/11/2013 14:30	3.805	1.091
21/11/2013 09:33	3.805	1.091
22/11/2013 09:40	3.65	1.246
25/11/2013 07:30	5.25	-0.354
25/11/2013 15:44	5.2	-0.304

Date, time	Dip (mbd)	Dip (mAOD)
26/11/2013 08:30	5.5	-0.604
26/11/2013 13:30	4.16	0.736
09/12/2013 08:31	4.98	-0.084
09/12/2013 10:45	3.86	1.036
09/12/2013 10:45	3.97	0.926
09/12/2013 10:46	4	0.896
09/12/2013 10:46	4.015	0.881
09/12/2013 10:47	4.03	0.866
09/12/2013 10:47	4.045	0.851
09/12/2013 10:48	4.05	0.846
09/12/2013 10:48	4.055	0.841
09/12/2013 10:49	4.06	0.836
09/12/2013 10:49	4.1	0.796
09/12/2013 10:50	4.09	0.806
09/12/2013 10:51	4.105	0.791
09/12/2013 10:52	4.11	0.786
09/12/2013 10:53	4.115	0.781
09/12/2013 10:54	4.12	0.776
09/12/2013 10:55	4.12	0.776
09/12/2013 10:57	4.125	0.771
09/12/2013 11:00	4.105	0.791
09/12/2013 11:02	4.105	0.791
09/12/2013 11:05	4.125	0.771
09/12/2013 11:10	4.12	0.776
09/12/2013 11:15	4.12	0.776
09/12/2013 11:25	4.11	0.786
09/12/2013 11:35	4.1	0.796
09/12/2013 11:45	4.095	0.801
09/12/2013 11:55	4.1	0.796
09/12/2013 12:05	4.11	0.786
09/12/2013 12:15	4.12	0.776
09/12/2013 12:25	4.14	0.756
09/12/2013 12:45	4.2	0.696
09/12/2013 13:05	4.275	0.621
09/12/2013 13:25	4.375	0.521
09/12/2013 14:15	4.72	0.176
09/12/2013 14:45	4.965	-0.069
09/12/2013 15:15	5.195	-0.299
09/12/2013 15:45	5.44	-0.544
09/12/2013 16:15	5.665	-0.769
09/12/2013 16:45	5.86	-0.964
09/12/2013 17:45	6.13	-1.234
09/12/2013 18:45	6.04	-1.144
09/12/2013 19:45	5.7	-0.804
09/12/2013 20:45	5.29	-0.394
09/12/2013 21:45	4.53	0.366
09/12/2013 22:45	4.82	0.076
09/12/2013 23:45	4.92	-0.024
10/12/2013 00:45	4.1	0.796
10/12/2013 01:45	4.39	0.506
10/12/2013 02:45	4.8	0.096
10/12/2013 03:45	5.28	-0.384
10/12/2013 04:45	5.74	-0.844

Date, time	Dip (mbd)	Dip (mAOD)
10/12/2013 06:45	6.3	-1.404
10/12/2013 08:45	5.85	-0.954
10/12/2013 10:45	4.99	-0.094
10/12/2013 12:45	4.41	0.486
10/12/2013 14:45	4.65	0.246
10/12/2013 16:45	5.48	-0.584
10/12/2013 18:00	5.52	-0.624
10/12/2013 18:45	6.03	-1.134
10/12/2013 19:00	5.63	-0.734
10/12/2013 20:45	5.61	-0.714
10/12/2013 22:45	4.79	0.106
11/12/2013 00:45	4.19	0.706
11/12/2013 02:45	4.5	0.396
11/12/2013 04:45	5.29	-0.394
11/12/2013 06:45	6.03	-1.134
11/12/2013 08:45	6.13	-1.234
11/12/2013 10:35	5.54	-0.644
11/12/2013 10:45	5.47	-0.574
11/12/2013 10:45	5.49	-0.594
11/12/2013 10:46	5.5	-0.604
11/12/2013 10:46	5.502	-0.606
11/12/2013 10:47	5.505	-0.609
11/12/2013 10:47	5.5	-0.604
11/12/2013 10:48	5.498	-0.602
11/12/2013 10:48	5.5	-0.604
11/12/2013 10:49	5.498	-0.602
11/12/2013 10:50	5.49	-0.594
11/12/2013 10:51	5.49	-0.594
11/12/2013 10:52	5.485	-0.589
11/12/2013 10:53	5.48	-0.584
11/12/2013 10:54	5.475	-0.579
11/12/2013 10:55	5.472	-0.576
11/12/2013 10:57	5.455	-0.559
11/12/2013 11:00	5.42	-0.524
11/12/2013 11:02	5.405	-0.509
11/12/2013 11:05	5.39	-0.494
11/12/2013 11:10	5.35	-0.454
11/12/2013 11:15	5.325	-0.429
11/12/2013 11:25	5.265	-0.369
11/12/2013 11:35	5.21	-0.314
11/12/2013 11:45	5.12	-0.224
11/12/2013 11:55	5.055	-0.159
11/12/2013 12:05	4.995	-0.099
11/12/2013 12:15	4.93	-0.034
11/12/2013 12:25	4.87	0.026
11/12/2013 12:45	4.765	0.131
11/12/2013 13:05	4.67	0.226
11/12/2013 13:25	4.602	0.294
11/12/2013 13:45	4.55	0.346
11/12/2013 14:15	4.52	0.376
11/12/2013 14:45	4.53	0.366
11/12/2013 15:15	4.595	0.301
11/12/2013 15:45	4.7	0.196

Date, time	Dip (mbd)	Dip (mAOD)
11/12/2013 16:15	4.865	0.031
11/12/2013 16:45	5.06	-0.164
11/12/2013 17:45	5.46	-0.564
11/12/2013 18:45	5.9	-1.004
11/12/2013 19:45	6.02	-1.124
11/12/2013 20:45	5.97	-1.074
11/12/2013 21:45	5.74	-0.844
11/12/2013 22:45	5.38	-0.484
11/12/2013 23:45	4.98	-0.084
12/12/2013 00:45	4.58	0.316
12/12/2013 01:45	4.33	0.566
12/12/2013 02:45	4.39	0.506
12/12/2013 03:45	4.62	0.276
12/12/2013 04:45	4.99	-0.094
12/12/2013 06:45	5.81	-0.914
12/12/2013 08:45	6.37	-1.474
12/12/2013 10:45	6.12	-1.224
12/12/2013 10:45	5.95	-1.054
12/12/2013 10:46	5.925	-1.029
12/12/2013 10:46	5.9	-1.004
12/12/2013 10:47	5.88	-0.984
12/12/2013 10:47	5.86	-0.964
12/12/2013 10:48	5.845	-0.949
12/12/2013 10:48	5.83	-0.934
12/12/2013 10:49	5.815	-0.919
12/12/2013 10:49	5.805	-0.909
12/12/2013 10:50	5.795	-0.899
12/12/2013 10:51	5.775	-0.879
12/12/2013 10:52	5.775	-0.879
12/12/2013 10:53	5.735	-0.839
12/12/2013 10:54	5.72	-0.824
12/12/2013 10:55	5.705	-0.809
12/12/2013 10:57	5.665	-0.769
12/12/2013 11:00	5.635	-0.739
12/12/2013 11:02	5.605	-0.709
12/12/2013 11:05	5.6	-0.704
12/12/2013 11:10	5.525	-0.629
12/12/2013 11:15	5.475	-0.579
12/12/2013 11:25	5.38	-0.484
12/12/2013 11:35	5.275	-0.379
12/12/2013 11:45	5.205	-0.309
12/12/2013 11:55	5.115	-0.219
12/12/2013 12:05	5.03	-0.134
12/12/2013 12:15	4.94	-0.044
12/12/2013 12:25	4.86	0.036
12/12/2013 12:45	4.7	0.196
12/12/2013 13:05	4.55	0.346
12/12/2013 13:25	4.405	0.491
12/12/2013 13:45	4.27	0.626
12/12/2013 14:15	4.1	0.796
12/12/2013 14:45	3.965	0.931
12/12/2013 15:15	3.88	1.016
12/12/2013 15:45	3.84	1.056

Date, time	Dip (mbd)	Dip (mAOD)
12/12/2013 16:15	3.855	1.041
12/12/2013 16:45	3.93	0.966
12/12/2013 17:45	4.245	0.651
12/12/2013 18:45	4.59	0.306
12/12/2013 19:45	4.94	-0.044
12/12/2013 20:45	5.15	-0.254
12/12/2013 21:45	5.15	-0.254
12/12/2013 22:45	4.92	-0.024
12/12/2013 23:45	4.81	0.086
13/12/2013 00:45	4.58	0.316
13/12/2013 01:45	4.17	0.726
13/12/2013 02:45	3.78	1.116
13/12/2013 03:45	3.57	1.326
13/12/2013 04:45	3.51	1.386
13/12/2013 06:45	4.63	0.266
13/12/2013 08:45	5.45	-0.554
13/12/2013 10:45	5.63	-0.734

BH29 (Chalk)

Datum: 4.5 mAOD Top edge of cover (ground level)
- taken from the edge across from the hinge

Date, time	Dip (mbd)	Dip (mAOD)
06/11/2013 11:53	3.978	0.522
07/11/2013 12:14	3.79	0.71
08/11/2013 15:59	5.11	-0.61
11/11/2013 16:43	3.73	0.77
13/11/2013 09:30	5.195	-0.695
14/11/2013 15:08	3.1	1.4
15/11/2013 08:58	4.603	-0.103
15/11/2013 09:44	4.98	-0.48
18/11/2013 16:05	4.235	0.265
19/11/2013 14:56	5.06	-0.56
21/11/2013 09:51	3.325	1.175
22/11/2013 09:48	3.1	1.4
25/11/2013 15:57	3.835	0.665
26/11/2013 13:46	3.62	0.88
09/12/2013 10:45	3.45	1.05
09/12/2013 10:45	3.51	0.99
09/12/2013 10:46	3.54	0.96
09/12/2013 10:46	3.56	0.94
09/12/2013 10:47	3.57	0.93
09/12/2013 10:47	3.58	0.92
09/12/2013 10:48	3.59	0.91
09/12/2013 10:48	3.6	0.9
09/12/2013 10:49	3.6	0.9
09/12/2013 10:49	3.61	0.89
09/12/2013 10:50	3.62	0.88
09/12/2013 10:51	3.63	0.87
09/12/2013 10:52	3.635	0.865
09/12/2013 10:53	3.64	0.86
09/12/2013 10:54	3.65	0.85
09/12/2013 10:55	3.65	0.85

Date, time	Dip (mbd)	Dip (mAOD)
09/12/2013 10:57	3.65	0.85
09/12/2013 11:00	3.65	0.85
09/12/2013 11:02	3.65	0.85
09/12/2013 11:05	3.65	0.85
09/12/2013 11:10	3.65	0.85
09/12/2013 11:15	3.64	0.86
09/12/2013 11:25	3.63	0.87
09/12/2013 11:35	3.62	0.88
09/12/2013 11:45	3.62	0.88
09/12/2013 11:55	3.63	0.87
09/12/2013 12:05	3.63	0.87
09/12/2013 12:15	3.65	0.85
09/12/2013 12:25	3.675	0.825
09/12/2013 12:45	3.74	0.76
09/12/2013 13:05	3.82	0.68
09/12/2013 13:25	3.93	0.57
09/12/2013 13:45	4.07	0.43
09/12/2013 14:15	4.3	0.2
09/12/2013 14:45	4.54	-0.04
09/12/2013 15:15	4.8	-0.3
09/12/2013 15:45	5.04	-0.54
09/12/2013 16:15	5.27	-0.77
09/12/2013 16:45	5.47	-0.97
09/12/2013 17:45	5.75	-1.25
09/12/2013 18:45	5.64	-1.14
09/12/2013 19:45	5.25	-0.75
09/12/2013 20:45	4.74	-0.24
09/12/2013 21:45	4.27	0.23
09/12/2013 22:45	3.78	0.72
09/12/2013 23:45	3.55	0.95
10/12/2013 00:45	4.64	-0.14
10/12/2013 01:45	3.96	0.54
10/12/2013 02:45	4.4	0.1
10/12/2013 03:45	4.9	-0.4
10/12/2013 04:45	5.2	-0.7
10/12/2013 06:45	5.91	-1.41
10/12/2013 08:45	4.43	0.07
10/12/2013 10:45	4.53	-0.03
10/12/2013 12:45	3.94	0.56
10/12/2013 14:45	4.09	0.41
10/12/2013 16:45	4.95	-0.45
10/12/2013 18:02	5.52	-1.02
10/12/2013 18:45	5.69	-1.19
10/12/2013 19:00	5.63	-1.13
10/12/2013 20:45	5.2	-0.7
10/12/2013 22:45	4.31	0.19
11/12/2013 00:45	3.72	0.78
11/12/2013 02:45	4.06	0.44
11/12/2013 04:45	4.9	-0.4
11/12/2013 06:45	5.5	-1
11/12/2013 08:45	5.6	-1.1
11/12/2013 10:34	5.2	-0.7
11/12/2013 11:27	4.79	-0.29

Date, time	Dip (mbd)	Dip (mAOD)
11/12/2013 11:49	4.65	-0.15
11/12/2013 11:59	4.55	-0.05
11/12/2013 12:07	4.48	0.02
11/12/2013 13:05	4.1	0.4
11/12/2013 13:25	4.05	0.45
11/12/2013 13:45	4.03	0.47
11/12/2013 14:15	4.03	0.47
11/12/2013 14:45	4.13	0.37
11/12/2013 15:15	4.24	0.26
11/12/2013 15:45	4.41	0.09
11/12/2013 16:15	4.62	-0.12
11/12/2013 17:45	5.05	-0.55
11/12/2013 18:45	5.41	-0.91
11/12/2013 19:45	5.61	-1.11
11/12/2013 20:45	5.55	-1.05
11/12/2013 21:45	5.31	-0.81
11/12/2013 22:45	4.93	-0.43
11/12/2013 23:45	4.49	0.01
12/12/2013 00:45	4.11	0.39
12/12/2013 01:45	3.8	0.7
12/12/2013 02:45	3.93	0.57
12/12/2013 03:45	4.19	0.31
12/12/2013 04:45	4.59	-0.09
12/12/2013 06:45	5.43	-0.93
12/12/2013 08:45	5.97	-1.47
12/12/2013 10:43	5.68	-1.18
12/12/2013 10:46	5.52	-1.02
12/12/2013 10:47	5.5	-1
12/12/2013 10:47	5.48	-0.98
12/12/2013 10:48	5.4	-0.9
12/12/2013 10:48	5.445	-0.945
12/12/2013 10:49	5.43	-0.93
12/12/2013 10:49	5.42	-0.92
12/12/2013 10:50	5.41	-0.91
12/12/2013 10:51	5.387	-0.887
12/12/2013 10:52	5.365	-0.865
12/12/2013 10:53	5.35	-0.85
12/12/2013 10:54	5.33	-0.83
12/12/2013 10:55	5.315	-0.815
12/12/2013 10:57	5.28	-0.78
12/12/2013 11:00	5.24	-0.74
12/12/2013 11:02	5.21	-0.71
12/12/2013 11:05	5.185	-0.685
12/12/2013 11:10	5.12	-0.62
12/12/2013 11:15	5.075	-0.575
12/12/2013 11:25	4.98	-0.48
12/12/2013 11:35	4.89	-0.39
12/12/2013 11:45	4.8	-0.3
12/12/2013 11:55	4.7	-0.2
12/12/2013 12:05	4.62	-0.12
12/12/2013 12:15	4.53	-0.03
12/12/2013 12:25	4.45	0.05
12/12/2013 12:45	4.28	0.22

Date, time	Dip (mbd)	Dip (mAOD)
12/12/2013 13:05	4.123	0.377
12/12/2013 13:25	3.98	0.52
12/12/2013 13:45	3.83	0.67
12/12/2013 14:15	3.67	0.83
12/12/2013 14:45	3.83	0.67
12/12/2013 15:15	3.45	1.05
12/12/2013 15:45	3.425	1.075
12/12/2013 16:15	3.44	1.06
12/12/2013 16:45	3.525	0.975
12/12/2013 17:45	3.845	0.655
12/12/2013 18:45	4.24	0.26
12/12/2013 19:45	4.61	-0.11
12/12/2013 20:45	4.83	-0.33
12/12/2013 21:45	5.79	-1.29
12/12/2013 22:45	4.59	-0.09
12/12/2013 23:45	4.19	0.31
13/12/2013 00:45	3.9	0.6
13/12/2013 01:45	3.4	1.1
13/12/2013 02:45	3.2	1.3
13/12/2013 03:45	3.2	1.3
13/12/2013 04:45	3.48	1.02
13/12/2013 06:45	4.27	0.23
13/12/2013 08:46	5.085	-0.585
13/12/2013 10:45	5.265	-0.765

BH14 (Cohesive Alluvium)

Datum: 3.6 mAOD Top edge of cover (ground level)
- taken from the edge across from the hinge

Date, time	Dip (mbd)	Dip (mAOD)
11/11/2013 15:48	3.075	0.525
12/11/2013 14:06	3.085	0.515
13/11/2013 08:58	3.095	0.505
14/11/2013 15:17	3.083	0.517
19/11/2013 16:56	3.1	0.5
22/11/2013 11:52	3.075	0.525
25/11/2013 16:09	3.115	0.485
26/11/2013 15:17	3.135	0.465
09/12/2013 10:26	3.085	0.515
09/12/2013 11:20	3.075	0.525
09/12/2013 11:35	3.07	0.53
09/12/2013 12:10	3.09	0.51
09/12/2013 12:28	3.09	0.51
09/12/2013 12:49	3.09	0.51
09/12/2013 13:12	3.09	0.51
09/12/2013 15:37	3.08	0.52
09/12/2013 16:43	3.07	0.53
09/12/2013 17:49	3.08	0.52
10/12/2013 08:50	3.1	0.5
10/12/2013 10:49	3.09	0.51
10/12/2013 13:20	3.09	0.51
10/12/2013 15:07	3.09	0.51
10/12/2013 17:15	3.095	0.505

Date, time	Dip (mbd)	Dip (mAOD)
11/12/2013 10:17	3.095	0.505
11/12/2013 14:55	3.09	0.51
12/12/2013 09:49	3.09	0.51
12/12/2013 13:12	3.09	0.51
12/12/2013 14:36	3.095	0.505
12/12/2013 15:26	3.095	0.505
12/12/2013 16:26	3.095	0.505
13/12/2013 10:34	3.09	0.51

BH15 (Glacial Till)

Datum: 3.55 mAOD Top edge of cover (ground level)
- taken from the edge across from the hinge

Date, time	Dip (mbd)	Dip (mAOD)
11/11/2013 15:47	3.095	0.455
12/11/2013 14:07	3.145	0.405
13/11/2013 09:00	3.15	0.4
14/11/2013 15:19	3.11	0.44
19/11/2013 16:54	3.08	0.47
21/11/2013 15:30	2.985	0.565
22/11/2013 22:53	3.13	0.42
25/11/2013 16:10	3.2	0.35
26/11/2013 15:19	3.19	0.36
09/12/2013 10:28	3.12	0.43
09/12/2013 11:21	3.115	0.435
09/12/2013 11:37	3.065	0.485
09/12/2013 12:08	3.15	0.4
09/12/2013 12:27	3.15	0.4
09/12/2013 12:47	3.12	0.43
09/12/2013 13:11	3.12	0.43
09/12/2013 15:38	3.12	0.43
09/12/2013 17:51	3.12	0.43
10/12/2013 08:52	3.165	0.385
10/12/2013 10:50	3.17	0.38
10/12/2013 13:20	3.18	0.37
10/12/2013 15:08	3.19	0.36
10/12/2013 17:15	3.19	0.36
11/12/2013 10:18	3.195	0.355
11/12/2013 14:56	3.18	0.37
12/12/2013 09:51	3.17	0.38
12/12/2013 13:14	3.175	0.375
12/12/2013 14:28	3.18	0.37
12/12/2013 15:28	3.18	0.37
12/12/2013 16:28	3.18	0.37
13/12/2013 10:35	3.12	0.43

BH25 (Glacial Till)

Datum: 4.647 mOAD Top edge of cover (ground level)
- taken from the edge across from the hinge

Date time	Dip (mbd)	Dip (mAOD)
06/11/2013 12:04	3.962	0.685
07/11/2013 12:25	3.886	0.761
08/11/2013 16:09	3.78	0.867
11/11/2013 15:18	3.845	0.802
14/11/2013 07:53	4.065	0.582
15/11/2013 08:44	4.185	0.462
18/11/2013 15:57	3.71	0.937
19/11/2013 14:35	4.55	0.097
21/11/2013 09:25	3.815	0.832
22/11/2013 09:44	3.805	0.842
25/11/2013 07:30	3.91	0.737
25/11/2013 15:47	3.8	0.847
26/11/2013 08:30	3.82	0.827
26/11/2013 13:28	3.92	0.727
09/12/2013 08:41	3.945	0.702
09/12/2013 10:45	3.96	0.687
09/12/2013 10:45	3.96	0.687
09/12/2013 10:46	3.96	0.687
09/12/2013 10:47	3.96	0.687
09/12/2013 10:47	3.96	0.687
09/12/2013 10:48	3.96	0.687
09/12/2013 10:50	3.96	0.687
09/12/2013 11:05	3.945	0.702
09/12/2013 11:10	3.945	0.702
09/12/2013 11:25	3.93	0.717
09/12/2013 11:36	3.93	0.717
09/12/2013 12:19	3.94	0.707
09/12/2013 12:39	3.93	0.717
09/12/2013 12:48	3.95	0.697
09/12/2013 13:08	3.945	0.702
09/12/2013 13:28	3.945	0.702
09/12/2013 14:18	3.945	0.702
09/12/2013 14:48	3.95	0.697
09/12/2013 15:18	3.95	0.697
09/12/2013 15:48	3.95	0.697
09/12/2013 16:18	3.95	0.697
09/12/2013 16:48	3.95	0.697
09/12/2013 17:48	3.95	0.697
09/12/2013 10:45	3.96	0.687
09/12/2013 10:45	3.96	0.687
09/12/2013 10:46	3.96	0.687
09/12/2013 10:47	3.96	0.687
09/12/2013 10:47	3.96	0.687
09/12/2013 10:48	3.96	0.687
09/12/2013 10:50	3.96	0.687
09/12/2013 11:05	3.945	0.702
09/12/2013 11:10	3.945	0.702
09/12/2013 11:25	3.93	0.717
09/12/2013 11:36	3.93	0.717
09/12/2013 12:19	3.94	0.707

Date, time	Dip (mbd)	Dip (mAOD)
09/12/2013 12:39	3.93	0.717
09/12/2013 12:48	3.95	0.697
09/12/2013 13:08	3.945	0.702
09/12/2013 13:28	3.945	0.702
09/12/2013 14:18	3.945	0.702
09/12/2013 14:48	3.95	0.697
09/12/2013 15:18	3.95	0.697
09/12/2013 15:48	3.95	0.697
09/12/2013 16:18	3.95	0.697
09/12/2013 16:48	3.95	0.697
09/12/2013 17:48	3.95	0.697
09/12/2013 18:47	3.94	0.707
09/12/2013 19:46	3.93	0.717
09/12/2013 20:46	3.89	0.757
09/12/2013 22:47	3.94	0.707
09/12/2013 23:47	3.94	0.707
10/12/2013 00:47	3.94	0.707
10/12/2013 01:47	3.94	0.707
10/12/2013 02:47	3.94	0.707
10/12/2013 03:47	3.94	0.707
10/12/2013 04:47	3.94	0.707
10/12/2013 06:47	3.95	0.697
10/12/2013 08:47	3.96	0.687
10/12/2013 10:47	3.96	0.687
10/12/2013 12:46	3.96	0.687
10/12/2013 14:47	3.89	0.757
10/12/2013 16:47	3.9	0.747
10/12/2013 18:47	3.94	0.707
10/12/2013 20:47	3.96	0.687
10/12/2013 22:47	3.96	0.687
11/12/2013 00:47	3.95	0.697
11/12/2013 02:47	3.9	0.747
11/12/2013 04:47	3.93	0.717
11/12/2013 06:46	3.89	0.757
11/12/2013 08:46	3.87	0.777
11/12/2013 11:37	3.97	0.677
11/12/2013 11:47	3.95	0.697
11/12/2013 11:57	3.96	0.687
11/12/2013 12:05	3.94	0.707
11/12/2013 12:16	3.93	0.717
11/12/2013 12:25	3.94	0.707
11/12/2013 12:46	3.94	0.707
11/12/2013 13:06	3.93	0.717
11/12/2013 13:26	3.93	0.717
11/12/2013 13:46	3.93	0.717
11/12/2013 14:16	3.93	0.717
11/12/2013 14:46	3.93	0.717
11/12/2013 15:16	3.93	0.717
11/12/2013 15:40	3.93	0.717
11/12/2013 16:16	3.93	0.717
11/12/2013 16:45	3.93	0.717
11/12/2013 17:46	3.93	0.717
11/12/2013 18:46	3.93	0.717

Date, time	Dip (mbd)	Dip (mAOD)
11/12/2013 19:46	3.93	0.717
11/12/2013 20:45	3.93	0.717
11/12/2013 21:45	3.93	0.717
11/12/2013 22:45	3.93	0.717
11/12/2013 23:45	3.93	0.717
12/12/2013 00:45	3.92	0.727
12/12/2013 01:45	3.93	0.717
12/12/2013 02:45	3.93	0.717
12/12/2013 03:45	3.92	0.727
12/12/2013 04:45	3.92	0.727
12/12/2013 06:45	3.92	0.727
12/12/2013 08:45	3.92	0.727
12/12/2013 10:45	3.93	0.717
12/12/2013 12:27	3.93	0.717
12/12/2013 12:47	3.93	0.717
12/12/2013 13:07	3.93	0.717
12/12/2013 13:27	3.93	0.717
12/12/2013 13:47	3.93	0.717
12/12/2013 14:17	3.93	0.717
12/12/2013 14:47	3.93	0.717
12/12/2013 15:17	3.93	0.717
12/12/2013 15:47	3.93	0.717
12/12/2013 16:17	3.93	0.717
12/12/2013 16:47	3.93	0.717
12/12/2013 17:47	3.93	0.717
12/12/2013 18:47	3.92	0.727
12/12/2013 19:46	3.93	0.717
12/12/2013 20:46	3.92	0.727
12/12/2013 21:45	3.93	0.717
12/12/2013 22:45	3.93	0.717
12/12/2013 23:45	3.93	0.717
13/12/2013 00:45	3.93	0.717
13/12/2013 01:45	3.93	0.717
13/12/2013 02:45	3.92	0.727
13/12/2013 03:46	3.92	0.727
13/12/2013 04:46	3.93	0.717
13/12/2013 06:46	3.92	0.727
13/12/2013 08:46	3.91	0.737
13/12/2013 10:47	3.9	0.747

BH26 (Glacial Till)

Datum: 4.543 mAOD Top edge of cover (ground level)
- taken from the edge across from the hinge

Date, time	Dip (mbd)	Dip (mAOD)
06/11/2013 12:13	3.866	0.677
07/11/2013 12:10	3.869	0.674
08/11/2013 16:07	3.862	0.681
11/11/2013 15:16	3.905	0.638
14/11/2013 07:51	3.955	0.588
15/11/2013 08:43	4.015	0.528
18/11/2013 15:56	3.965	0.578
19/11/2013 14:40	3.95	0.593
21/11/2013 09:19	3.945	0.598
22/11/2013 09:42	3.925	0.618
25/11/2013 15:50	3.995	0.548
26/11/2013 13:34	4.02	0.523
09/12/2013 08:45	3.9	0.643
09/12/2013 11:54	3.91	0.633
09/12/2013 11:05	3.92	0.623
09/12/2013 11:34	3.91	0.633
09/12/2013 12:17	3.91	0.633
09/12/2013 12:37	3.91	0.633
09/12/2013 12:50	3.925	0.618
09/12/2013 13:10	3.93	0.613
09/12/2013 13:30	3.93	0.613
09/12/2013 14:20	3.935	0.608
09/12/2013 14:50	3.935	0.608
09/12/2013 15:20	3.935	0.608
09/12/2013 15:50	3.935	0.608
09/12/2013 16:20	3.935	0.608
09/12/2013 16:50	3.935	0.608
09/12/2013 17:50	3.935	0.608
09/12/2013 18:49	3.91	0.633
09/12/2013 19:49	3.91	0.633
09/12/2013 20:49	3.92	0.623
09/12/2013 21:49	3.92	0.623
09/12/2013 22:49	3.92	0.623
09/12/2013 23:49	3.92	0.623
10/12/2013 00:49	3.93	0.613
10/12/2013 01:49	3.92	0.623
10/12/2013 02:49	3.92	0.623
10/12/2013 03:49	3.92	0.623
10/12/2013 04:49	3.92	0.623
10/12/2013 06:49	3.92	0.623
10/12/2013 08:49	3.92	0.623
10/12/2013 10:48	3.94	0.603
10/12/2013 12:47	3.92	0.623
10/12/2013 14:48	3.46	1.083
10/12/2013 16:48	3.5	1.043
10/12/2013 18:48	3.51	1.033
10/12/2013 20:48	3.64	0.903
10/12/2013 22:48	3.56	0.983
11/12/2013 00:48	3.66	0.883
11/12/2013 02:48	3.64	0.903

Date, time	Dip (mbd)	Dip (mAOD)
11/12/2013 04:48	3.62	0.923
11/12/2013 06:48	3.65	0.893
11/12/2013 08:48	3.67	0.873
11/12/2013 11:38	3.77	0.773
11/12/2013 11:48	3.77	0.773
11/12/2013 11:58	3.77	0.773
11/12/2013 12:06	3.76	0.783
11/12/2013 12:15	3.76	0.783
11/12/2013 12:26	3.76	0.783
11/12/2013 12:46	3.77	0.773
11/12/2013 13:06	3.75	0.793
11/12/2013 13:26	3.68	0.863
11/12/2013 13:46	3.76	0.783
11/12/2013 14:17	3.76	0.783
11/12/2013 14:46	3.76	0.783
11/12/2013 15:17	3.76	0.783
11/12/2013 15:47	3.77	0.773
11/12/2013 16:16	3.77	0.773
11/12/2013 16:46	3.77	0.773
11/12/2013 17:46	3.78	0.763
11/12/2013 18:46	3.78	0.763
11/12/2013 19:46	3.79	0.753
11/12/2013 20:46	3.79	0.753
11/12/2013 21:46	3.79	0.753
11/12/2013 22:46	3.79	0.753
11/12/2013 23:46	3.79	0.753
12/12/2013 00:46	3.79	0.753
12/12/2013 01:46	3.78	0.763
12/12/2013 02:46	3.82	0.723
12/12/2013 03:46	3.79	0.753
12/12/2013 04:46	3.81	0.733
12/12/2013 06:46	3.81	0.733
12/12/2013 08:46	3.82	0.723
12/12/2013 10:45	3.89	0.653
12/12/2013 12:29	3.865	0.678
12/12/2013 12:49	3.865	0.678
12/12/2013 13:09	3.875	0.668
12/12/2013 13:29	3.87	0.673
12/12/2013 13:49	3.865	0.678
12/12/2013 14:19	3.87	0.673
12/12/2013 14:49	3.87	0.673
12/12/2013 15:19	3.87	0.673
12/12/2013 15:49	3.87	0.673
12/12/2013 16:19	3.87	0.673
12/12/2013 16:49	3.87	0.673
12/12/2013 17:49	3.87	0.673
12/12/2013 18:45	3.77	0.773
12/12/2013 19:45	3.76	0.783
12/12/2013 20:47	3.76	0.783
12/12/2013 21:46	3.76	0.783
12/12/2013 22:46	3.86	0.683
13/12/2013 00:45	3.87	0.673
13/12/2013 01:45	3.88	0.663

Date, time	Dip (mbd)	Dip (mAOD)
13/12/2013 02:46	3.86	0.683
13/12/2013 03:47	3.86	0.683
13/12/2013 04:47	3.87	0.673
13/12/2013 06:47	3.86	0.683
13/12/2013 08:47	3.86	0.683
13/12/2013 10:48	3.86	0.683

BH27 (Cohesive Alluvium)

Datum: 4.398 mAOD Top edge of cover (ground level)
- taken from the edge across from the hinge

Date, time	Dip (mbd)	Dip (mAOD)
06/11/2013 12:16	3.435	0.963
07/11/2013 12:12	3.473	0.925
08/11/2013 16:04	3.585	0.813
11/11/2013 16:46	3.756	0.642
12/11/2013 13:58	3.775	0.623
13/11/2013 08:51	3.815	0.583
14/11/2013 07:49	3.585	0.813
15/11/2013 08:41	3.755	0.643
18/11/2013 15:52	3.585	0.813
19/11/2013 14:56	3.6	0.798
21/11/2013 11:10	3.56	0.838
22/11/2013 09:25	3.7	0.698
25/11/2013 15:51	3.83	0.568
26/11/2013 13:36	3.795	0.603
04/12/2013 16:00	3.533	0.865
05/12/2013 00:00	3.41	0.988
09/12/2013 08:48	3.49	0.908
09/12/2013 10:58	3.49	0.908
09/12/2013 11:07	3.49	0.908
09/12/2013 11:10	3.49	0.908
09/12/2013 11:18	3.49	0.908
09/12/2013 11:33	3.48	0.918
09/12/2013 11:38	3.73	0.668
09/12/2013 12:15	3.49	0.908
09/12/2013 12:35	3.49	0.908
09/12/2013 12:53	3.49	0.908
09/12/2013 13:12	3.49	0.908
09/12/2013 13:32	3.495	0.903
09/12/2013 14:22	3.485	0.913
09/12/2013 14:52	3.495	0.903
09/12/2013 15:22	3.495	0.903
09/12/2013 15:52	3.485	0.913
09/12/2013 16:22	3.495	0.903
09/12/2013 16:52	3.5	0.898
09/12/2013 17:52	3.5	0.898
09/12/2013 18:48	3.5	0.898
09/12/2013 19:49	3.51	0.888
09/12/2013 20:49	3.5	0.898
09/12/2013 21:51	3.49	0.908
09/12/2013 22:51	3.5	0.898
09/12/2013 23:51	3.49	0.908

Date, time	Dip (mbd)	Dip (mAOD)
10/12/2013 00:51	3.5	0.898
10/12/2013 01:59	3.49	0.908
10/12/2013 02:51	3.5	0.898
10/12/2013 03:51	3.5	0.898
10/12/2013 04:51	3.5	0.898
10/12/2013 06:51	3.5	0.898
10/12/2013 08:49	3.51	0.888
10/12/2013 10:49	3.5	0.898
10/12/2013 12:47	3.5	0.898
10/12/2013 14:49	3.42	0.978
10/12/2013 16:48	3.41	0.988
10/12/2013 18:49	3.5	0.898
10/12/2013 20:49	3.5	0.898
10/12/2013 22:49	3.49	0.908
11/12/2013 00:49	3.5	0.898
11/12/2013 02:49	3.49	0.908
11/12/2013 04:49	3.48	0.918
11/12/2013 06:49	3.39	1.008
11/12/2013 08:48	3.4	0.998
11/12/2013 11:49	3.49	0.908
11/12/2013 11:58	3.47	0.928
11/12/2013 12:06	3.45	0.948
11/12/2013 12:16	3.45	0.948
11/12/2013 12:27	3.45	0.948
11/12/2013 12:47	3.45	0.948
11/12/2013 13:07	3.45	0.948
11/12/2013 13:27	3.44	0.958
11/12/2013 13:47	3.44	0.958
11/12/2013 14:47	3.44	0.958
11/12/2013 15:17	3.44	0.958
11/12/2013 15:47	3.44	0.958
11/12/2013 16:17	3.44	0.958
11/12/2013 16:47	3.43	0.968
11/12/2013 17:47	3.44	0.958
11/12/2013 18:47	3.48	0.918
11/12/2013 19:47	3.45	0.948
11/12/2013 20:47	3.48	0.918
11/12/2013 21:47	3.49	0.908
11/12/2013 22:47	3.45	0.948
11/12/2013 23:47	3.45	0.948
12/12/2013 00:47	3.45	0.948
12/12/2013 01:47	3.45	0.948
12/12/2013 02:47	3.44	0.958
12/12/2013 03:47	3.43	0.968
12/12/2013 04:47	3.43	0.968
12/12/2013 06:47	3.4	0.998
12/12/2013 08:47	3.38	1.018
12/12/2013 12:31	3.14	1.258
12/12/2013 12:51	3.4	0.998
12/12/2013 13:11	3.4	0.998
12/12/2013 13:31	3.395	1.003
12/12/2013 13:51	3.4	0.998
12/12/2013 14:21	3.4	0.998

Date, time	Dip (mbd)	Dip (mAOD)
12/12/2013 14:51	3.395	1.003
12/12/2013 15:21	3.395	1.003
12/12/2013 15:51	3.4	0.998
12/12/2013 16:21	3.4	0.998
12/12/2013 16:51	3.395	1.003
12/12/2013 17:51	3.39	1.008
12/12/2013 18:45	3.44	0.958
12/12/2013 19:46	3.41	0.988
12/12/2013 20:47	3.41	0.988
12/12/2013 21:47	3.41	0.988
12/12/2013 22:46	3.4	0.998
12/12/2013 23:46	3.41	0.988
13/12/2013 00:45	3.4	0.998
13/12/2013 01:45	3.41	0.988
13/12/2013 02:47	3.4	0.998
13/12/2013 03:47	3.41	0.988
13/12/2013 04:47	3.4	0.998
13/12/2013 06:47	3.37	1.028
13/12/2013 08:46	3.34	1.058
13/12/2013 10:49	3.34	1.058

BH28 (Glacial Till)

Datum: 4.476 mAOD Ground level (straight edge across top of borehole)

Date, time	Dip (mbd)	Dip (mAOD)
06/11/2013 11:50	3.66	0.816
07/11/2013 12:19	3.666	0.81
08/11/2013 15:52	3.645	0.831
11/11/2013 16:38	3.73	0.746
13/11/2013 09:33	3.8	0.676
14/11/2013 15:05	3.755	0.721
15/11/2013 09:00	3.827	0.649
15/11/2013 09:42	3.815	0.661
15/11/2013 16:07	3.585	0.891
19/11/2013 14:54	3.805	0.671
21/11/2013 09:57	3.875	0.601
22/11/2013 09:51	3.75	0.726
25/11/2013 15:55	3.835	0.641
26/11/2013 13:44	3.83	0.646
09/12/2013 08:53	3.75	0.726
09/12/2013 10:11	3.705	0.771
09/12/2013 11:18	3.72	0.756
09/12/2013 11:27	3.73	0.746
09/12/2013 11:59	3.73	0.746
09/12/2013 12:21	3.73	0.746
09/12/2013 12:40	3.73	0.746
09/12/2013 12:55	3.735	0.741
09/12/2013 13:14	3.755	0.721
09/12/2013 13:34	3.755	0.721
09/12/2013 14:24	3.755	0.721
09/12/2013 14:54	3.755	0.721
09/12/2013 15:24	3.76	0.716

Date, time	Dip (mbd)	Dip (mAOD)
09/12/2013 15:54	3.735	0.741
09/12/2013 16:24	3.735	0.741
09/12/2013 16:54	3.755	0.721
09/12/2013 17:54	3.76	0.716
09/12/2013 19:51	3.7	0.776
09/12/2013 20:50	3.75	0.726
09/12/2013 21:52	3.77	0.706
09/12/2013 22:52	3.76	0.716
09/12/2013 23:52	3.76	0.716
10/12/2013 00:52	3.76	0.716
10/12/2013 01:52	3.76	0.716
10/12/2013 02:52	3.77	0.706
10/12/2013 03:52	3.77	0.706
10/12/2013 04:52	3.76	0.716
10/12/2013 06:50	3.8	0.676
10/12/2013 08:49	3.79	0.686
10/12/2013 10:49	3.79	0.686
10/12/2013 12:48	3.79	0.686
10/12/2013 14:50	3.66	0.816
10/12/2013 16:49	3.66	0.816
10/12/2013 18:49	3.8	0.676
10/12/2013 20:49	3.79	0.686
10/12/2013 22:49	3.79	0.686
11/12/2013 00:49	3.79	0.686
11/12/2013 02:49	3.8	0.676
11/12/2013 04:49	3.8	0.676
11/12/2013 06:49	3.64	0.836
11/12/2013 08:49	3.66	0.816
11/12/2013 11:40	3.69	0.786
11/12/2013 11:50	3.8	0.676
11/12/2013 12:00	3.78	0.696
11/12/2013 12:08	3.76	0.716
11/12/2013 12:18	3.77	0.706
11/12/2013 12:28	3.77	0.706
11/12/2013 12:48	3.77	0.706
11/12/2013 13:29	3.76	0.716
11/12/2013 13:49	3.76	0.716
11/12/2013 14:18	3.76	0.716
11/12/2013 14:48	3.75	0.726
11/12/2013 15:18	3.76	0.716
11/12/2013 15:48	3.75	0.726
11/12/2013 16:18	3.75	0.726
11/12/2013 16:48	3.76	0.716
11/12/2013 17:49	3.75	0.726
11/12/2013 18:49	3.77	0.706
11/12/2013 19:49	3.79	0.686
11/12/2013 20:49	3.77	0.706
11/12/2013 21:49	3.77	0.706
11/12/2013 22:49	3.79	0.686
11/12/2013 23:49	3.77	0.706
12/12/2013 00:49	3.78	0.696
12/12/2013 02:49	3.76	0.716
12/12/2013 03:49	3.35	1.126

Date, time	Dip (mbd)	Dip (mAOD)
12/12/2013 04:49	3.76	0.716
12/12/2013 06:49	3.72	0.756
12/12/2013 08:49	3.72	0.756
12/12/2013 12:33	3.74	0.736
12/12/2013 12:53	3.74	0.736
12/12/2013 13:13	3.745	0.731
12/12/2013 13:33	3.74	0.736
12/12/2013 13:53	3.73	0.746
12/12/2013 14:23	3.73	0.746
12/12/2013 14:53	3.73	0.746
12/12/2013 15:23	3.735	0.741
12/12/2013 15:53	3.735	0.741
12/12/2013 16:23	3.735	0.741
12/12/2013 16:53	3.74	0.736
12/12/2013 17:53	3.735	0.741
12/12/2013 18:46	3.77	0.706
12/12/2013 19:47	3.75	0.726
12/12/2013 20:48	3.75	0.726
12/12/2013 21:47	3.72	0.756
12/12/2013 22:47	3.75	0.726
12/12/2013 23:47	3.74	0.736
13/12/2013 00:47	3.74	0.736
13/12/2013 01:47	3.74	0.736
13/12/2013 02:47	3.79	0.686
13/12/2013 03:48	3.79	0.686
13/12/2013 04:48	3.76	0.716
13/12/2013 06:48	3.72	0.756
13/12/2013 08:50	3.58	0.896
13/12/2013 10:40	3.68	0.796
13/12/2013 10:50	3.61	0.866

SBP04 (Cohesive Alluvium)

Datum: 3.49 mAOD Top edge of cover (ground level)
- taken from the edge across from the hinge

Date, time	Dip (mbd)	Dip (mAOD)
11/11/2013 15:40	2.335	1.155
12/11/2013 14:02	2.337	1.153
13/11/2013 08:54	2.355	1.135
14/11/2013 15:15	2.32	1.17
21/11/2013 15:34	2.29	1.2
22/11/2013 11:42	2.325	1.165
25/11/2013 16:12	2.375	1.115
26/11/2013 15:20	2.67	0.82
04/12/2013 16:02	2.355	1.135
05/12/2013 09:39	2.345	1.145
09/12/2013 10:30	2.055	1.435
09/12/2013 11:23	2.05	1.44
09/12/2013 11:40	2.075	1.415
09/12/2013 12:12	2.06	1.43
09/12/2013 12:30	2.06	1.43
09/12/2013 12:40	2.07	1.42
09/12/2013 13:12	2.06	1.43

Date, time	Dip (mbd)	Dip (mAOD)
09/12/2013 15:39	2.06	1.43
09/12/2013 16:47	2.06	1.43
09/12/2013 17:52	2.065	1.425
10/12/2013 08:54	2.09	1.4
10/12/2013 10:51	2.095	1.395
10/12/2013 13:22	2.095	1.395
10/12/2013 15:09	2.1	1.39
10/12/2013 17:16	2.11	1.38
11/12/2013 10:19	2.12	1.37
11/12/2013 14:57	2.12	1.37
12/12/2013 09:53	2.13	1.36
12/12/2013 13:15	2.125	1.365
12/12/2013 14:30	2.135	1.355
12/12/2013 15:30	2.14	1.35
12/12/2013 16:30	2.14	1.35
13/12/2013 10:37	2.14	1.35

Annex 3 – Water Quality Sample Results

Multiplier: 1 x"<" A63 Castle Street - PUMPING TEST

Screening	Substance	Units																								
	Sample Description		DOCKS	DOCKS	DOCKS	DOCKS	DOCKS	DOCKS	DOCKS	DOCKS	DOCKS	DOCKS	DOCKS	DOCKS	DOCKS	DOCKS								Chalk	Chalk	Chalk
																								PARTIAL ROUN	RTIAL ROUN	ULL ROUND
	Date		05/09/2013	20/08/2013	23/10/2013	04/12/2013	09/12/2013	11/12/2013	17/12/2013	05/09/2013	20/08/2013	23/10/2013	04/12/2013	09/12/2013	11/12/2013	17/12/2013	27/11/2013	27/11/2013	06/12/2013	09/12/2013	10/12/2013	11/12/2013	18/12/2013	05/09/2013	16/09/2013	03/10/2013
	Sample ID		SW 2	SW 2	SW 2	SW 2	SW 2	SW 2	SW 2	SW 3	SW 3	SW 3	SW 3	SW 3	SW 3	SW 3	LDBH_01 (A)	LDBH_01 (B)	LDBH_01	LDBH_01	LDBH_01	LDBH_01	LDBH_01	BH24	BH24	BH24
	Depth	m																								
Screening	Substance	Units																								
-																										
-	pH		7.7	7.7	7.7	7.8	7.8	7.7	7.7	7.9	8	7.7	7.9	7.8	7.7	7.7	7.2	7.1	7	7.1	7.2	7	9.8 *	7.4	7.6	7.2
-																										
50	Arsenic	ug/l	270	32	110	220	210	220	48	260	34	93	260	200	210	59	130	110		320	96	310	25*	75	37	68
2000	Boron	ug/l	1800	1800	1900	1300	1400	1500	1500	1800	1800	1900	1200	1400	1500	1400	1900	1900		2100	2000	2100	160 *	330	380	2000
0.15	Cadmium	ug/l	0.15	0.15	0.13	0.11	0.11	0.12	0.08	0.12	0.15	0.1	0.12	0.1	0.11	0.12	0.15	0.05		0.06	0.03	0.04	0.89	0.04	0.11	0.07
-	Chromium (total)	ug/l	18	2	12	18	16	18	10	16	2	9	17	16	17	10	12	12		23	11	23	1	16	19	16
4.7	Chromium (III)	ug/l	13	<3	12	18	16	18	10	12	<3	9	17	16	17	10	12	12		23	11	23	<3	13	<3	16
3.4	Chromium (VI)		5	<3	<3	<3	<3	<3	<3	4	<3	<3	<3	<3	<3	<3	<3	<3		<3	<3	<3	<3	3	17	<3
10	Copper	ug/l	67	170	12000 *	27	35	42	170	67	250	10000 *	29	38	42	200	94	160		74	200	49	160	12	14	520
7.2	Lead	ug/l	4.5	<0.3	<0.3	2.9	<0.3	0.3	0.4	0.5	<0.3	<0.3	2.5	<0.3	<0.3	0.5	<0.3	<0.3		<0.3	<0.3	<0.3	<0.3	1.1	17	0.5
0.05	Mercury	ug/l	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	0.1	<0.05		<0.05	<0.05	<0.05	0.08	<0.05	0.08	<0.05
20	Nickel	ug/l	13	6	18	10	14	16	10	13	6	15	10	14	16	12	32	31		47	31	44	5 *	16	14	57
-	Selenium	ug/l	120	67	<0.5	140	150	170	<0.5	95	75	<0.5	130	150	160	<0.5	<0.5	<0.5		210	<0.5	200	85	54	57	<0.5
75	Zinc	ug/l	41	36	26	33	39	47	28	35	36	21	30	45	49	31	51	40		78	26	57	2	32	22	13
-																										
-	Calcium	ug/l	160000	230000	180000	140000	160000	160000	160000	160000	220000	180000	140000	160000	160000	160000	260000	260000	270000	210000	210000	210000	170000	130000	170000	200000
-	Magnesium	ug/l	520000	540000	590000	360000	380000	390000	430000	540000	520000	610000	350000	390000	400000	430000	580000	580000	610000	590000	490000	580000	94000	99000	130000	530000
-	Potassium	ug/l	210000	230000	230000	140000	140000	150000	180000	220000	220000	240000	140000	150000	150000	170000	230000	230000	260000	220000	200000	230000	820000	62000	75000	220000
-	Sodium	ug/l	3900000	3500000	4600000	2500000	2200000	2600000	3700000	4000000	3000000	4700000	2400000	2200000	2100000	3600000	4400000	4200000	4100000	3300000	3400000	2200000	2600000	860000	1100000	3900000
1000	Iron (dissolved)	ug/l			220				76			200				84	<10	33		<10	<10	200	230			250
1000	Iron (total)	ug/l			370				330			400				500	9700	6800		6600	6100	6300	4200			4600
-	Manganese (dissolved)	ug/l			49				54			27				51	420	420		400	380	420	11 *			460
-	Manganese (total)	mg/l			66				62			72				66	430	440		410	400	430	110			470
-																										
-	Nitrate	mg/l	1.7	3.8	2.3	5.8	5.1	5.5	4.9	1.8	4.1	3.4	5.8	5.4	5.5	5	<0.5	<0.5	<0.5	<0.5	0.9	<0.5	<0.5	<0.5	5	<0.5
-	Nitrite	mg/l	0.2	0.2	<0.1	0.1	<0.1	<0.1	<0.1	0.2	0.2	<0.1	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	5.2	0.2
-	TON	mg/l	0.4	0.9	0.5	1.3	1.2	1.2	1.1	0.4	1	0.8	1.3	1.2	1.2	1.1	<0.1	<0.1	<0.1	<0.1	0.2	<0.1	<0.1	<0.1	2.7	0.2
400	Sulphate	mg/l	1300	1300	1300	850	890	880	970	1300	1300	1300	860	880	720	980	1200	1200		1200	180 *	1300	930	130	130	1300
-	Sulphide	mg/l	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05		<0.05	0.05	<0.05	<0.05	<0.05	<0.05	<0.05
-	Sulphur	mg/l	450	410	430	300	340	350	370	450	400	430	290	340	350	370	420	420		480	440	490	380	56	74	420
250	Chloride	mg/l	9700	9100	9800	6000	6500	6300	6800	9700	9000	10000	6000	6600	5300	6900	10000	9500	10000	10000	79 *	9300	5500	1700	2100	11000
0.2	Ammoniacal Nitrogen as N	mg/l	0.57	0.6	0.55	0.5	0.31	0.63	0.69	0.52	0.55	0.65	0.45	0.31	0.48	1.6	1.2	1.2		1.9	0.32	1.7	23	14	5.5	1.1
0.26	Ammoniacal Nitrogen as NH4	mg/l	0.73	0.76	0.71	0.65	0.4	0.81	0.88	0.67	0.7	0.83	0.57	0.4	0.61	2.1	1.6	1.5		2.4	0.39	2.2	30	17	7.1	1.4
-	Total Suspended Solids	mg/l	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	14	74	85	86	62	100	72	43	210	14	<10
-	Electrical Conductivity	uS/cm	27000	2500	26000	22000	17000	18000	21000	27000	2500	26000	20000	17000	18000	21000	26000	26000	24000	24000	25000	24000	16000	7200	9100	25000
-	Total Alkalinity as CaCO3	mg/l	130	130	160				110	120	120	160				120	200	190	170	160	350	170	250			
-																										
7.7	Phenols (total)	ug/l	<0.5	<0.5	0.6	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	1.1	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5		<0.5	1	<0.5	<0.5	0.7	1.2	<0.5
1	Cyanide (total)	ug/l	<10	<10	<10	<10	10	<10	26	<10	<10	<10	<10	<10	<10	<10	36	71		20	<10	<10	<10	<10	<10	26
1	Cyanide (free)	ug/l	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	24	31		<10	<10	<10	<10	<10	<10	<10
-																										
-	Acenaphthene	ug/l	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.02	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01	<0.01	<0.01	0.03	<0.01
-	Acenaphthylene	ug/l	0.06	<0.01	<0.01	0.02	<0.01	<0.01	<0.01	0.02	<0.01	<0.02	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01	<0.01	0.05	0.02	<0.01
0.1	Anthracene	ug/l	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.02	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01	<0.01	<0.01	0.03	<0.01
-	Benzo(a)anthracene	ug/l	<0.01	<0.01	<0.01	<0.01	0.02	<0.01	<0.01	<0.01	<0.01	<0.02	<0.01	<0.01	0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01	<0.01	<0.01	0.04	<0.01
0.05	Benzo(a)pyrene	ug/l	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.02	<0.01	<0.01	<0.01	0.01	<0.01	<0.01		<0.01	<0.01	<0.01	<0.01	<0.01	0.03	<0.01
-	Benzo(b)fluoranthene	ug/l	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.02	<0.01	<0.01	<0.01	<0.01	0.02	<0.01		<0.01	<0.01	<0.01	<0.01	<0.01	0.03	<0.01
-	Benzo(ghi)perylene	ug/l	<0.10	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.10	<0.10	<0.02	<0.01	<0.01	<0.01	<0.01	0.01	<0.01		<0.01	<0.01	<0.01	<0.01	<0.01	0.03	<0.01
-	Benzo(k)fluoranthene	ug/l	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.02	<0.01	<0.01	<0.01	<0.01	0.01	<0.01		<0.01	<0.01	<0.01	<0.01	<0.01	0.03	<0.01
-	Chrysene	ug/l	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.02	<0.01	<0.01	<0.01	<0.01	0.01	<0.01		<0.01	<0.01	<0.01	<0.01	<0.01	0.03	<0.01
-	Dibenzo(ah)anthracene	ug/l	<0.10	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.10	<0.10	<0.02	<0.01	<0.01	<0.01	<0.01	0.02	<0.01		<0.01	<0.01	<0.01	<0.01	<0.01	0.03	<0.01
0.1	Fluoranthene	ug/l	<0.01	<0.01	<0.01	0.01	0.02	<0.01	<0.01	<0.01	<0.01	<0.02	<0.01	0.02	0.02	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01	<0.01	<0.01	0.1	<0.01
-	Fluorene	ug/l	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.02	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01	<0.01	<0.01	0.03	<0.01
-	Indeno(1,2,3cd)pyrene	ug/l	<0.10	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.10	<0.10	<0.02	<0.01	<0.01	<0.01	<0.01	0.02	<0.01		<0.01	<0.01	<0.01	<0.01	<0.01	0.03	<0.01
2.4	Naphthalene	ug/l	0.11	<0.01	<0.01	0.3	<0.01	<0.01	0.06	0.21	<0.01	<0.02	0.27	<0.01	<0.01	0.03	0.05	0.02		<0.01	0.27	<0.01	<0.01	0.04	<0.01	0.34
-	Phenanthrene	ug/l	<0.01	<0.01	<0.01	0.03	0.02	<0.01	<0.01	<0.01	<0.01	<0.02	<0.01	0.01	<0.01	<0.01	0.01	0.02		<0.01	<0.01	<0.01	<0.01	<0.01	0.04	<0.01
-	Pyrene	ug/l	<0.01	<0.01	<0.01	<0.01	0.02	<0.01	<0.01	<0.01	<0.01	<0.02	<0.01	0.02	0.02	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01	<0.01	<0.01	0.12	<0.01
-	Polyaromatic Hydrocarbons (Total)	ug/l	0.17	<0.01	<0.01	0.36	0.08	<0.01	0.06	0.23	<0.10	<0.02	0.27	0.05	0.05	0.04	0.15	0.04		<0.01	0.27	<0.01	<0.01	0.09	0.62	0.34
-	Sum of 4No. PAHs	ug/l	<0.21	<0.04	<0.04	<0.04	<0.04	<0.04	<0.04	<0.21	<0.21	<0.08	<0.04	<0.04	<0.04	<0.04	0.06	<0.04		<0.04	<0.04	<0.04	<0.04	<0.04	0.12	<0.04
0.03	Sum of benzo(b) and benzo(k)fluoranthene	ug/l	<0.02	<0.02	<0.02	<0.02	<0.02	<0.02	<0.02	<0.02	<0.02	<0.04	<0.02	<0.02	<0.02	<0.02	0.03	<0.02		<0.02	<0.02	<0.02	<0.02	<0.02	0.06	<0.02
0.002	Sum of indeno(123cd)pyrene and benzo(ghi)perylene	ug/l	<0.2	<0.02	<0.02	<0.02	<0.02	<0.02	<0.02	<0.2	<0.2	<0.04	<0.02	<0.02	<0.02	<0.02	0.03	<0.02		<0.02	<0.02	<0.02	<0.02	<0.02	0.06	<0.02
-		ug/l																								
-	TPH (C8-C10)	ug/l	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10		<10	<10	<10	<10	<10	<10	<10
-	TPH (C10-C12)	ug/l	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10		<10	<10	<10	<10	<10	<10	<10
-	TPH (C12-C16)	ug/l	<10	<10	23	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10		<10	<10	<10	100 *	11	11	<10
-	TPH (C16-C21)	ug/l	<10	<10	24	<10	<10	<10	<10	<10	<10	29	<10	<10	<10	<10	<10	<10		<10	<10	<10	20 *	32	21	<10
-	TPH (C21-C35)	ug/l	<10	<10	17	<10	<10	<10	<10	<10	<10	28	<10	<10	<10	<10	<10	<10		<10	<10	<10	<10	160	58	33
-	TPH (C35-C40)	ug/l	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10		<10	<10	<10	<10	21	<10	<10
-	Total TPH (C8-C40)	ug/l	<10	<10	60	<10	<10	<10	<10	<10	<10	60	<10	<10	<10	<10	<10	<10		<10	<10	<10	120 *	220	90	30
-																										
-	Total Organic Carbon (TOC)	mg/l															4	5		17	5	6				
-																										
10	Benzene	ug/l																					<1			
20	Ethylbenzene	ug/l																					<1			
50	Toluene	ug/l																					<1			
-	Meta/Para-Xylene	ug/l																					<1			
-	Ortho-Xylene	ug/l																					<1			
-	Methyl tert-Butyl Ether	ug/l																					<1			
30	Sum of Xylenes	ug/l																					<2			
-																										
-	Notes: * Surious Result																									

1							
Sample Description	**Chalk**	**Chalk**	**Chalk**	**Chalk**	**Chalk**	**Chalk**	**Chalk**
	ULL ROUND	**RTIAL ROUN**	**ULL ROUND**	**RTIAL ROUN**	**ULL ROUND**	**RTIAL ROUN**	**ULL ROUND**
Date	**24/10/2013**	**28/11/2013**	**17/12/2013**	**16/09/2013**	**24/10/2013**	**28/11/2013**	**17/12/2013**
Sample ID	**BH24**	**BH24**	**BH24**	**BH29**	**BH29**	**BH29**	**BH29**
Depth							
Substance							
pH	7.1	7.1	7	**11.9 ***	7	7.1	7.6
Arsenic	**170**	**78**	29	36	**170**	**76**	17
Boron	1900	2000	1600	63	1800	2000	850
Cadmium	0.04	0.03	0.02	0.15	0.03	0.04	<0.02
Chromium (total)	11	10	2	3	10	13	51
Chromium (III)	**11**	**10**	<3	3	**10**	**13**	**51**
Chromium (VI)	<3	<3	<3	<3	<3	<3	<3
Copper	**610**	**120**	**450**	**50**	**4500 ***	**250**	**35**
Lead	<0.3	<0.3	<0.3	**7.9**	<0.3	<0.3	0.7
Mercury	<0.05	<0.05	**0.28**	**0.11**	<0.05	<0.05	<0.05
Nickel	**42**	**32**	**24**	**39**	**36**	**29**	14
Selenium	<0.5	<0.5	<0.5	94	<0.5	<0.5	8.8
Zinc	8	7	4	3	8	9	13
Calcium	200000	240000	260000	240000	190000	240000	100000
Magnesium	590000	520000	590000	<100 *	570000	510000	52000
Potassium	220000	210000	250000	78000	210000	200000	82000
Sodium	4400000	3800000	4300000	1500000	4500000	3400000	330000
Iron (dissolved)	220	150	310		210	150	95
Iron (total)	**4400**	**12000**	**24000**		**5500**	**11000**	**7100**
Manganese (dissolved)	440	470	470		510	500	510
Manganese (total)	460	500	510		510	560	580
Nitrate	<0.5	<0.5	<0.5	10	<0.5	<0.5	<0.5
Nitrite	<0.1	<0.1	<0.1	0.8	<0.1	<0.1	<0.1
TON	<0.1	<0.1	<0.1	2.6	<0.1	<0.1	<0.1
Sulphate	**1000**	**1200**	**1300**	320	**990**	**1100**	**1100**
Sulphide	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Sulphur	420	480	500	120	410	450	120
Chloride	**7100**	**9800**	**9700**	**3200**	**7300**	**8800**	**8800**
Ammoniacal Nitrogen as N	**0.96**	**1.5**	**1.1**	**18**	**2.2**	**1.5**	**3.7**
Ammoniacal Nitrogen as NH4	**1.2**	**2**	**1.4**	**23**	**2.9**	**1.9**	**4.7**
Total Suspended Solids	33	56	74	52	41	130	28
Electrical Conductivity	26000	26000	24000	7100	26000	25000	27000
Total Alkalinity as CaCO3		180				220	
Phenols (total)	0.6	<0.5	<0.5	2.4	0.5	<0.5	0.9
Cyanide (total)	**<10**	**17**	**<10**	**<10**	**<10**	**48**	**15**
Cyanide (free)	**<10**	**18**	**<10**	**<10**	**<10**	**25**	**<10**
Acenaphthene	<0.01	<0.01	<0.01	<0.01	<0.01	0.02	<0.01
Acenaphthylene	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01
Anthracene	<0.01	<0.01	<0.01	0.02	<0.01	<0.01	<0.01
Benzo(a)anthracene	<0.01	<0.01	<0.01	0.03	<0.01	<0.01	<0.01
Benzo(a)pyrene	<0.01	<0.01	<0.01	0.02	<0.01	<0.01	0.02
Benzo(b)fluoranthene	<0.01	<0.01	<0.01	0.02	<0.01	<0.01	0.01
Benzo(ghi)perylene	<0.01	<0.01	<0.01	0.02	<0.01	<0.01	<0.01
Benzo(k)fluoranthene	<0.01	<0.01	<0.01	0.02	<0.01	<0.01	<0.01
Chrysene	<0.01	<0.01	<0.01	0.03	<0.01	<0.01	<0.01
Dibenzo(ah)anthracene	<0.01	<0.01	<0.01	0.02	<0.01	<0.01	<0.01
Fluoranthene	<0.01	<0.01	<0.01	**0.17**	<0.01	0.02	<0.01
Fluorene	<0.01	<0.01	<0.01	0.03	<0.01	<0.01	<0.01
Indeno(1,2,3cd)pyrene	<0.01	<0.01	<0.01	0.02	<0.01	<0.01	0.02
Naphthalene	0.03	0.02	<0.01	<0.01	0.02	<0.01	0.03
Phenanthrene	<0.01	<0.01	<0.01	0.12	<0.01	0.02	<0.01
Pyrene	<0.01	<0.01	<0.01	0.12	<0.01	0.01	<0.01
Polyaromatic Hydrocarbons (Total)	0.03	0.02	<0.01	0.64	0.02	0.07	0.08
Sum of 4No. PAHs	<0.04	<0.04	<0.04	0.08	<0.04	<0.04	<0.05
Sum of benzo(b) and benzo(k)fluoranthene	<0.02	<0.02	<0.02	**0.04**	<0.02	<0.02	<0.02
Sum of indeno(123cd)pyrene and benzo(ghi)perylene	**<0.02**	**<0.02**	**<0.02**	**0.04**	**<0.02**	**<0.02**	**<0.03**
TPH (C8-C10)		<10		<10		<10	<10
TPH (C10-C12)		<10		<10		<10	<10
TPH (C12-C16)		<10		<10		<10	<10
TPH (C16-C21)		<10		11		<10	<10
TPH (C21-C35)		<10		20		<10	<10
TPH (C35-C40)		<10		<10		<10	<10
Total TPH (C8-C40)		<10		30		<10	<10
Total Organic Carbon (TOC)							
Benzene							
Ethylbenzene							
Toluene							
Meta/Para-Xylene							
Ortho-Xylene							
Methyl tert-Butyl Ether							
Sum of Xylenes							

Notes: * Surious Result

InSitu Water Quality Analysis

Date & Approx time	Alkalinity (ppm)	pH	Conductivity (pS/cm)	RDO (mg/l)	ORP (mv)	Temp
09/12/2013 12:00	219	7.26	29.69	5.21	-115.1	15.2
09/12/2013 15:30	212	7.29	29.67	5.31	-110.4	12.425
09/12/2013 20:00	209	7.31	29.17	2.92	-107.7	15.675
10/12/2013 07:45	212	7.31	29.71	1.58	-122.8	14
10/12/2013 13:50	211	7.31	29.63	3.06	-116.6	14.05
10/12/2013 18:30	213	7.29	29.09	3.55	-131.4	14.15
11/12/2013 09:40	223	7.28	28.84	1.85	-51.7	13.175
11/12/2013 13:30	215	7.28	28.84	1.93	-115.2	14.075
11/12/2013 19:00	221	7.21	28.4	1.87	-118.2	13.95
12/12/2013 07:30	210	7.3	29.21	2.94	-119.2	13.725

Annex 4 – Photos

Photo 1: LDBH01 pumping test discharge point at Railway Dock, 5 Dec 13 (prior to pumping test)

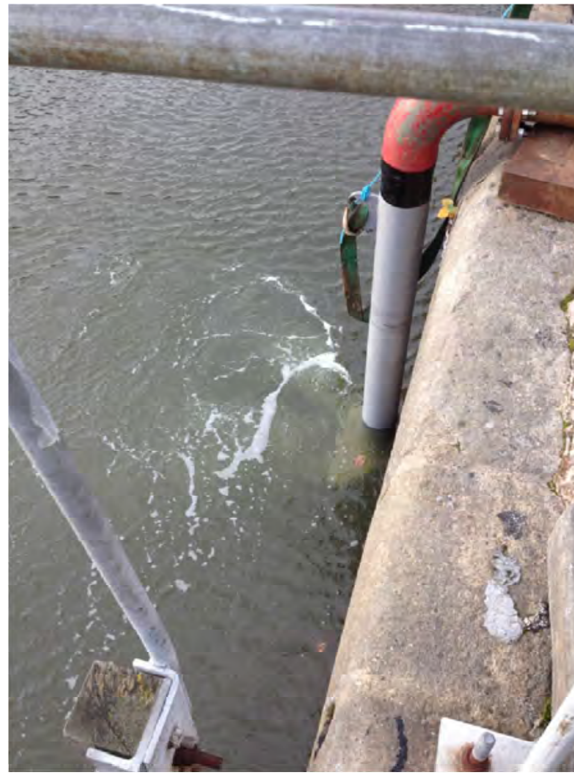


Photo 2: Discharge point, 5 Dec 13, during calibration testing

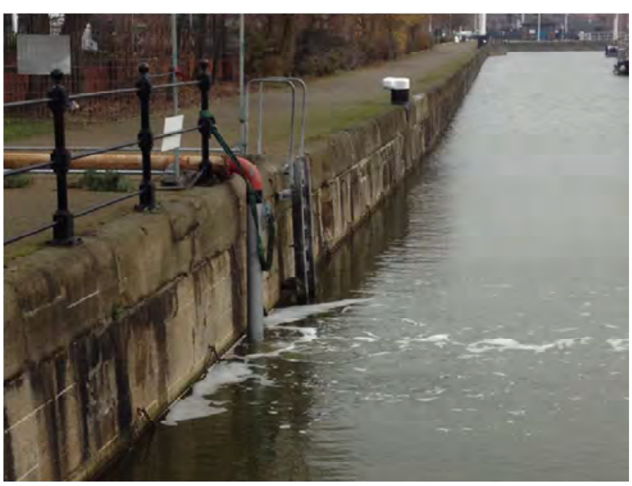


Photo 3: Discharge point, 12 Dec 13, during pumping test

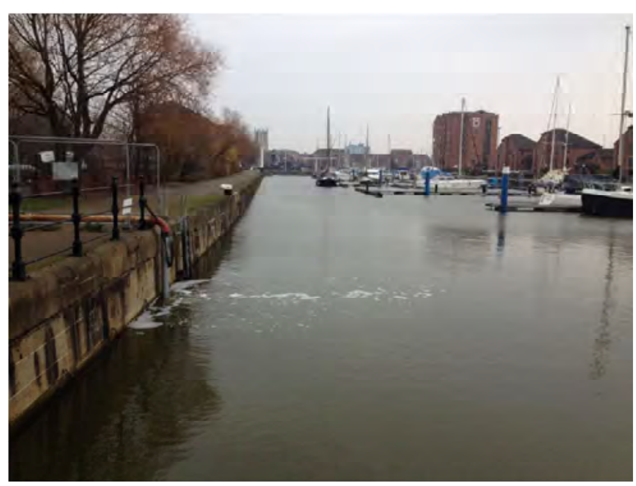


Photo 4: Discharge point, 12 Dec 13, showing wider view of Railway Dock

Photo 5: LDBH02 headworks

Photo 6: LDBH01 headworks

Photo 7: LDBH02 Chamber Cover, looking NW towards Mytongate Junction

Photo 8: LDBH01 Chamber Cover, looking N towards Mytongate Junction

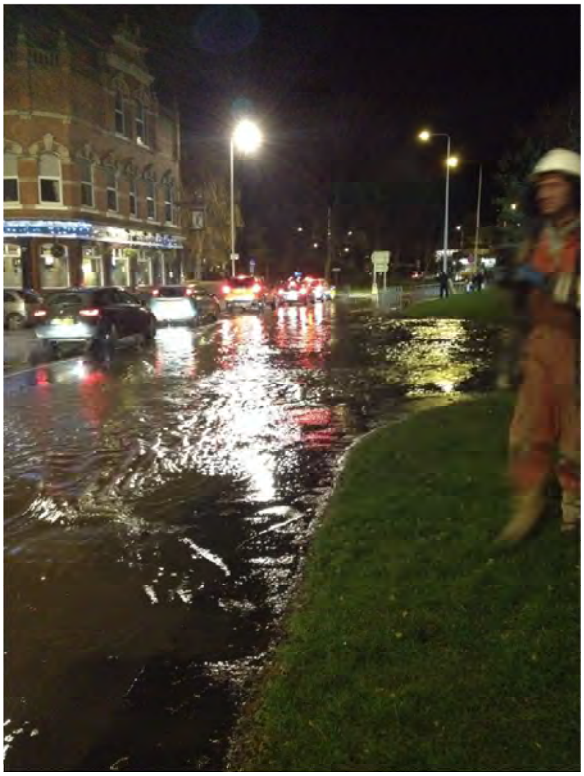


Photo 9: Flooding caused by tidal surge, looking N along Commercial Road

Photo 10: Flooding caused by tidal surge, looking S along Commercial Road

Photo 11: Flooding caused by tidal surge, looking W across Commercial Road

A63 Castle Street Improvements, Hull
Environmental Statement

Volume 3 Appendix 11.6
ROAD DRAINAGE AND THE WATER ENVIRONMENT – GROUNDWATER MODELLING REPORT

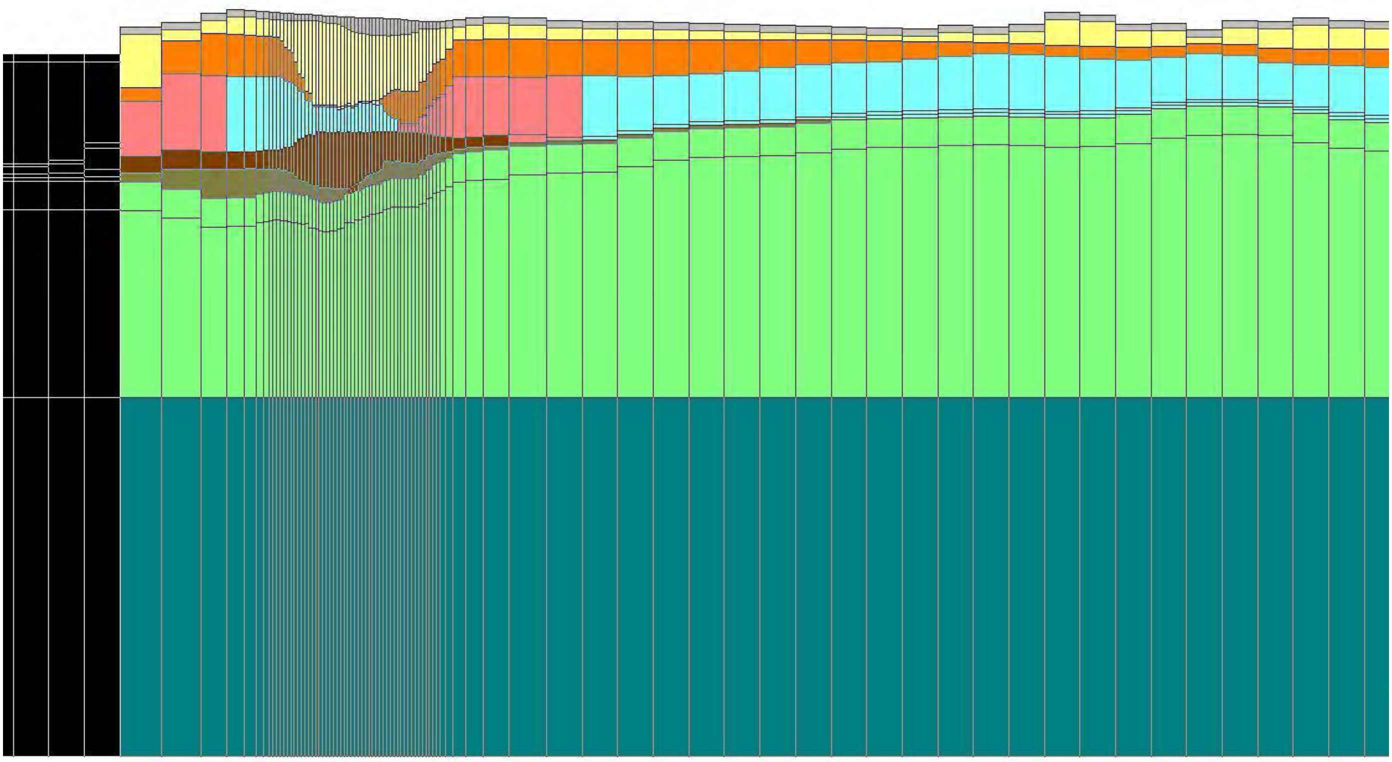
TR010016/APP/6.3
HE514508-MMSJV-EWE-S0-RP-LE-000002
31 July 2018

A63 Castle Street, Hull: Groundwater Modelling

Final Report

Document reference: 1168-10-223-RE-003-PD1

August 2014



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Revision History

Revision Ref / Date Issued	Amendments	Issued to
Draft / 19th February 2014		Harriet Carlyle
Revised Draft / 6th March 2014		Harriet Carlyle
Final / 20th August 2014		Harriet Carlyle

Contract

This report describes work commissioned by Harriet Carlyle, on behalf of Mott MacDonald Grontmij Joint Venture (MMG JV), by an e-mail dated 15th May 2013 and by a later e-mail dated 20th January 2014. MMG JVMMG JV's representatives for the contract were Harriet Carlyle and Ellen Spencer. Sam Bishop of JBA Consulting carried out this work.

Prepared by ...[redacted].........Samuel Bishop BSc MSc PhD FGS CGeol

Chartered Senior Analyst (Hydrogeologist)

Reviewed by ....[redacted]..........Susan Wagstaff MA MSc C.Geol EurGeol FGS

Technical Director (Hydrogeology)

Purpose

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JBA Consulting has no liability regarding the use of this report except to MMG JV.

Acknowledgements

JBA would like to thank James Senior and JP Camus of the Environment Agency for supplying data and for discussing the modelling approach.

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JBA is aiming to reduce its per capita carbon emissions.

Executive Summary

A road improvement scheme is proposed for the A63 in southern Hull (the Project). The proposal is for the road to be lowered into a cutting (referred to in this report as the scheme). From a hydrogeological perspective the most important features of the scheme are:

- An excavated road cutting (about 600 m long and up to about 10 m deep before laying of the road construction) with secant pile walls.
- The use of tension piles (down into the Chalk bedrock or into cohesive superficial deposits) to secure the road deck and prevent flotation.
- The proximity of the scheme to the Humber Estuary, giving tidal fluctuations in groundwater head. These fluctuations are most strongly expressed in the Chalk, but are also present in the overlying superficial deposits.
- The existence of an ongoing groundwater monitoring programme being undertaken by MMG JV.

This report presents a numerical groundwater flow model developed to predict the likely impact of the scheme on groundwater levels and flows. The model is a nine-layer finite difference model developed using MODFLOW. It represents the Chalk aquifer and the overlying superficial deposits, the latter consisting of interbedded aquifer and aquitard layers. The model has been calibrated using observed groundwater levels.

The modelling suggests that the proposed scheme will modify local groundwater levels within the superficial deposits. Steady-state simulations predict a lowering of average groundwater levels in the vicinity of the scheme. This lowering is predicted to be greater during the construction phase than during the operation phase, and much greater within the secant pile walls than outside. Maximum drawdown is predicted in the central part of the scheme, within the walls. The zone of predicted significant drawdown extends further southwards than northwards, reflecting a slight "damming" of groundwater flow by the cutting. However, this effect reflects a regional hydraulic gradient that is a function of the model boundary heads rather than measured heads. Monitoring data collected from the Project construction footprint (which is small compared to the modelled area, linear and orientated perpendicular to the modelled hydraulic gradient) do not provide evidence of a consistent regional hydraulic gradient, and the picture is complicated by tidal fluctuations. The main quantitative predictions from the steady-state model (which represents average groundwater levels) are:

- Construction phase:
  - Maximum drawdown of 7.2 m below current groundwater levels in the central part of the scheme, inside the secant pile walls
  - Drawdown immediately outside the secant pile walls less than 0.6 m.
  - Inflow of groundwater to open-based excavation = 13.4 m^3/d.
- Operation phase:
  - Maximum drawdown of 4.8 m in the central part of the scheme, inside the secant pile walls.
  - Drawdowns immediately outside the secant pile walls less than 0.4 m.
  - Inflow of groundwater to road drainage = 7 m^3/d.

Transient simulations (including tidal fluctuations) reveal a similar picture, with the scheme causing small changes in groundwater level within the superficial deposits in the vicinity of the scheme (although some small rises in groundwater level are predicted up-gradient of the scheme). No significant impact on groundwater heads or flows is predicted for the Chalk Aquifer. In particular, the tension piles are not considered likely to have a significant impact on groundwater flow (regardless of whether piling is into the Chalk or into cohesive superficial deposits).

The model is a simplification of a complex natural system, and is subject to considerable uncertainty. The limitations of the model should be borne in mind when using the results.

Contents

List of Figures

List of Tables

Abbreviations

BFI.... Baseflow Index
BGS.... British Geological Survey
FEH .... Flood Estimation Handbook
JBA.... Jeremy Benn Associates
mAOD.... metres above Ordnance Datum
MODFLOW .... Modular 3D Finite-Difference Ground-Water Flow Model
OS .... Ordnance Survey
SAAR.... Standard Average Annual Rainfall

1 Introduction

1.1 Background

The A63, Castle Street, Hull is located to the south of Hull city centre. The current road is a dual carriageway that provides a vital route link between the M62 motorway, the Humber Bridge and A15 to the west of the city, and the Port of Humber to the east of the city. The route is reputed to be one of the busiest sections of road within East Yorkshire, where the current daily traffic flow along Castle Street is in excess of 54,000 annual average daily traffic (AADT). The route currently experiences congestion, particularly around the Mytongate junction, due to the traffic signals and high proportion of heavy goods vehicles.

The Highways Agency is proposing to relieve congestion and provide better access to the Port of Hull by improving the A63 Castle Street between the St James Street/ Porter Street junctions and the Market Place/Queen Street junctions.

The preferred option comprises a grade separated junction with the alignment of the A63 lowered approximately 7 m below ground level. To provide the grade separated geometry in the congested urban environment of Hull, an embedded retaining wall solution is proposed. A cast in-situ concrete slab is to be constructed at the maximum excavation depth, acting as the permanent prop and providing the foundation to the new carriageway. Tension piles are required beneath the slab to counteract the uplift pressures induced by the shallow groundwater conditions across the scheme. In addition to the grade separated junction, several structures including the Mytongate Bridge and two footbridges are to be constructed as part of the scheme.

The scheme will involve a number of activities, including dewatering and piling, which have the potential to affect groundwater levels and flows. From a hydrogeological perspective the most important features of the scheme are:

- An excavated road cutting (about 600 m long) with secant pile walls.
- The use of tension piles (down into the Chalk bedrock or potentially into cohesive superficial deposits) to secure the road deck and prevent flotation.
- The proximity of the scheme to the Humber Estuary, giving tidal fluctuations in groundwater head (most strongly expressed in the Chalk, but also present in the overlying superficial deposits).
- The existence of an ongoing groundwater monitoring programme being undertaken by MMG JV.

MMG JV requires a groundwater model to inform the Environmental Statement (ES) for the scheme. The model will be used to predict the likely impacts of the scheme on groundwater levels and flows. There is particular concern about any drawdown in groundwater levels, as ground conditions mean that nearby buildings are potentially vulnerable to settlement damage.

1.2 Project Brief

On 15th May 2013, MMG JV commissioned JBA to assist MMG JV staff with the construction of a numerical groundwater flow model using the United States Geological Survey (USGS) code MODFLOW with the Groundwater Vistas v6 interface (McDonald and Harbaugh, 1988; ESI, 2011). JBA's original brief was to provide MMG JV's hydrogeologists with advice on numerical modelling and to help with using the software. MMG JV began to construct a regional model as initially it was thought that dewatering of the Chalk bedrock aquifer might be required, and that any impacts on groundwater could therefore be widespread. However, as site investigation and groundwater monitoring data became available the conceptual understanding of the site changed considerably. The design of the scheme also evolved, and dewatering of the Chalk was no longer proposed. As a result of these changes the modelling brief was adjusted.

In late 2013 MMG JV asked JBA to take on all of the remaining modelling work, rather than just providing an advisory and reviewing role. The groundwater modelling needed to address the following questions:

1. How much groundwater is likely to seep into the excavation (i) during construction (open excavation supported by secant pile walls) and (ii) during operation (when the road deck and tension piles are in place)?

2. What is the likely impact of the tension piles on groundwater levels and flows within the Chalk? To what extent will the piled ground act as a barrier to groundwater flow?

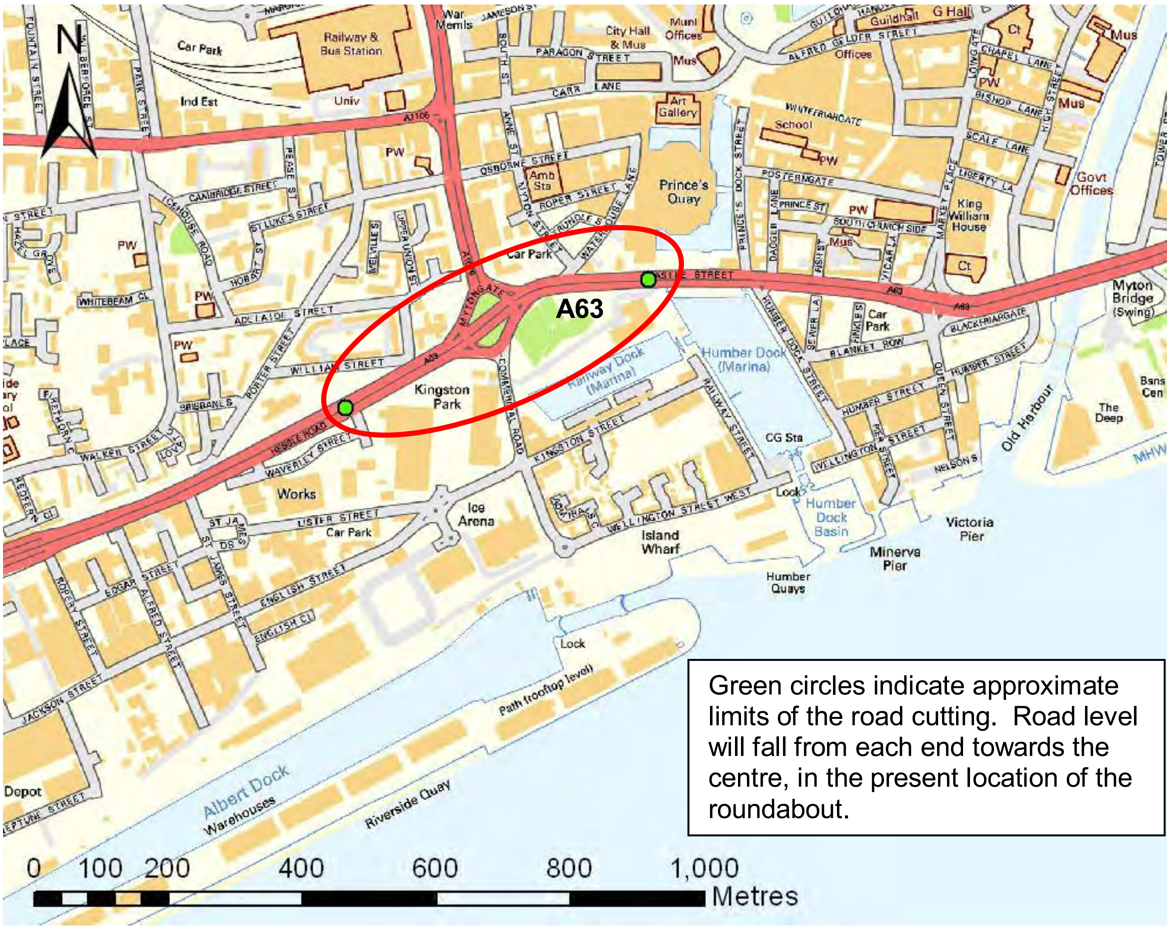
A third potential issue, whether the structure had the potential to act as a barrier to groundwater flow in the superficial deposits, was also to be investigated using the model. However, the groundwater monitoring programme undertaken by MMG JV had not identified an obvious hydraulic gradient across the scheme in either the superficial deposits or the Chalk, other than that arising from tidal fluctuations.

Ultimately, the model needed to be transient (time-variant) as it would be necessary to represent tidal fluctuations in groundwater level. The modelled area needed to be sufficiently large that the model could later be used (if necessary) to address hydrogeological questions relating to a burial ground that partially lies within the scheme footprint. Additionally, the model needed to be flexible enough (or fairly easily modified) so that it could be used to represent the hydrogeological effects of ground treatment, should this be required to stabilise the ground.

In August 2014 MMG JV commissioned JBA to undertake some additional assessment work (including simple numerical modelling) to examine the potential hydrogeological impact of piling into a cohesive superficial deposits layer rather than into the Chalk bedrock. The second question above was therefore expanded to include piling into the superficial deposits as an alternative to piling into the bedrock. This alternative proposal involved the deepest piles partially penetrating a layer of cohesive glaciolacustrine deposits.

Following an instruction from MMG JV, ground treatment (jet grouting and soil mixing) was not represented explicitly in the numerical modelling work as the hydraulic properties of the treated soil were considered to lie within the envelope of existing natural variation within the ground layers involved.

Figure 1-1 Location Map



1.3 Data Sources

Data were obtained from the following sources:

- Development proposals and design of scheme:
    - Drawings and description supplied by MMG JV (including ground engineering concept - MMG JV, 2014a)
- Topography, climate and land use:
    - Digital Ordnance Survey (OS) mapping: OS OpenData, including Raster250k (1:250,000 scale), VectorMap (1:50,000) and StreetView (1:10,000).
    - Digital Terrain Models (DTMs):
        - Terrain50 from OS OpenData (low resolution: 50 m grid)
        - LIDAR topography (high resolution: 0.5 m grid).
    - Catchment descriptors from the Flood Estimation Handbook (FEH) and CD-ROM (CEH, 2009).
- Soils:
    - 1:250,000-scale mapping by the Soil Survey of England and Wales (Soil Survey of England and Wales, 1983).
- Geology and hydrogeology:
    - 1:50,000 scale mapping by the British Geological Survey (BGS, 1981, 1983, 1995, 1998)
    - Geological desk study review undertaken by MMG JV (including an interpretation of the BGS's Lithoframe model)
    - BGS reports on the superficial deposits of the Holderness area (Burke *et al*., 2010) and on the Yorkshire Chalk (Gale and Rutter, 2006).
    - Baseline hydrochemical report on the Chalk Aquifer of Yorkshire and North Humberside (Smedley *et al*., 2004)
    - Major (Principal) Aquifer properties manual (Allen *et al*., 1997)
    - ESI reports relating to the regional Chalk model used by the Environment Agency (ESI, 2010, 2013)
    - Borehole logs produced by Geotechnics (2013) and supplied by MMG JV.
    - Groundwater level monitoring (including pumping test) data supplied by MMG JV.
    - Aquifer properties data from the 2013 ground investigation and pumping test supplied by MMG JV.
    - Information on licensed groundwater abstractions (supplied by the Environment Agency).
- Existing MMG JV groundwater model:
    - Partially completed regional MODFLOW model in Groundwater Vistas version 6 format with layer tops/bases derived from BGS Lithoframe-50 ESRI data (BGS, 2013), and preliminary aquifer properties and boundary conditions.
- Tidal data for Albert Dock (Jan 2014: water level time series with 10 minute time step):
    - ABP Humber Estuary Services website:
        - http://www.humber.com/Live_Information/Live_Tide_Data/

2 Data Review and Conceptual Model

2.1 Introduction

"A conceptual model is a description of how a hydrogeological system is believed to behave" (Environment Agency, 2002, p.4.1-1). "Conceptual models describe how water enters an aquifer system, flows through the aquifer system and leaves the aquifer system" (Rushton, 2003, p.2). A conceptual model is therefore "an understanding of how a particular groundwater system works" (Brassington, 2007, p.5). A conceptual model is a prerequisite to any mathematical modelling exercise.

The development of a conceptual model is generally "an iterative or cyclical process of development and testing" (Environment Agency, 2002, p.4.1-2) in which new observations are used to evaluate and improve the model. As noted in Section 1.2, MMG JV's conceptual understanding of the hydrogeology of the A63 Castle Street site evolved as more data became available.

This chapter provides a brief review of the data used to develop an understanding of the hydrogeology of the A63 Castle Street road improvement site and surrounding area. It also presents the conceptual model that informs the numerical modelling work.

Please note that this chapter is intended only as a brief summary and that a more detailed account of the hydrogeology will be presented by MMG JV in the Groundwater Report, a technical appendix to the Environmental Statement (ES). This report will also form a technical appendix to the ES.

2.2 Topography, Climate and Land Use

The site (National Grid Reference 509337 428392) is located in south-central Hull, 315 - 540 m north of the Humber Estuary. It is centred on Mytongate Roundabout, which forms the junction between Castle Street (A63), Commercial Road, Hessel Road (A63) and Ferensway (A1079). The site is low-lying and of relatively low relief, with ground levels varying from about 3 mAOD to 5 mAOD. Land-use at the site is urban, with an existing road and junction. Nearby are docks associated with the River Humber.

The Flood Estimation Handbook (FEH) CD-ROM includes long-term average rainfall data for catchments in the UK. For a small FEH catchment immediately north of the site, the Standard Average Annual Rainfall (SAAR) is 640 mm for the period 1961-1990 and 637 mm for the period 1941-1970 (CEH, 2009). Figure 3.6 in ESI (2010) gives the Long-Term Average (LTA) rainfall for the site and immediately surrounding area as 1.7 to 1.9 mm/day, which is 621 to 694 mm/yr.

2.3 Geology and Soils

The bedrock immediately underlying the site belongs to the Cretaceous Chalk Group (Table 2-1). Strata within the Chalk dip eastwards or east-northeastwards at an angle of about 1° to the horizontal (estimated from Figure 2.3 in Gale and Rutter, 2006). Overlying the bedrock are superficial deposits some 20 to 34 m thick. These include fluvioglacial deposits, glaciolacustrine deposits, glacial till, tidal flat deposits and (artificial) made ground (Table 2-1). The superficial deposits display rapid lateral variations in thickness (Burke *et al*., 2010).

Soils mapping by the Soil Survey of England and Wales (1983) shows the area of Hull as unsurveyed. However, ESI (2010) report seasonally wet deep clay and seasonally wet deep loam as the dominant soil types.

Table 2-1 Stratigraphy

Age	Group	Formation / Unit		Description	Thickness [m]
Quaternary		Made Ground		Highly variable: clay, silt, sand, gravel. Includes brick, tarmac, concrete, ash and stone (chalk, other limestone, flint, sandstone, quartzite)	0.69 - 13 (on site)
		Tidal Flat Deposits and Alluvium	Cohesive Alluvium (MMG JV terminology)	Silt and clay (locally sandy) with local development of peat.	0 - 13.4 (on site)
			Granular Alluvium (MMG JV terminology)	Sand and gravel	0 - 13.6 (on site)
		Glacial Till		Diamicton ("boulder clay")	0 - 6.3 (on site)
		Glaciolacustrine Deposits		Laminated clay (locally silty / sandy)	1.5 - 9.7 (on site)
		Fluvioglacial Deposits		Sand and gravel	0 - 9.6 m (on site)
Cretaceous	Chalk Group	Flamborough Chalk Formation		Chalk with little or no flint	260 - 280 (regionally)
		Burnham Chalk Formation*		Thinly-bedded chalks with flint bands	130 - 150 (regionally)
		Welton Chalk Formation		Massive or thickly-bedded chalk with flint nodules	44 - 53 (regionally)

Notes:

*The Burnham Chalk directly underlies the superficial deposits across most of the model area.
Total thickness of superficial deposits on site ranges from 20.6 to 33.6 m.

Sources:
BGS (1981, 1983, 1995, 1998)
Smedley *et al.* (2004)
Gale and Rutter (2006)
Burke *et al.* (2010)
Geotechnics (2013)

2.4 Surface Water Hydrology

The tidal Humber Estuary is located 315 to 540 m south of the site, and the River Hull (also tidal in its lower reaches) is about 600 m to the east. The River Hull flows southwards and joins the Humber some 560 m southeast of the site. Beverley and Barmston Drain flows south-eastwards through Hull and joins the River Hull about 1.3 km northeast of the site. Ganstead/Holderness Drain flows broadly from north to south through eastern Hull, joining the Humber Estuary 3.5 km east of the site. The agricultural land surrounding Hull is drained by a network of open ditches.

There are a number of docks in the vicinity of the site, of which the closest are Railway Dock (c.65 to 125 m to the south or southeast), Humber Dock (20 m to the southeast) and Prince's Dock (30 m to the northeast). Further away are Albert Dock (c.400 m to the south) and Humber Dock Basin (c.375 m to the southeast). The docks are surrounded by walls, and lock gates are used to maintain water levels against the falling tide.

The tidal range in the Humber Estuary is approximately six metres (ABP Humber Estuary Services website).

2.5 Hydrogeology

2.5.1 Geological Framework: Aquifers and Aquitards

Table 2-2 summarises the local hydrogeology.

Table 2-2 Hydrogeological Units

<table>
<tr><th>Age</th><th>Group</th><th colspan="2">Formation / Unit</th><th>Hydrogeology</th></tr>
<tr><td rowspan="6">Quaternary</td><td rowspan="6"></td><td colspan="2">Made Ground</td><td>**AQUIFER** or AQUITARD
Highly variable hydraulic properties, depending on its local composition. For example, concrete may be a barrier to groundwater flow, whereas gravel-filled service trenches may be preferential flow paths.

Commonly dry in boreholes drilled by Geotechnics (2013).

Perched groundwater bodies likely to be developed above low permeability layers in the made ground or above the underlying low permeability tidal flat deposits.</td></tr>
<tr><td rowspan="2">Tidal Flat Deposits and Alluvium</td><td>Cohesive Alluvium (MMG JV terminology)</td><td>AQUITARD
Dominated by low permeability silt and clay.

Boreholes drilled by Geotechnics (2013) most commonly struck water in this layer.</td></tr>
<tr><td>Granular Alluvium (MMG JV terminology)</td><td>**AQUIFER**
Intergranular flow and storage.</td></tr>
<tr><td colspan="2">Glacial Till</td><td>AQUITARD
Dominated by low permeability diamicton rich in silt and clay.</td></tr>
<tr><td colspan="2">Glaciolacustrine Deposits</td><td>AQUITARD
Dominated by low permeability laminated silt and clay.</td></tr>
<tr><td colspan="2">Fluvioglacial Deposits</td><td>**AQUIFER**
Intergranular flow and storage. In hydraulic continuity with the underlying Chalk aquifer.</td></tr>
<tr><td rowspan="3">Cretaceous</td><td rowspan="3">Chalk Group</td><td colspan="2">Flamborough Chalk Formation</td><td rowspan="3">**PRINCIPAL AQUIFER** of regional importance for water supply. There are a number of licensed abstractions from the Chalk, some of them large abstractions for public supply.

"Dual porosity" aquifer. Groundwater flow takes place mainly through fractures.

Effective aquifer taken by ESI (2010) to be the upper 30 - 50 m of the unconfined Chalk and the upper 10 - 15 m of the confined Chalk.</td></tr>
<tr><td colspan="2">Burnham Chalk Formation*</td></tr>
<tr><td colspan="2">Welton Chalk Formation</td></tr>
<tr><td colspan="5">Notes:
Stratigraphy from sources listed in Table 2-1.
Aquifer classification of Chalk from Environment Agency website
General information on Chalk hydrogeology from Allen et al. (1997)
Abstractions data provided by the Environment Agency.</td></tr>
</table>

2.5.2 Aquifer properties

This section provides a brief summary of available data relating to the following aquifer properties: porosity, hydraulic conductivity (K), transmissivity (T=Kb, where b = saturated thickness), storativity (S = volume of water released per unit surface area of aquifer per unit decline in head), specific storage (Ss = volume of water released per unit volume of aquifer per unit decline in head) and specific yield (Sy = volume of water drained from an unconfined aquifer per unit decline in the water table).

Chalk Aquifer

The Chalk is a Principal (formerly Major) Aquifer, a classification that includes strata with high permeability and storage and that "may support water supply and/or river base flow on a strategic scale" (Environment Agency website; Allen *et al.*, 1997). The Chalk has a 'dual porosity' system, with (i) a low permeability microporous matrix and (ii) fractures; most groundwater flow occurs within the fractures (Allen *et al.*, 1997). Matrix porosity is generally high (Allen et al., 1997, give a mean value of about 19% for the Middle Chalk of Northern England, n = 191), but pore throats are very small, giving a low permeability. Fracture pores make up a much smaller proportion of the total aquifer volume, but contribute most of the permeability.

The Chalk displays significant variation in hydraulic properties with depth. In particular, hydraulic conductivity is greatest in the upper part of the aquifer, especially within the zone of water table fluctuation, where fracture permeability is enhanced by dissolution (Allen *et al.*, 1997). There is generally relatively little groundwater flow at depths greater than 50 m below the water table (or below the top of the Chalk where the aquifer is confined) (Allen *et al.*, 1997). ESI (2010) took the effective aquifer to consist of the upper 30 to 50 m of the unconfined Chalk and the upper 10 - 15 m of the confined Chalk. ESI (2013) used MODFLOW VKD to model the Chalk as a layer with K varying with depth.

Allen *et al.* (1997) provide the following aquifer properties data for the Yorkshire Chalk (87 pumping tests from 68 sites):

T: <1 to >10,000 m^2/d, with a geometric mean of 1,258 m^2/d.

S: 1.5 x 10^{-4} to 1.0 x 10^{-1}, with a geometric mean of 7.2 x 10^{-3}.

The wide range of transmissivity values reflects the importance of fracture permeability in determining local transmissivity. Table 2-3, from Allen *et al.* (1997), shows storage properties estimated by the BGS for the Chalk of the northern province, which includes Yorkshire.

Within their regional groundwater model, ESI (2013) defined three hydraulic conductivity (and transmissivity) zones in the Chalk beneath Hull; from west to east these are:

Zone 19 - Confined Hull corridor: K = 25 m/d; T = 750 m^2/d.

Zone 8 - Confined (north): K = 15 m/d; T = 500 m^2/d

Zone 9 - Holderness: K = 5 m/d; T = 150 m^2/d.

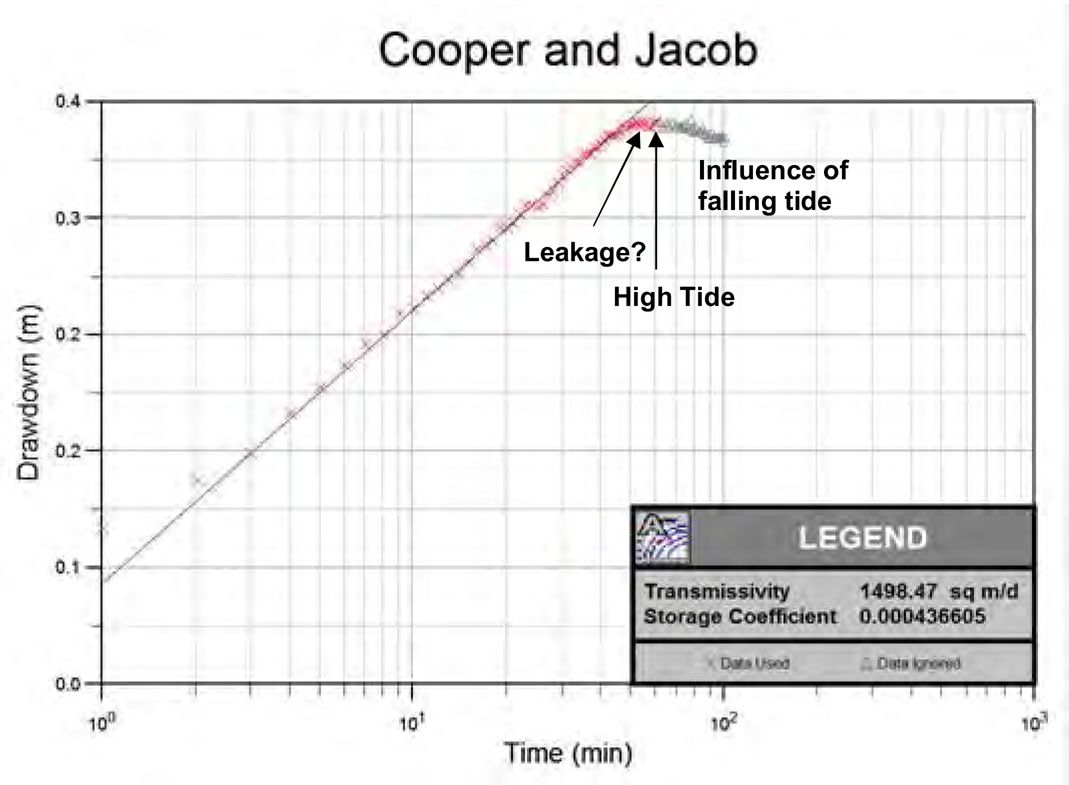
Table 2-3 Storage coefficients for the Chalk in the northern province (from Allen et al., 1997).

	<10 m below top Chalk		10–30 m below top Chalk		>30 m below top Chalk	
	Sy	Ss (m^{-1})	Sy	Ss (m^{-1})	Sy	Ss (m^{-1})
RWL above top of Chalk	$0.5–2 \times 10^{-2}$	$1.5–1.6 \times 10^{-5}$	$2–7 \times 10^{-3}$	$1.5–1.6 \times 10^{-5}$	1×10^{-3}	$0.66–1.6 \times 10^{-5}$
RWL in Upper Chalk	$0.5–2 \times 10^{-2}$	$1.5–1.6 \times 10^{-5}$	$0.5–2 \times 10^{-2}$	$1.5–1.6 \times 10^{-5}$	$1–2 \times 10^{-3}$	$0.66–1.6 \times 10^{-5}$
RWL in Middle Chalk	$0.5–2 \times 10^{-2}$	$6.6–7.1 \times 10^{-6}$	$2–7 \times 10^{-3}$	$6.6–7.1 \times 10^{-6}$	1×10^{-3}	$6.6–7.1 \times 10^{-6}$
RWL in Lower Chalk	$0.5–1 \times 10^{-2}$	$6.6–7.1 \times 10^{-6}$	2×10^{-3}	$6.6–7.1 \times 10^{-6}$	1×10^{-3}	$6.6–7.1 \times 10^{-6}$

Sy = specific yield; Ss = specific storage; RWL = rest water level in Spring 1975.

MMG JV undertook a pumping test in Chalk borehole LDBH01, located close to the centre of the site (just to the south of the deepest part of the proposed excavation). The data had to be corrected for tidal influence. Once this was done, standard analysis techniques could be applied (Figure 2-1). Theis analysis and Cooper-Jacob analysis (undertaken by MMG JV using the software AquiferWin32) gave T values ranging from about 1,400 to 1,600 m^2/d and S values ranging from 0.0003 to 0.0015. For an assumed effective aquifer thickness of 20 m the T values correspond to hydraulic conductivity (K) values of about 70 to 80 m/d. Figure 2-1 shows deviation from a confined response after about 50 minutes. It is understood from MMG JV that high tide occurred at 60 minutes, and the response after this reflects the falling tide. However, the flattening of the trace between 50 and 60 minutes may reflect leakage from the overlying superficial deposits.

Figure 2-1 Cooper-Jacob Time-Drawdown Plot for Borehole LDBH01 Pumping Test (provided by MMG JV)



Superficial Deposits

Pumping tests undertaken by MMG JV on boreholes at the site gave the following ranges of hydraulic conductivity for the superficial deposits:

Cohesive alluvium	0.06 - 0.2 m/d	(3 tests)
Granular alluvium	0.8 - 3.9 m/d	(16 tests)
Glacial till	0.02 - 0.03 m/d	(4 tests)
Glaciolacustrine deposits	0.02 m/d	(1 test)

ESI (2013) used the following initial values for their numerical model:

Glacial clays	Kxy = 0.01 m/d; Kz = 0.001 m/d; Sy = 0.001.
Sands and gravels	Kxy = 10 m/d; Kz = 0.5 m/d; Sy = 0.04.
Alluvium	Kxy = 1 m/d; Kz = 0.1 m/d; Sy = 0.04.
Tidal flat deposits	Kxy = 0.1 m/d; Kz = 0.01 m/d; Sy = 0.02.

These initial values were revised locally during model calibration (ESI, 2013).

Table 2-4 and Table 2-5 list typical literature values for the hydraulic conductivity and storage properties of unconsolidated sediments.

Table 2-4 Literature Hydraulic Conductivity (K) Values for Unconsolidated Sediments

Material	K [m/d] - order of magnitude range
Clean gravel	≥ 1000
Clean sand / sand and gravel	10 to 100
Fine sand	0.1 to 10
Silt, clay and mixtures of sand, silt and clay	0.0001 to 0.1
Massive clay	< 0.0001
Source: Brassington (2007) These are indicative values. Natural deposits have variable porosity and specific yield. * Drainable storage.	

Table 2-5 Literature Porosity and Specific Yield Values for Unconsolidated Sediments

Material	Porosity (fraction)	Specific Yield* (fraction)
Coarse gravel	0.28	0.23
Medium gravel	0.32	0.24
Fine gravel	0.34	0.25
Coarse sand	0.39	0.27
Medium sand	0.39	0.28
Fine sand	0.43	0.23
Silt	0.46	0.08
Clay	0.42	0.03
Source: Brassington (2007) These are indicative values. Natural deposits have variable porosity and specific yield. * Drainable storage.		

2.5.3 Recharge

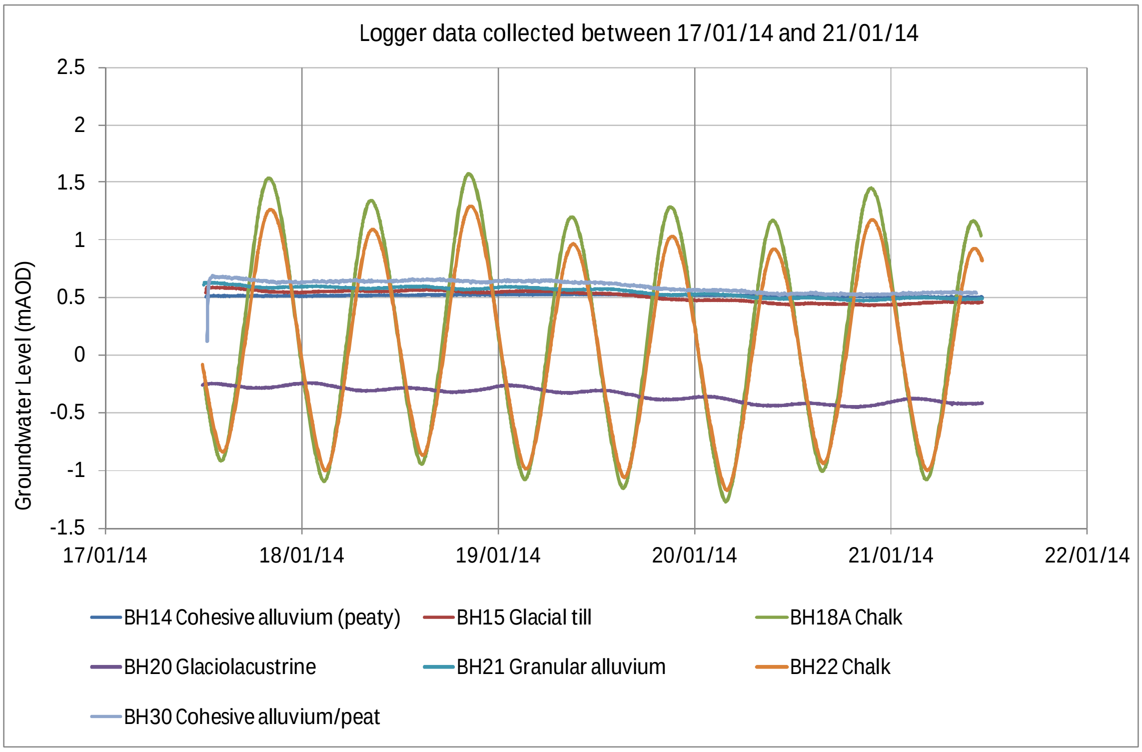
ESI (2013) used the distributed recharge model 4R to estimate recharge to groundwater over the area of the East Yorkshire Chalk Model (ESI, 2013). Figure 2.8 in ESI (2013) indicates that the study area has a recharge of 50 to 100 mm/year, i.e. 1.4 x 10^{-4} m/d to 2.7 x 10^{-4} m/d, or about 8% to 15% of the Long Term Average (LTA) rainfall of 663 mm for the model area (ESI, 2010; averaging period 1971 - 2000).

2.5.4 Groundwater levels and flows

Groundwater contours drawn by ESI (2010) for the East Yorkshire Chalk aquifer for February 2001 and December 2005 suggest that hydraulic gradients are very low in the Hull area, with heads close to 0 mAOD. Regionally, flow is broadly eastwards from the unconfined Chalk in the Yorkshire Wolds to the North Sea. However, in the area of the site there would appear to be little horizontal variation in head, except for (i) drawdown in the vicinity of pumping boreholes and (ii) a variable north-south gradient induced by tidal fluctuations in the level of the River Humber. Groundwater monitoring data collected by MMG JV confirm that the confined Chalk beneath the coastal parts of Hull displays strongly defined tidal fluctuations (see Figure 2-2).

Since December 2013, MMG JV has monitored groundwater levels in a number of boreholes screened in the superficial deposits. Groundwater levels in these deposits show little variation with the tides, and lie mostly in the elevation interval 0.25 mAOD to 1.25 mAOD (Figure 2-2). For comparison, average heads in the Chalk are about 0 to 0.25 mAOD. This means that there is typically - on average - a very slight downward hydraulic gradient; however, tidal fluctuations will reverse this periodically. Groundwater flow is likely to be more-or-less horizontal in the permeable sand and gravel layers and vertical in the silt/clay aquitard layers.

Figure 2-2 Example Groundwater Level Hydrographs from the A63 Castle Street Site (provided by MMG JV)



2.5.5 Groundwater - surface water interaction

The Baseflow Index (BFI) is an FEH catchment descriptor that provides an indication of the proportion of streamflow made up by baseflow (mainly groundwater input). For a small FEH catchment immediately north of the site the BFI is 0.73. This reflects the presence of Chalk bedrock which, where unconfined, gives rise to watercourses fed mainly by groundwater. However, beneath Hull the Chalk is covered (and confined) by relatively low permeability superficial deposits, so the BFI value is unrealistically high.

No information is available regarding flows in the River Hull, Beverley and Barmston Drain, or Ganstead/Holderness Drain. It is likely that these watercourses do receive water from the ground but that the degree of connection between surface water and groundwater is restricted by the relatively low permeability of the clay-rich tidal flat deposits (alluvium) in the upper part of the ground profile. In the tidal part of the River Hull the flow of water between the river and the ground may reverse, depending on the stage of the tide.

ESI (2013) assumed the following conductances (=KA/b where K = hydraulic conductivity of river bed, A = channel width x length, and b = bed thickness) for rivers and drains in the East Yorkshire Chalk Model:

Rivers on sand and gravel:	2000 m^2/d
Rivers on alluvium:	1000 m^2/d
Rivers on tidal flat deposits:	200 m^2/d
Rivers on till:	100 m^2/d
Drains on sand and gravel:	200 m^2/d
Drains on alluvium:	10 m^2/d
Drains on glacial clays and tidal flat deposits:	1 m^2/d

Groundwater monitoring data from the A63 Castle Street site (Figure 2-2) indicate that there is a connection between groundwater and the tidal Humber Estuary (the tidal River Hull may also be exerting an influence). Groundwater quality data suggest that this connection may involve estuarine water flowing in and out of the aquifer, and not merely a pressure head response (information from MMG JV). ESI (2013) modelled the estuary as having an average stage of 0 mAOD, a near-constant depth of 9 fathoms or 16.46 m (based on data from the University of Birmingham) and a conductance of 100 m^2/d, representing a limited connection between the estuary and the Chalk. Monitoring data suggest a limited connection between the estuary and the superficial deposits (Figure 2-2); however, tidal fluctuations are discernible in some boreholes.

2.5.6 Groundwater Abstractions

There are a number of licensed groundwater abstractions from the Chalk aquifer in the vicinity of the site (Table 2-6). Further away, on the western and northwestern edge of Hull are several large public groundwater abstractions belonging to Yorkshire Water: Dunswell, Cottingham, Keldgate and Springhead. These all abstract from the Chalk and have Source Protection Zones (SPZs) defined around them.

Table 2-6 Licensed Groundwater Abstractions within a 5 km Radius of the Site

Easting	Northing	Licence	Name	Aquifer	Annual Quantity [m^3/yr]	Use
508660	427850	2/26/32/049	Smith and Nephew Medical Ltd	Chalk	221,686	Cooling
506200	430050	2/26/32/059	Ideal Standard Manufacturing (UK) Ltd	Chalk	700 (daily rate)	Machinery and electronics
508950	429200	2/26/32/423	Hull Truck Theatre Co Ltd	Chalk	33,600	Cooling
509981	430400	NE/026/0032/038	Robin Concrete and Waste Disposal Ltd	Chalk	3,500	Refuse and recycling; dust suppression.
Notes: Data supplied by the Environment Agency						

2.5.7 Saline intrusion

Historically, artesian conditions existed in the Chalk under Hull, and groundwater discharged from springs adjacent to the Humber Estuary (ESI, 2010). Heavy groundwater abstraction from the Chalk aquifer has led to a lowering of groundwater heads; this has caused springs to dry up and has also reversed hydraulic gradients, leading to the intrusion of brackish/saline water from the estuary into the aquifer (ESI, 2010).

2.6 Hydrogeological Conceptual Model

The main features of the hydrogeological conceptual model developed for the site are summarised below. Further details are provided in the ES produced by MMG JV.

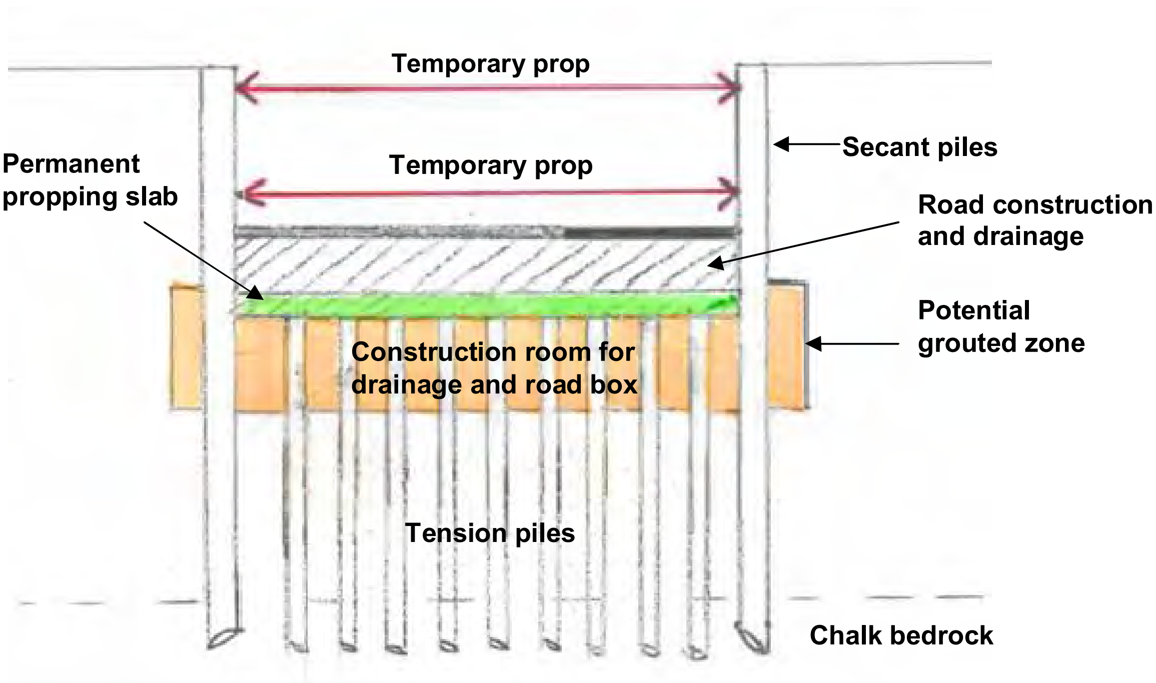
- The site is a low-lying urban area of low topographic relief with an annual average rainfall of about 640 mm. Recharge to groundwater is likely to be between about 50 and 100 mm/year.
- The site is underlain by about 20 - 34 m of superficial deposits that rest on Chalk bedrock.
- The Chalk is a Principal Aquifer, and the superficial deposits include several aquifer layers (made ground, granular alluvium and fluvioglacial deposits) interbedded with aquitards (cohesive alluvium, glacial till and glaciolacustrine deposits). The uppermost layer comprises made ground with highly variable hydraulic properties.
- The Chalk and overlying fluvioglacial deposits are in hydraulic continuity and effectively form a single aquifer unit. This aquifer is confined by the overlying glaciolacustrine and glacial till aquitards. A pumping test undertaken by MMG JV suggests that the Chalk is a leaky confined aquifer with a high hydraulic conductivity.
- Within the Chalk, most flow takes place within fractures. The effective aquifer is limited to the upper, more fractured, part of the Chalk. This effective aquifer layer is likely to be some 10 to 30 m thick.
- Within the superficial deposits the main aquifer layers are the granular alluvium and fluvioglacial deposits, both of which are dominated by sand and gravel. The made ground can also act as an aquifer layer and locally contains perched groundwater.
- Groundwater beneath the site is hydraulically connected to surface water bodies, the nearest being the River Hull and the Humber Estuary. Both of these show tidal fluctuations in water level. In the Humber Estuary, the tidal range is about six metres.
- On average (and across much of the Hull) the hydraulic gradients are low, although groundwater heads along the southern edge of Hull - especially in the Chalk - show significant tidal fluctuations, potentially leading to flow reversals.
- There are a number of licensed groundwater abstractions in the area, all abstracting from the Chalk aquifer. Large public water supply abstractions to the west and northwest of Hull have created a flow divide, within the Chalk, running roughly northeast - southwest and corresponding to the outer edge of SPZ3 for the boreholes.
- Historically, heavy abstraction from the Chalk aquifer has led to saline intrusion from the Humber Estuary. There is some evidence to suggest that groundwater salinity varies in response to tidal fluctuations in some parts of the Chalk aquifer close to the estuary.

2.7 Relevant Features of the Proposed Development

The proposed development is described in detail in the ES produced by MMG JV (MMG JV, 2014b). Only the features relevant to development of the groundwater model are described here. Figure 2-3 is a schematic cross-section showing the basic structure of the scheme at the deepest part of the cutting (the road will slope down from both ends towards the location of the existing Mytongate roundabout, which will be the deepest part).

The road will run within a newly-excavated "canyon" defined by faced secant pile walls. The secant pile wills will be 0.9 m thick and will have a low hydraulic conductivity of 8.6 x 10^{-5} m/d. Temporary props will be used to brace the sides of the excavation during construction, and a permanent propping slab will be positioned in the base. The road construction will sit above the permanent propping slab. Tension piles will anchor the slab, with the piles being driven into either cohesive superficial deposits (about 2 m into the glaciolacustrine deposits at their deepest point) or the Chalk bedrock below (about 4 m into the Chalk at their deepest point). Jet grouting or soil mixing may possibly be used to strengthen the ground beneath the slab.

Figure 2-3 Idealised Transverse Vertical Cross-Section of Scheme at Maximum Dredge (from MMG JV, 2014)



Note: there is an alternative proposal to pile into the cohesive superficial deposits rather than into the Chalk bedrock

3 Groundwater Modelling: Baseline

3.1 Introduction

This chapter describes the construction of the numerical groundwater flow model and its calibration to represent the baseline, i.e. pre-development, hydrogeological conditions in the site and surrounding area. The following chapter (Chapter 4) describes the use of the model to predict the impact of the development on groundwater levels and flows.

3.2 General Approach

The three-dimensional transient (time-variant) flow of groundwater of constant density through an anisotropic porous medium in rectangular Cartesian Coordinates[1] is described by the following partial differential equation:

$$\frac{\partial}{\partial x}\left(K_{xx}\frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(K_{yy}\frac{\partial h}{\partial y}\right)+\frac{\partial}{\partial z}\left(K_{zz}\frac{\partial h}{\partial z}\right)-W=S_s\frac{\partial h}{\partial t} \qquad \text{Equation 1}$$

where x,y,z are distances [L] parallel to the x, y and z axes respectively, K_{xx} = hydraulic conductivity [LT^{-1}] in x direction, K_{yy} = hydraulic conductivity [LT^{-1}] in y direction, K_{zz} = hydraulic conductivity [LT^{-1}] in z direction, W = source or sink [LT^{-1}], S_s = specific storage [dimensionless], h = hydraulic head [L] and t = time [T]. A derivation of this equation may be found in Anderson and Woessner (2002).

Given suitable boundary conditions (heads or flows at the boundaries of the system) and an initial condition (heads at t=0) the above equation can be solved to yield groundwater heads (and therefore flows) as a function of space and time. For steady-state problems heads do not change with time, and the right-hand-side of the equation is zero; in this case only boundary conditions are required.

For very simple problems, such as those involving one-dimensional flow through a homogeneous aquifer, the groundwater flow equation can be solved analytically to yield an equation expressing head as a function of space and/or time. However, many practical problems require modelling of heterogeneous anisotropic aquifer systems with complex three-dimensional geometry. In such cases, numerical modelling techniques are usually employed. These approximate the problem by dividing up ("discretizing") the model domain into discrete spatial grid cells or elements, and discrete time steps. This project uses the finite difference method, in which the porous medium is represented by a rectangular grid of cells and the space and time derivatives are approximated using finite differences (see Wang and Anderson, 1982). The result is a finite set of algebraic equations (with a version of the flow equation written for each cell or node) that can be represented in matrix form and solved simultaneously to yield the head in each cell for each time step.

Note that the flow equation in the form written above assumes that the groundwater has a constant density. This is unlikely to be strictly true in Hull, where the groundwater is brackish due to its proximity to the Humber Estuary, and where saline intrusion is known to have occurred in the past. However, for the purposes of this project (which is concerned with a small site) it has been assumed that density variations can be neglected. This aspect could potentially be investigated as part of a separate project, but would require good information on salinity variations in space and time.

3.3 Previous Numerical Modelling Work

In the mid-1980s the University of Birmingham (1985) developed a numerical (finite difference) model of the northern outcrop of the East Yorkshire Chalk. The model was produced using an in-house computer code written in FORTRAN. Aspinwall (1995) extended the model to cover the entire outcrop of the Chalk of East Yorkshire, and this extended model became known as YORKMOD. It was later updated by Entec (2006). YORKMOD was used by the Environment Agency to inform its Catchment Abstraction Management Strategy (CAMS) for the area (ESI, 2010). ESI (2010) describes YORKMOD as it existed in 2010. It was a single layer model of the Chalk but represented vertical variations in hydraulic conductivity, with shallow fissured chalk

[1] As written, the equation assumes that the coordinate axes are aligned parallel to the principal directions of K anisotropy and that the z axis is oriented vertically, i.e. parallel to the direction in which gravity acts.

passing downwards into less fissured, less permeable, chalk (ESI, 2010). The grid spacing ranged from 1,000 m to 250 m. The model was transient, and simulated the period January 1975 to December 2001.

ESI (2010, 2013) produced for the Environment Agency a new regional numerical model (using the USGS's MODFLOW code - see Section 3.4) of the Yorkshire Chalk and overlying superficial deposits. The model was designed for regional water resource management. It has three layers (superficial deposits, active Chalk aquifer and deeper Chalk/Jurassic aquifers) and a model grid composed of square cells each measuring 200 m by 200 m. Vertical variations in hydraulic conductivity within the Chalk are represented using MODFLOW-VKD (Environment Agency, 2003). The model is transient (time variant) and calibrated against long-term groundwater monitoring records. The Hull area is represented at the southern edge of the model, but there are few groundwater level targets in Hull, reflecting a lack of available hydrogeological information with which to constrain the model. According to ESI (2010), "Interaction between the Chalk aquifer and the Humber Estuary is complex, and cannot be quantitatively assessed" (p.65). These complexities led to some uncertainty in the representation of the Humber boundary in the model.

As noted in Section 1.2, MMG JV constructed a regional numerical model (using MODFLOW) to represent groundwater levels and flows in the A63 Castle Street site and surrounding area. This was informed by the existing ESI regional model and also incorporated detailed geological information based on the British Geological Survey's (BGS's) drift model for the superficial deposits in the area (Burke *et al*., 2010). The incorporation of detailed information about the superficial deposits reflected a need to simulate the behaviour of groundwater in these deposits; in contrast, the ESI model focussed on water resource management in the Chalk.

The MMG JV model had eight layers (two representing the Chalk and the rest representing superficial deposits) and a uniform grid composed of square cells each measuring 100 m by 100 m. Individual cells were much wider than the planned road scheme. This layer geometry in the MMG JV model, although much more detailed than that used in the ESI regional model, necessarily simplifies the BGS drift model layer geometry. The BGS drift model is based on 1,398 BGS borehole records, from which 74 geological cross-sections were produced to create an ‘egg crate’ model of the Holderness area. Gridded surfaces were generated from these (Burke *et al*., 2010). Although the BGS drift model represented 28 grouped lithologies, MMG JV focussed on those with reasonable coverage that were also of hydrogeological significance within the area of investigation, in some cases grouping lithologies together. The tops of the selected BGS model layers were used to represent layer geometry in the MODFLOW model. Although some of these layers are not laterally continuous, it proved necessary to assign a minimum layer thickness of 0.5 m to aid model convergence, particularly given the substantially different hydraulic properties between some adjacent layers. MMG JV converted the BGS model layer data into upper surface data for the eight layers of this model and then imported this new layer data into MODFLOW. Where layers pinched out, MODFLOW ‘zones’ were used in each layer to represent the most representative hydraulic properties (see Section 3.9).

The model presented in this report makes use of the geological information contained in the MMG JV model, but focuses on a smaller area essentially restricted to the city of Hull. Grid refinement is used to create a finer grid in the area of the scheme, allowing representation of relatively small-scale (10 m) geometry.

3.4 Numerical Modelling Code and Solver

The model was produced using the USGS's open source numerical modelling code MODFLOW (McDonald and Harbaugh, 1984; McDonald and Harbaugh, 1988; Harbaugh and McDonald, 1996a and 1996b; Harbaugh *et al.*, 2000). The version of MODFLOW employed for this project was MODFLOW2000 (Harbaugh *et al.*, 2000).

MODFLOW uses the finite difference method to solve the partial differential equation describing groundwater flow. Application of the finite-difference method leads to matrix equations that can be solved by direct or by iterative methods. In iterative approaches an initial estimate of the solution is repeatedly refined so that successive solutions approach the true solution. Solution convergence is assumed when the difference in results (e.g. difference in calculated heads) between successive iterations is less than a user-specified convergence criterion.

A number of automated solvers are available for use with MODFLOW. For this project the matrix equations were solved using the Preconditioned Conjugate-Gradient 2 (PCG2) solver, which uses iteration (Hill, 1990). The settings used for PCG2 were: 5,000 maximum outer iterations, 25 maximum inner iterations, head change criterion (for convergence) 0.001 m, residual criterion for convergence = 1, relaxation parameter = 1, matrix preconditioning method = Cholesky, maximum bound on eigenvalue = 2, solver printing option = "Print All", PCG2 summary data printed every five iterations, damping factor = 1. For the steady-state modelling the solver was set to converge if the convergence criteria were met for 9999 outer iterations. For the transient modelling this was relaxed to 5 outer iterations following advice in ESI (2011); this was done to aid convergence. Care was taken to ensure that the model water balance error was acceptable after changing the setting from 9999 to 5 (see ESI, 2011).

MODFLOW2000 has options for resaturation of dry cells (the original version of MODFLOW allowed cells to dry out, but not to become wet again). Whether a dry cell becomes resaturated or not depends on the heads in neighbouring cells. MODFLOW has a number of settings that allow the user to control the way in which resaturation operates, including how often (during the solution process) MODFLOW checks to see if any cells should be re-wetted. For this project the following resaturation options were selected: wetting factor = 1, wetting threshold = 0.2, head assigned to dry cells = -1×10^{30}, wetting iteration interval = 10, wetting equation number = 0 and rewetting option = "Use Only Node Below Dry Cell".

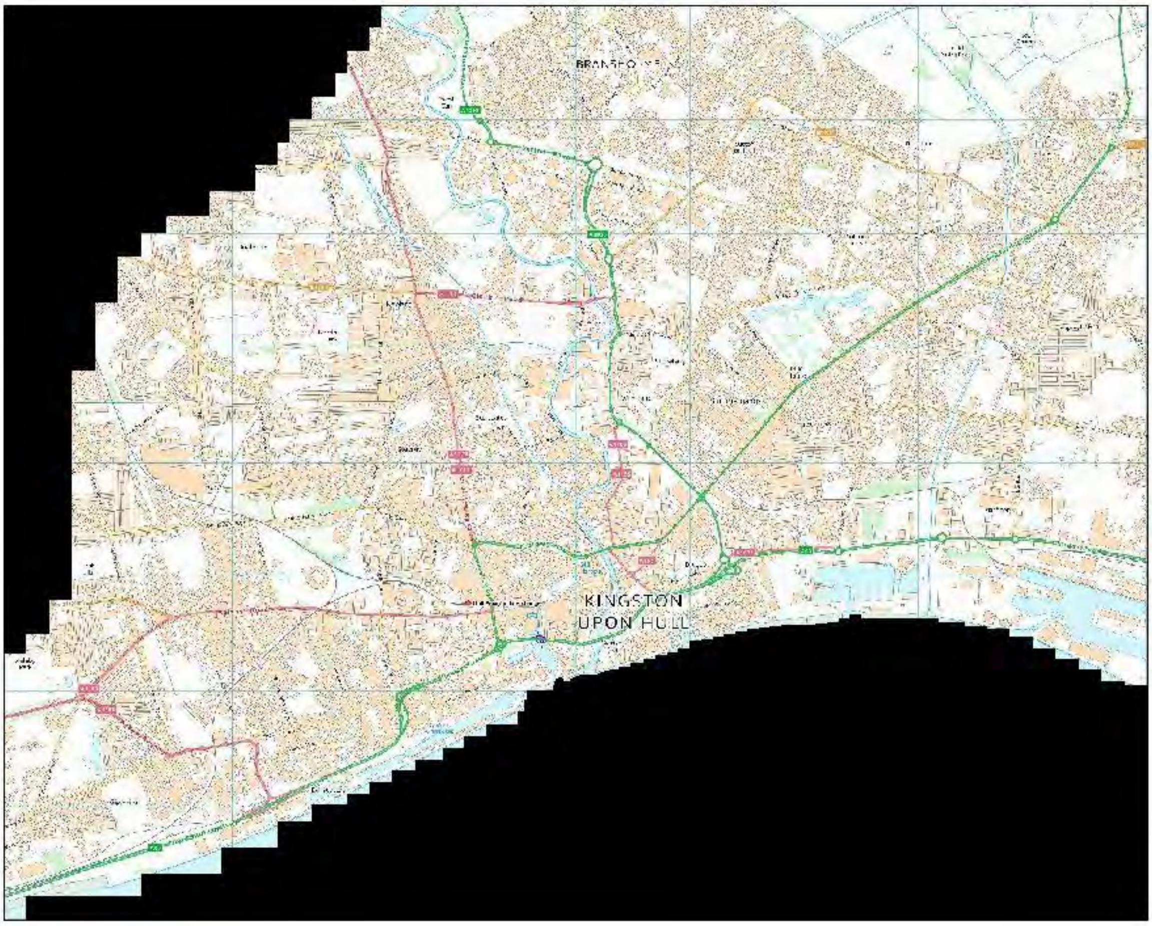
3.5 Graphical User Interface (GUI)

A number of Windows-based Graphical User Interfaces (GUIs) are available to aid with the pre-processing of input data for MODFLOW and also the presentation and analysis of modelling results. This project used Groundwater Vistas, a popular and widely used interface (Groundwater Vistas Version 6.53, Build 8; ESI, 2011).

3.6 Modelled Area

Figure 3-1 shows the extent of the modelled area. The Humber Estuary was chosen as a natural southern boundary. The other boundaries were chosen to be distant from the area of interest so that uncertain boundary conditions didn't impact too strongly on the solution in the area of interest, and also so that the effects of stresses imposed on the system (such as drainage to the road excavation) did not extend to the boundaries (thereby invalidating any heads or flows specified there). Boundary conditions are described in more detail in Sections 3.11 and 3.15.

Figure 3-1 Modelled Area



3.7 Discretization of Space and Time

The numerical approach requires that space and time be "discretized", i.e. divided up into a finite number of small chunks. Spatially, the modelling domain is divided up into cells or blocks. Temporally, the modelled time period is divided up into discrete time steps. In MODFLOW the model domain is subdivided into a finite number of discrete cells using an orthogonal grid; each cell has a central node for which heads are calculated. Time is subdivided into stress periods (for which stresses such as abstraction are constant), and stress periods are further divided into individual time steps. Within a stress period the time steps are set to increase in length according to a geometric progression; this is because closely-spaced steps are required to capture the response of the system to a sudden change in stress (Anderson and Woessner, 2002).

3.7.1 Spatial Discretization: Model Grid and Layering

The model grid is of rectangular outline, measuring 10 km east-west and 8 km north-south. Its lower left-hand corner has National Grid Reference 505000 426000.

The grid contains 114 rows (oriented east-west) and 210 columns. There are 215,460 grid cells, of which 175,923 are defined as active flow cells. Initially individual grid cells were set to be 200 m x 200 m in plan view. The grid was then refined in the area of interest, with minimum cell dimensions of 10 m x 10 m. This smaller grid size allowed:

- Adequate geometrical representation of the 20 m wide road scheme
- Detailed representation of the local geology of the superficial deposits as revealed by the site investigation
- Detailed representation of groundwater heads and flows in the area of interest.

Grid refinement produced variable grid spacing and rectangular cells that were non-square. Care was taken to ensure that the width (or length) of adjacent cells in plan view did not "jump" by a factor of more than 1.5. This was to minimise numerical errors associated with variable grid spacing when using the finite difference method (Anderson and Woessner, 2002).

The model has nine layers; in descending order these are:

1. Made ground
2. Cohesive alluvium
3. Granular alluvium
4. Glacial till
5. Glaciolacustrine deposits
6. Fluvioglacial deposits
7. Uppermost Chalk (uppermost 4 m - to be piled)
8. Main Chalk (main aquifer layer c.20 m thick)
9. Deep Chalk (lower permeability Chalk layer).

Where a geological layer is absent its layer in the model (as listed above) actually represents an underlying, or overlying, stratigraphic unit as described in Section 3.8.

Figure 3-2 Finite Difference Grid (Plan View)

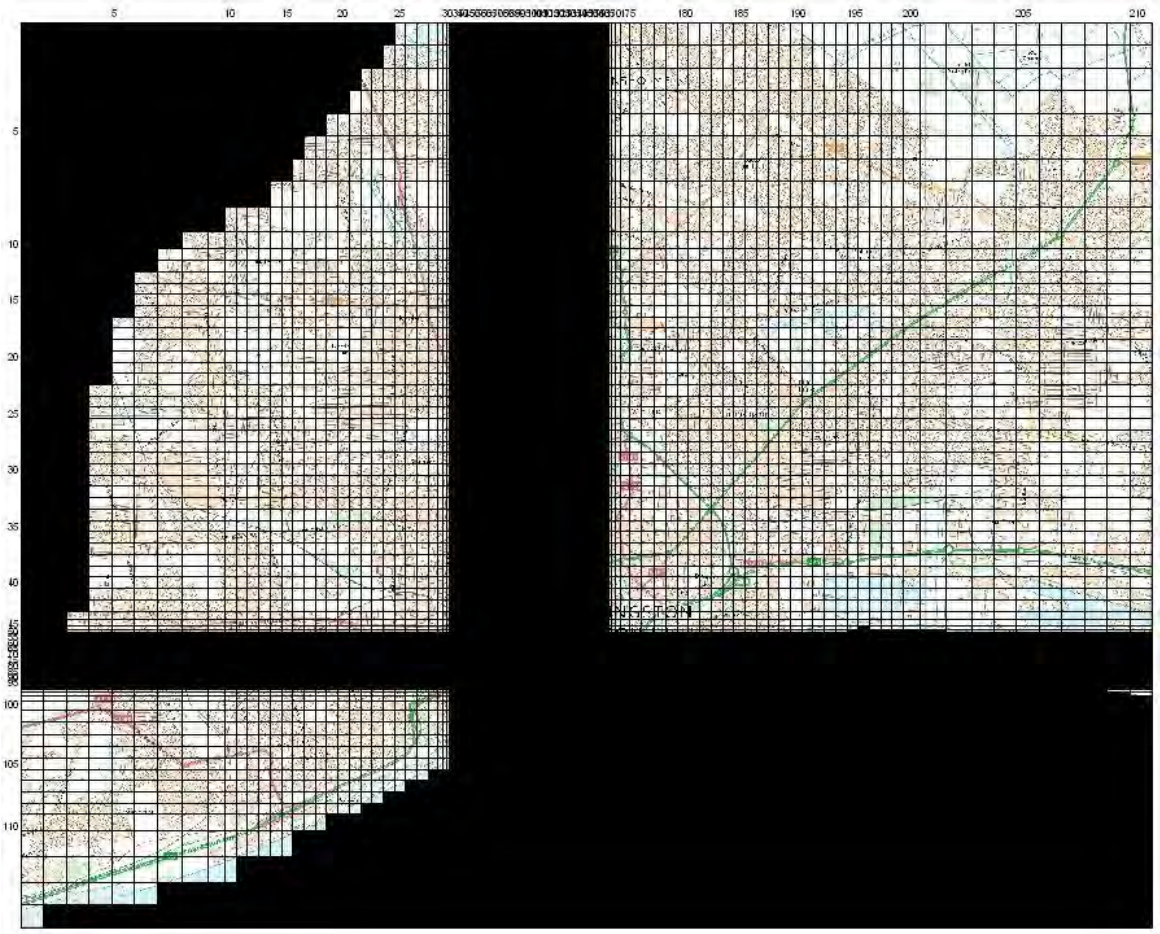


Figure 3-3 Finite Difference Grid (Plan View): View Showing Close-up of Refined Area Around Site

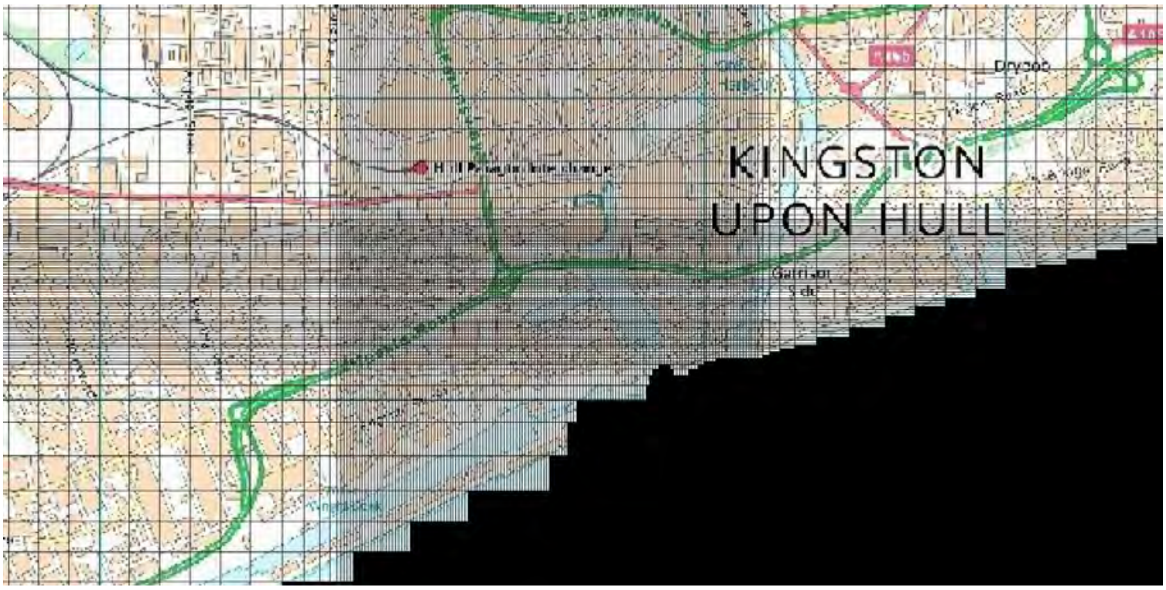
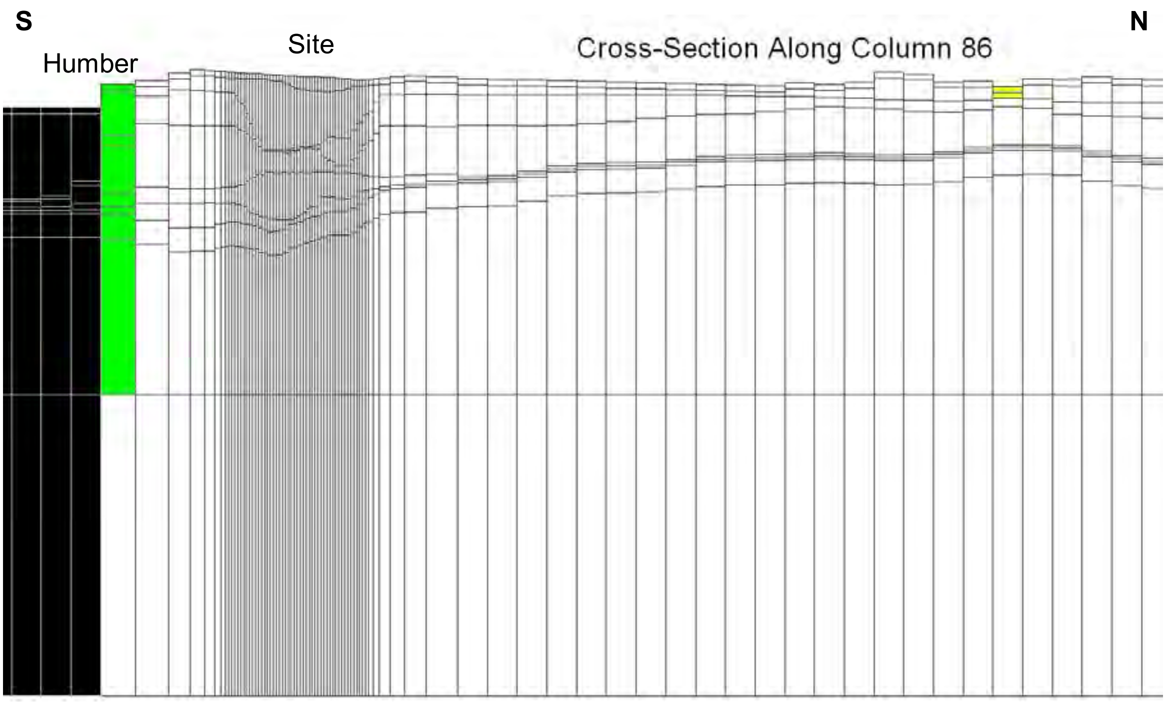


Figure 3-4 Finite Difference Grid (Vertical Section) Showing Layering



3.7.2 Temporal Discretization

Two types of simulation were undertaken using the model: steady-state (heads and flows constant with time) and transient (heads and flows allowed to vary with time to reflect the tidal fluctuations in water level in the Humber Estuary).

For the transient simulations, 50 stress periods were defined. The first stress period was defined as having a single time step and a length of 3650 days; however, when the model was run it was specified as steady-state. This initial stress period established the groundwater heads at a constant level corresponding to an average position of the tide in the Humber. The subsequent stress periods each lasted 0.020833333 days (30 minutes) and had 10 time steps, with a time step multiplier of 1.4 (the multiplier used in the geometric progression used to define time step intervals). Stress periods 2 to 50 were used to represent a single day of tidal fluctuation (two tidal cycles) in the level of the River Humber.

3.8 Layer Geometry

The top of the model was defined using a combination of LIDAR (where available) and OS Terrain50 digital elevation data. Both data types took the form of raster grids in ArcGIS. The LIDAR (0.5 m resolution) was much more detailed than the model grid, so was coarsened (to 2 m grid size) before use. The LIDAR was "stamped on top" of the Terrain50 data and then the combined dataset converted to points (x,y,z) before being imported to Groundwater Vistas. Interpolation between the points to form a surface was undertaken in Groundwater Vistas using Nearest Neighbour analysis. The result was an averaged top elevation for each grid cell in the uppermost layer of the model.

The bases of the various geological layers were determined using (i) elevations from the MMG JV MODFLOW model (exported as point x,y,z data) (refer to Section 3.3.) and (ii) layer bases picked on borehole logs from the recent site investigation. In the detailed area of the site the general MMG JV model elevations were discarded and replaced by data from the site investigation. As with the topography, interpolation between points was undertaken using Groundwater Vistas. Two layer bases were defined differently to the others: the base of Layer 1 was taken to be 1 m below ground level, except in the area of the scheme, where elevations from the borehole data were used instead; the base of Layer 7 was taken as everywhere 4 m below the top of Layer 7 (to represent the top 4 m of the Chalk).

In the superficial deposits, small layer thicknesses meant that interpolation led to layer top/base overlap in some cells. These overlap errors were corrected using the "Fix layer overlap" tool in Groundwater Vistas, with a minimum thickness of 0.25 m being imposed.

MODFLOW requires that layers be continuous across the entire model domain. Where a certain geological layer is absent, this is represented by making the model layer thin and by assigning to it the properties of the underlying, or overlying, layer. This is a standard approach for representing layer "pinch-outs" in MODFLOW.

3.9 Hydraulic Properties

3.9.1 Hydraulic Conductivity

Hydraulic conductivity (K) values were based on: (i) hydraulic test data (from pumping tests, falling head tests and packer tests) supplied by MMG JV, (ii) the ESI regional model and (iii) hydrogeological literature (Brassington, 2007). K values were assigned to numbered zones within the model (see Table 3-1). It was assumed that the layers were isotropic in the horizontal plane (Kx=Ky=Kxy). In general the vertical K value (Kz) was set an order of magnitude lower than the horizontal in order to reflect the influence of stratification. The Glacial Till and Deep Chalk were assumed to be isotropic (Kx=Ky=Kz).

The K values were refined during model calibration (see Section 3.12). During the calibration process, care was taken to ensure that the Kxy value for each zone was realistic, given the hydraulic test results and literature values. Table 3-1 shows the final (calibrated) JBA model values.

3.9.2 Storage and Confinement

Storage properties (Table 3-2) were estimated based on a pumping test undertaken by MMG JV (in the Chalk), values used by ESI (2013) and literature values. The property values were refined during transient calibration so that the amplitude of groundwater level fluctuations matched those observed in monitoring boreholes.

The upper two layers of the model (Made Ground and Cohesive Alluvium) were specified as unconfined. Deeper layers were specified as confined.

Figure 3-5 Vertical Section Showing Layering and K/Storage Zones (zones numbered)

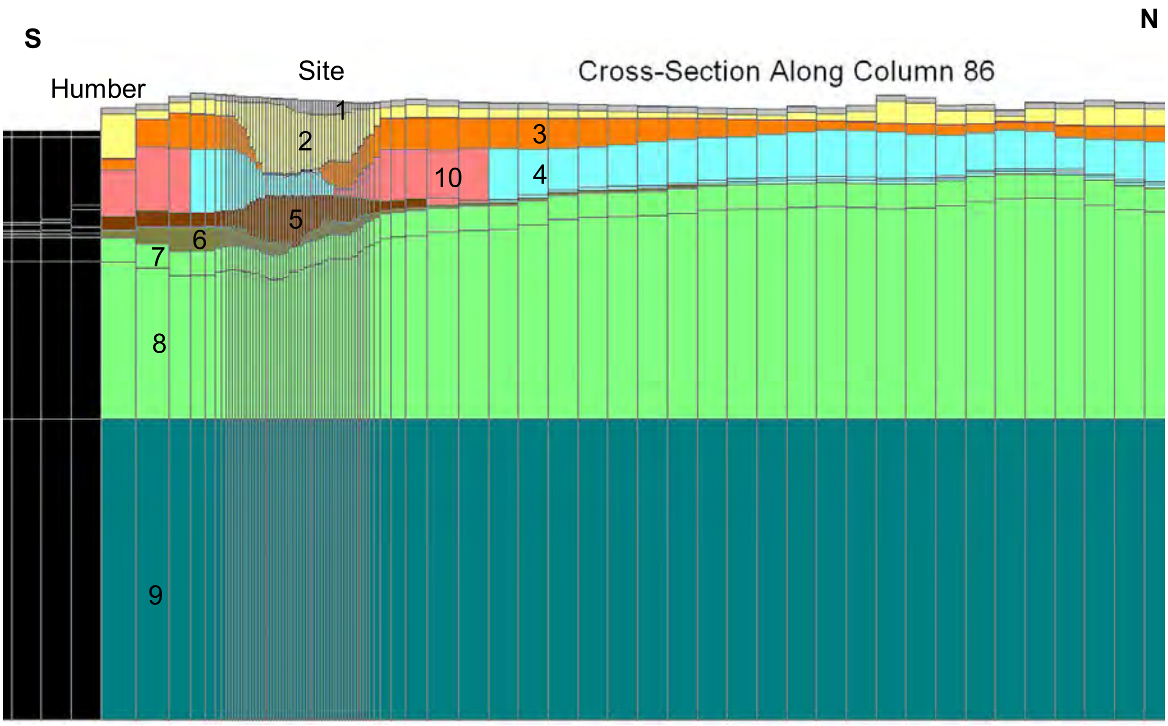


Figure 3-6 K/Storage Zones in Layer 1 (colours as in Figure 3-5)

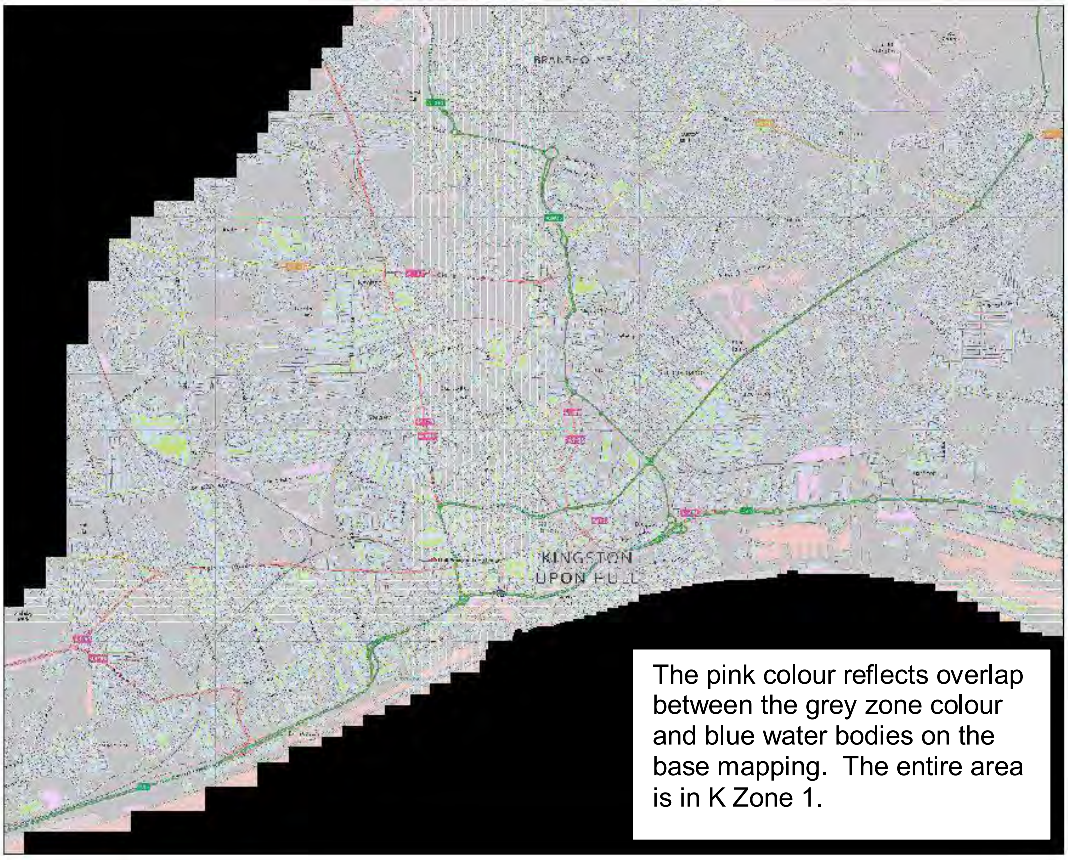


Figure 3-7 K/Storage Zones in Layer 2 (colours as in Figure 3-5)

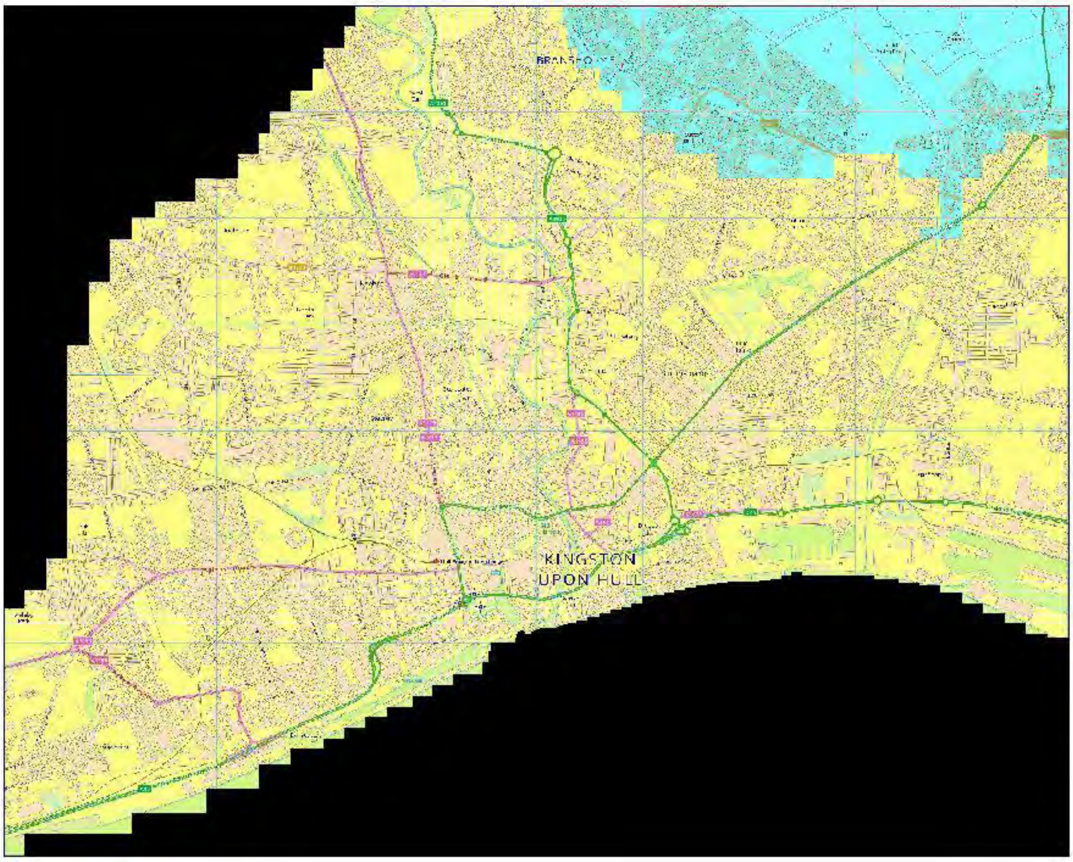


Figure 3-8 K/Storage Zones in Layer 3 (colours as in Figure 3-5)

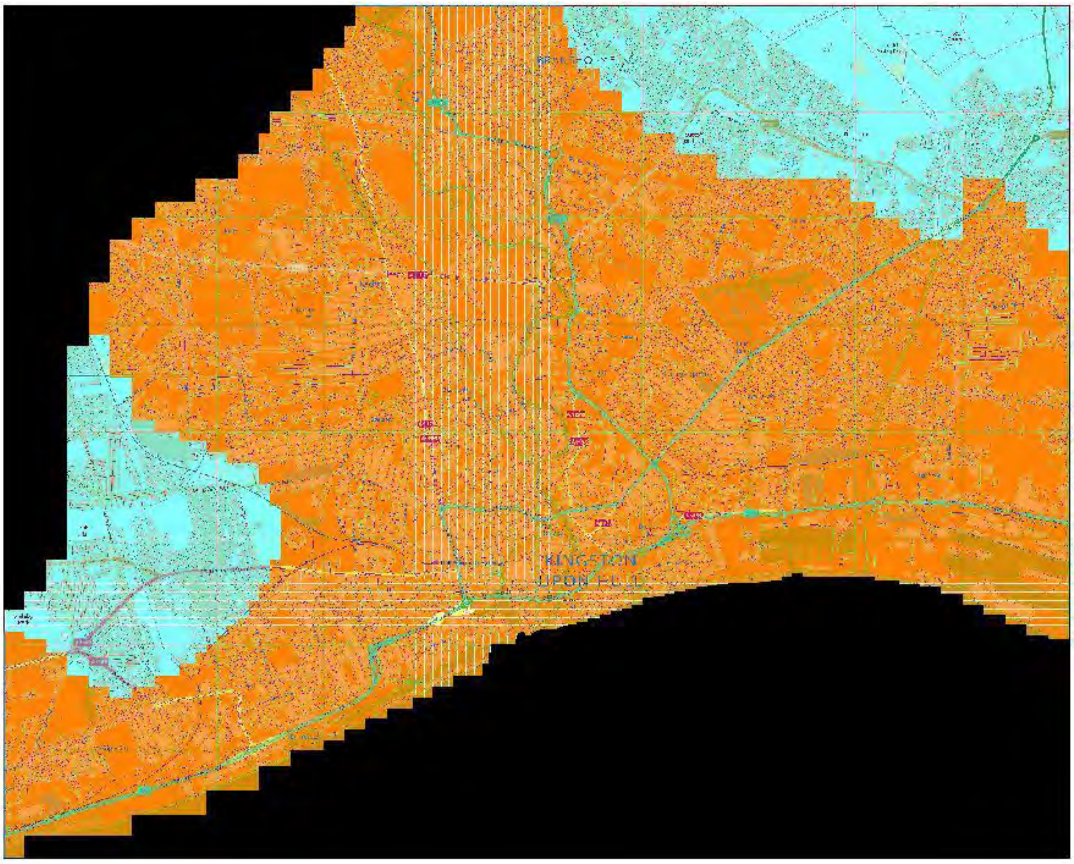


Figure 3-9 K/Storage Zones in Layer 4 (colours as in Figure 3-5)

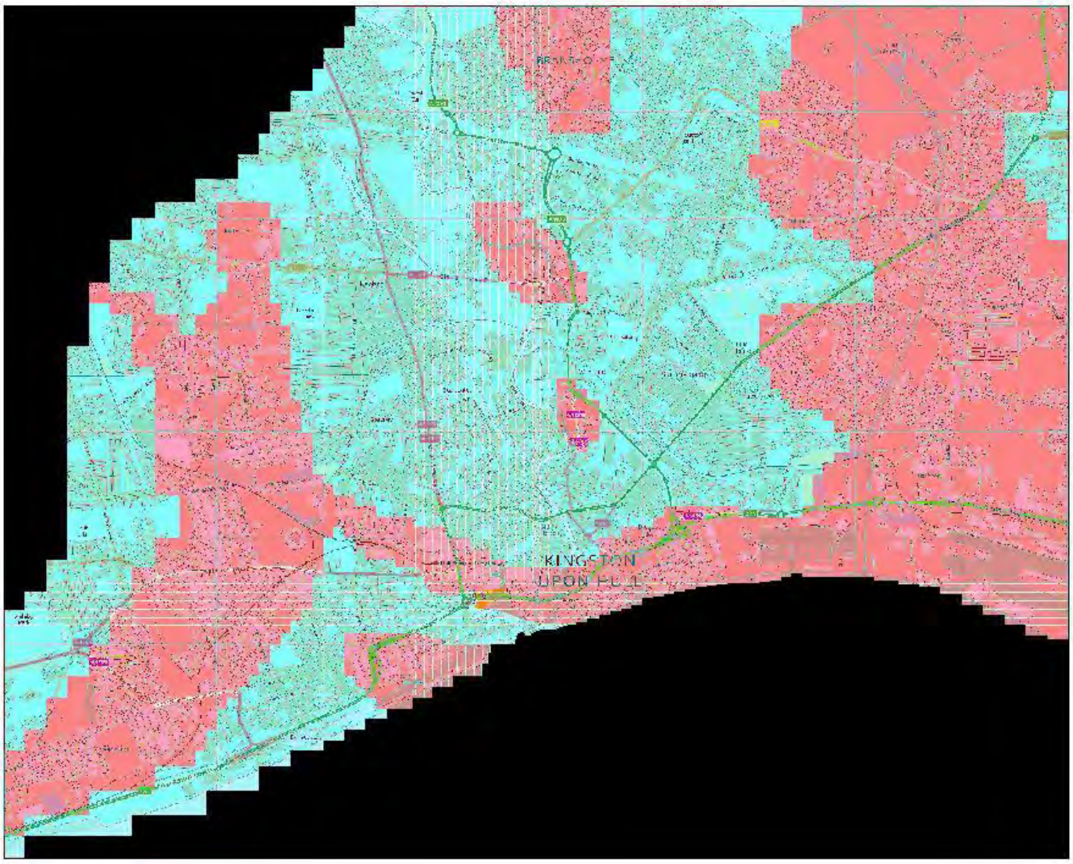


Figure 3-10 K/Storage Zones in Layer 5 (colours as in Figure 3-5)

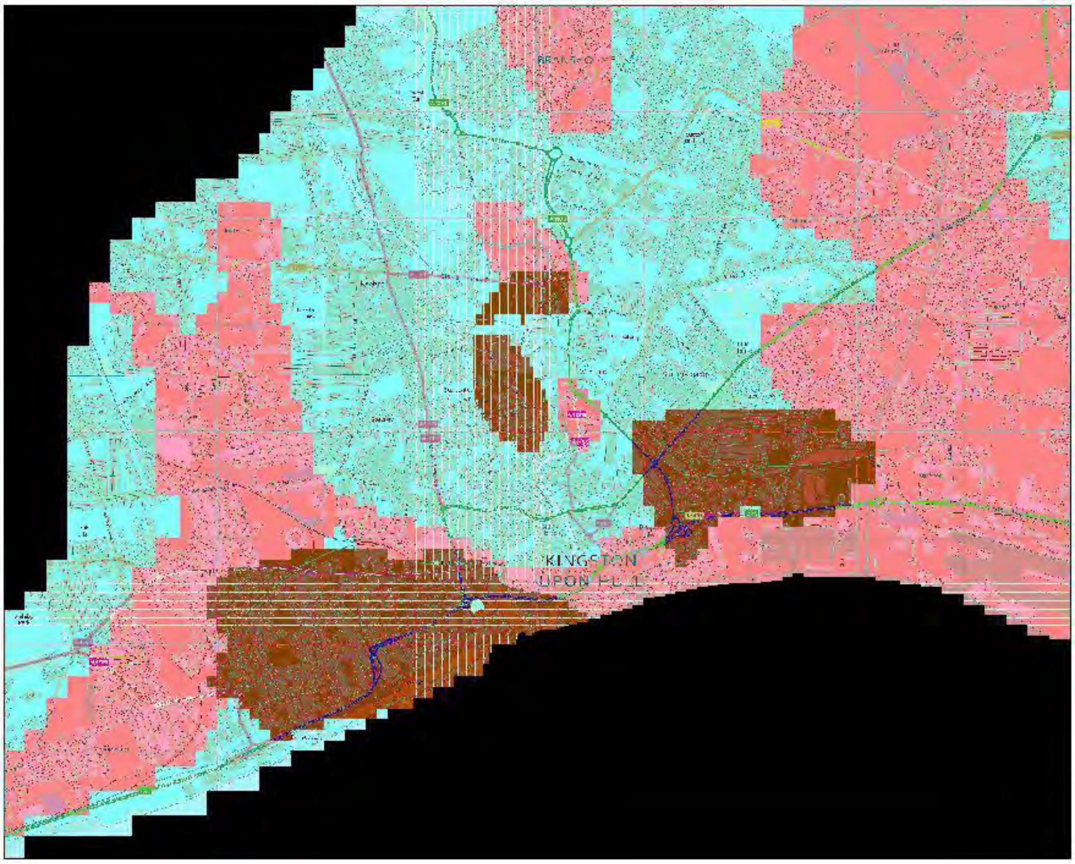


Figure 3-11 K/Storage Zones in Layer 6 (colours as in Figure 3-5)

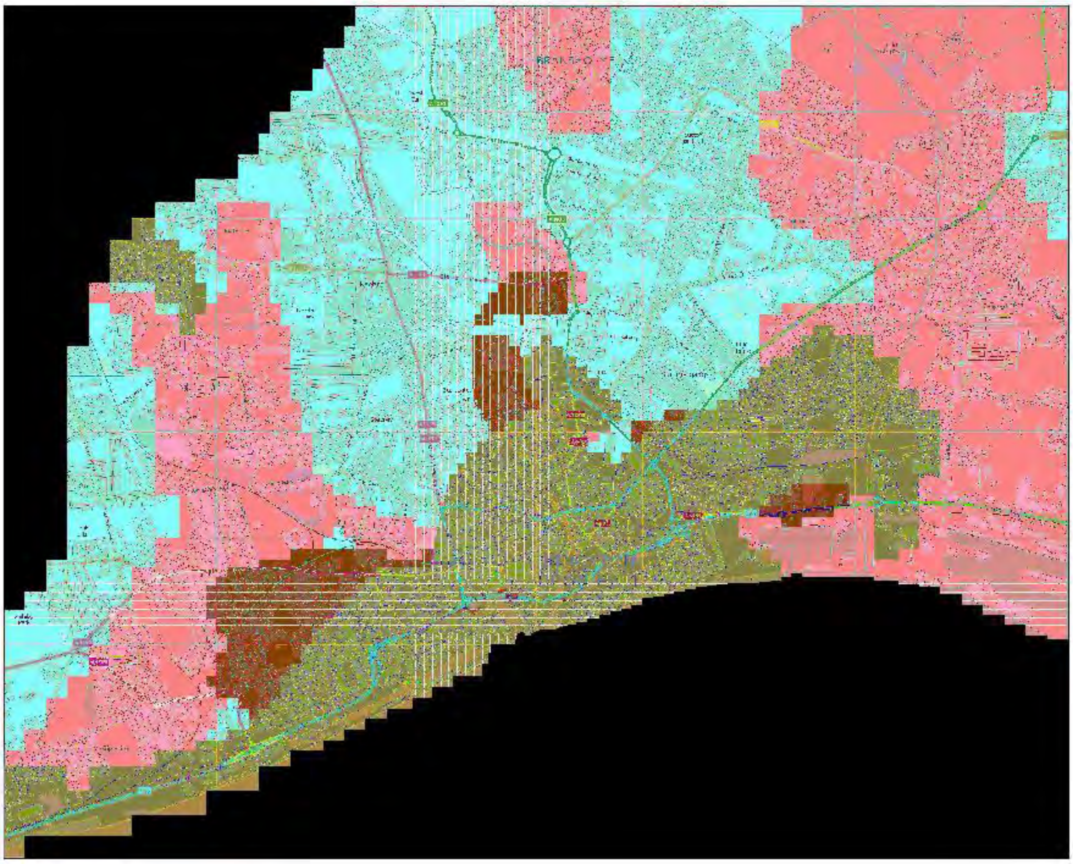


Figure 3-12 K/Storage Zones in Layers 7 and 8 (colours as in Figure 3-5)

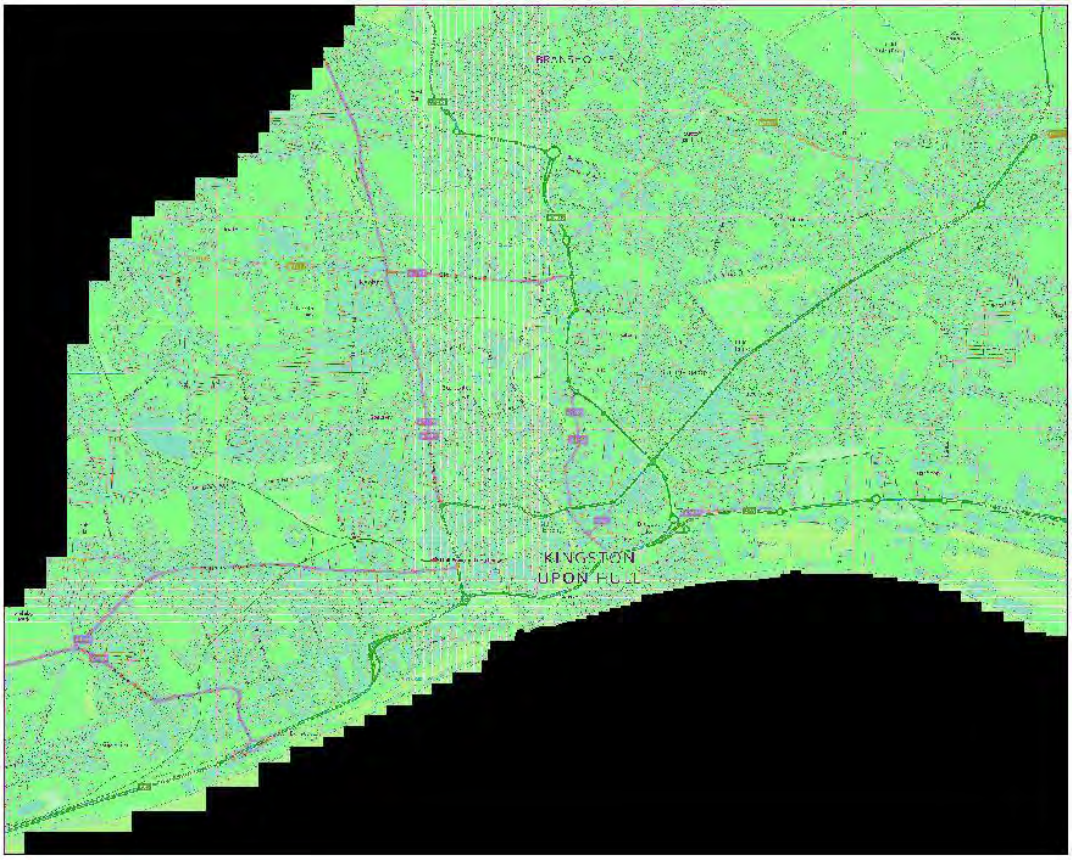


Figure 3-13 K/Storage Zones in Layer 9 (colours as in Figure 3-5)

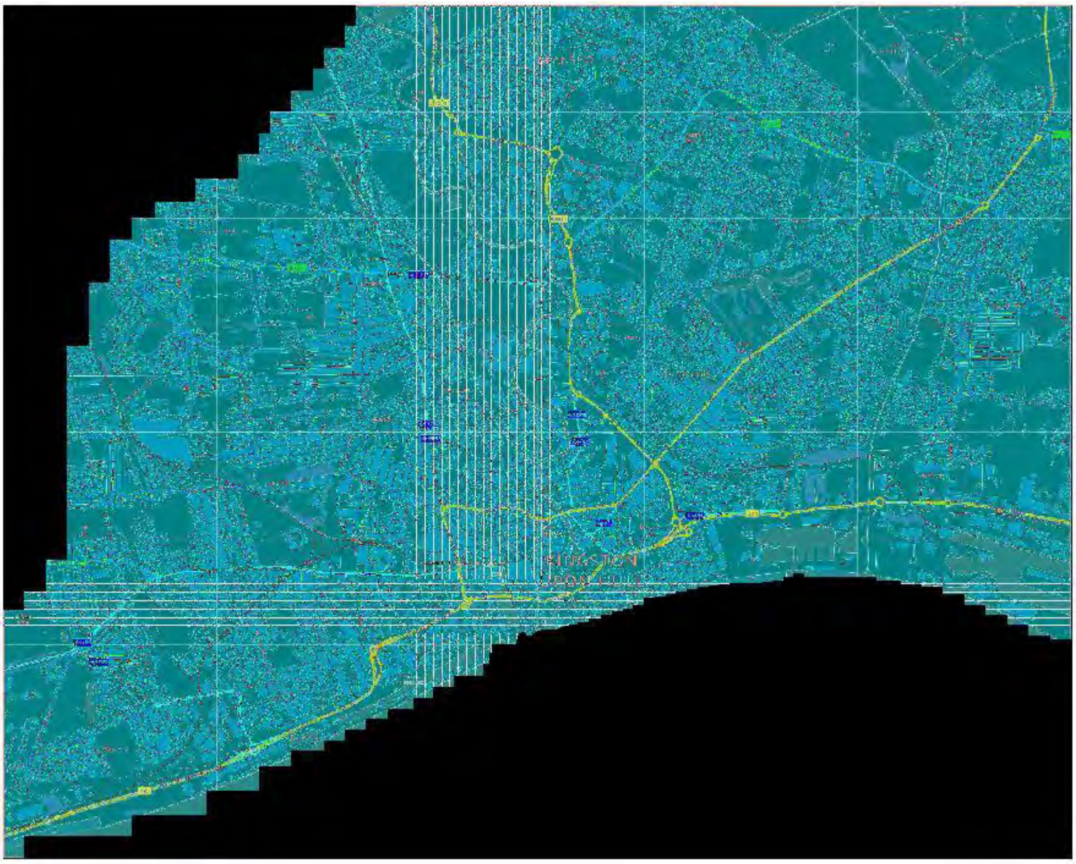


Table 3-1 Hydraulic Conductivity (K)

<table>
<tr><th>Zone</th><th>Hydraulic Testing (MMG JV)
Kxy
[m/d]</th><th>ESI Model
Kxy
[m/d]</th><th>Literature
Kxy
[m/d]</th><th>JBA Model
Kxy, Kz
[m/d]</th></tr>
<tr><td>1. Made Ground</td><td></td><td></td><td>0.001 - 0.1
[sand, silt, clay]</td><td>0.1, 0.001</td></tr>
<tr><td>2. Cohesive Alluvium</td><td>0.06 - 0.2
[3 tests]</td><td>0.01 - 0.1</td><td>0.0001 - 0.1
[silt and clay]</td><td>0.05, 0.005</td></tr>
<tr><td>3. Granular Alluvium</td><td>0.8 - 3.9
[16 tests]</td><td></td><td>0.1 - 100
[sand and gravel]</td><td>10, 0.1</td></tr>
<tr><td>4. Glacial Till</td><td>0.02 - 0.03
[4 tests]</td><td>0.001 - 0.01</td><td>0.00001 - 0.01
[boulder clay]</td><td>0.05, 0.05</td></tr>
<tr><td>5. Glaciolacustrine Deposits</td><td>0.02
[1 test]</td><td>0.001 - 0.01</td><td>0.00001 - 0.01
[silt and clay]</td><td>0.01, 0.001</td></tr>
<tr><td>6. Fluvioglacial Deposits</td><td></td><td>0.5 - 10</td><td>0.1 - 100
[sand and gravel]</td><td>1, 0.1</td></tr>
<tr><td>7. Uppermost Chalk</td><td rowspan="2">0.001 - 81.1 [pumping test at LDBH01 with three observation boreholes: BH18A, BH24 and BH29. Gave K = 69 to 81 m/d for an effective aquifer thickness of 20 m]</td><td rowspan="2">5 - 25</td><td></td><td>75, 0.75</td></tr>
<tr><td>8. Main Chalk</td><td></td><td>75, 0.75</td></tr>
<tr><td>9. Deep Chalk</td><td></td><td></td><td></td><td>0.01, 0.01</td></tr>
<tr><td>10. Additional zone to represent sandy Glacial Till</td><td></td><td></td><td></td><td>0.1, 0.01</td></tr>
</table>

Table 3-2 Storage and Confinement Properties

<table>
<tr><th>Zone
[Confinement]</th><th>MMG JV Pumping Test
S [-]</th><th>ESI Model
S [-]</th><th>ESI Model
Sy [-]</th><th>Literature*
Sy [-]</th><th>JBA Model
Ss [1/m]</th><th>JBA Model
Sy [-]</th></tr>
<tr><td>1. Made Ground
[UNCONFINED]</td><td></td><td></td><td></td><td>0.03 - 0.3
[clay to sand]</td><td>1×10^{-4}</td><td>0.1</td></tr>
<tr><td>2. Cohesive Alluvium
[UNCONFINED]
The dewatered excavation will penetrate into the cohesive alluvium.</td><td></td><td></td><td>0.02</td><td>0.03
[clay]</td><td>1×10^{-6}</td><td>0.03</td></tr>
<tr><td>3. Granular Alluvium
[CONFINED]</td><td></td><td></td><td>0.04</td><td>0.25
[sand]</td><td>1×10^{-6}</td><td>0.25</td></tr>
<tr><td>4. Glacial Till
[CONFINED]</td><td></td><td></td><td>0.001</td><td>0.03 - 0.1
[clay to silt]</td><td>5×10^{-4}</td><td>0.05</td></tr>
<tr><td>5. Glaciolacustrine Deposits
[CONFINED]</td><td></td><td></td><td>0.001</td><td>0.03
[clay]</td><td>5×10^{-4}</td><td>0.03</td></tr>
<tr><td>6. Fluvioglacial Deposits
[CONFINED]</td><td></td><td></td><td>0.04</td><td>0.25
[sand]</td><td>1×10^{-4}</td><td>0.25</td></tr>
<tr><td>7. Uppermost Chalk
[CONFINED]</td><td rowspan="2">0.0003 to 0.0015 Pumping test at LDBH01 with three observation boreholes: BH18A, BH24 and BH29.
The pumping test suggests a possible leaky confined response.</td><td rowspan="2">0.0001</td><td>0.01</td><td></td><td>5×10^{-6}</td><td>0.01</td></tr>
<tr><td>8. Main Chalk
[CONFINED]</td><td>0.01</td><td></td><td>5×10^{-6}</td><td>0.01</td></tr>
<tr><td>9. Deep Chalk
[CONFINED]</td><td></td><td></td><td>0.01</td><td></td><td>1×10^{-6}</td><td>0.01</td></tr>
<tr><td>10. Additional zone to represent sandy Glacial Till</td><td></td><td></td><td></td><td>0.1 - 0.25
[silt to sand]</td><td>1×10^{-4}</td><td>0.15</td></tr>
<tr><td colspan="7">Notes:

*Based on Brassington (2007)
Ss = Specific storage
S = Storativity
Sy = Specific yield
b = vertical thickness
[-] = no units (dimensionless)

Porosity values were assigned to allow calculation of average linear velocity. The following porosity values were based on Brassington (2007):
Zone 1 = 0.45, Zone 2 = 0.45, Zone 3 = 0.4, Zone 4 = 0.35, Zone 5 = 0.4, Zone 6 = 0.4, Zone 10 = 0.3. In addition, the Chalk was given a low effective porosity to reflect its dual porosity nature (with fractures providing most of the drainable storage: Zone 7 = 0.01, Zone 8 = 0.01, Zone 9 = 0.01.</td></tr>
</table>

3.10 Recharge

The JBA model assumes a spatially uniform recharge rate of 1.4 x 10^{-4} m/d, i.e. equal to the lower limit of the ESI (2013) range (Section 2.5.3). Most of the JBA model area is urban (with hard surfaces and drainage systems widespread), so it is not unrealistic to choose the lower end of the range (although mains water leakage can be a significant source of recharge in urban areas). The recharge rate was checked using sensitivity analysis (see Section 3.12). However, it should be noted that steady-state calibrations based on heads are potentially non-unique as the same calibration can be achieved using different combinations of recharge and hydraulic conductivity.

3.11 Boundary Conditions for Steady-state Simulation

Model boundary conditions may be classified as (Environment Agency, 2002):

- External boundary conditions - defined at the edges of the model domain
- Internal boundary conditions - defined within the model domain.

These may be further subdivided into:

- Specified head boundaries
- Specified flow boundaries
- Head-dependent flow boundaries.

At head-dependent flow boundaries the flow is proportional to the hydraulic gradient developed between the groundwater system and the boundary head; the proportionality constant is termed the conductance. In MODFLOW, head-dependent flow boundaries include River, Stream, Lake and General Head boundaries.

"Wall" boundaries in MODFLOW represent reduced conductance between individual cells. Each wall has a specified thickness and hydraulic conductivity. Walls are not true boundary conditions as they do not specify head or flow. The hydraulic conductivity must be greater than zero, so walls cannot be used to represent no-flow boundaries.

In the account that follows, the term "reach" refers to a labelled boundary, or portion of a boundary. This labelling allows MODFLOW to calculate flows for individual features such as a particular length of river or a particular edge of the model in a certain layer, etc.

3.11.1 External Boundary Conditions

The external boundary conditions specified for the model are summarised in Table 3-3.

No-Flow Boundaries

No-flow boundaries are specified flow boundaries with the flow set to zero. The edges of a MODFLOW model are no-flow by default, but No-Flow cells can also be defined within the grid itself. No-Flow cells were used to "turn off" those parts of the model grid that were not required as part of the flow model. This allowed for a realistic representation of the plan view geometry (rather than just a rectangle).

In the Chalk (Layers 7, 8 and 9), the western boundary was taken as a no-flow boundary coinciding approximately with a groundwater divide separating the modelled area from an area of heavy groundwater abstraction to the west. The shape of the boundary was based on Source Protection Zone (SPZ) boundaries for major public water supply abstractions located immediately to the west of the modelled area (outlines from Environment Agency website; SPZ3 was used as this represents the total catchment for a borehole).

Short sections of no-flow boundary occur along the edges of the upper layers of the model wherever a general head boundary (see below) would have given a head below the base of the layer concerned. In these locations no boundary condition was assigned, so MODFLOW used its default setting of no-flow. These short sections of no-flow boundary are distant from the area of interest and are present only in the upper layers, so will not have had a significant impact on the model results.

General Head Boundaries

General head boundaries are head-dependent flow boundaries in which an external head is specified a certain distance from the edge of the model. The conductance, C, is specified as C = Kws/L, where K = hydraulic conductivity, w = cell width, s = saturated thickness and L = distance to the boundary. If flows out of the model are taken as negative then the volumetric flux, Q, across the boundary is given by Q = C([external "general" head] - [calculated groundwater head in boundary cell]).

General head boundaries were used to allow groundwater flow into, or out of, the northern, eastern and western boundaries of the model. The specified heads were based on approximate groundwater (or stream) levels in the surrounding area, and hydraulic conductivities were selected so as to represent approximately the ground materials present between the edge of the model and the location at which the head was specified. Details of individual boundaries are provided in Table 3-3.

Rivers

A river boundary represents flow between the groundwater system and a river. It is assumed that the river is separated from the aquifer by a layer of river bed sediment. The river boundary condition includes a river stage (head) and a conductance that reflects the area of the river bed, the thickness of river bed sediment, and the hydraulic conductivity of the river bed sediment. One of two equations is used to calculate the flow between the river and groundwater, depending on whether the river is "perched" above the water table (Anderson and Woessner, 2002):

$$QRIV = CRIV(HRIV - h) \quad \text{for } h > RBOT$$

$$QRIV = CRIV(HRIV - RBOT) \quad \text{for } h \leq RBOT \text{ ("perched" condition)}$$

where QRIV = flow between river and groundwater (negative for rivers that remove water from the groundwater system; positive for rivers that add water to the groundwater system), CRIV = river bed conductance, HRIV = head in river, h = head in aquifer and RBOT is the elevation of the river bed minus the thickness of river bed sediment. CRIV is given by CRIV = (KLW)/M where K = hydraulic conductivity of river bed, length = length of river channel, w = width of river channel and M = thickness of river bed sediment.

A river boundary was used to represent the Humber Estuary, with stage = 0 mAOD, bottom elevation = -16.46 mAOD (depth of 9 fathoms - ESI, 2013), river bed thickness = 1 m and K = 1 m/d. The bed thickness and K were chosen not to be realistic in themselves, but to give a conductance similar to that used in the ESI (2013) regional model (100 m^2/d). Note that the conductance is high compared to the conductivities of the near-surface model layers. This means that in the model the degree of interaction between the estuary and groundwater will be determined by the hydraulic conductivity of the ground rather than by the riverbed conductance.

3.11.2 Internal Boundary Conditions

Rivers

River cells were used to represent the River Hull. Two reaches were defined:

Reach 1: Humber Estuary (stage 0 mAOD) to High Flaggs gauging station (stage 0.61 mAOD)

Reach 2: High Flaggs to northern edge of model (stage 1 mAOD based on topographic elevation)

The river was set as 30 m wide, with a 1 m thick river bed having K = 1 m/d. As with the Humber river boundary, the conductance is high compared to the conductivity of the upper model layers (so the conductivity of the model layers will control the degree of modelled aquifer - river interaction).

Drains

Drain boundaries are similar to river boundaries, but they can only remove water from the model. If the groundwater head falls below the stage of the drain then the drain ceases to flow. Drains were used to represent two minor watercourses that cross the model area: Beverley and

Barmston Drain in the west, and Ganstead/Holderness Drain in the east. In both cases a 1 m thick bed with a hydraulic conductivity of 0.1 m/d was assumed. Heads were set to vary linearly between the end-points as follows:

> *Beverley and Barmston Drain:* 1.5 mAOD upstream (based on topographic elevation); 0.65 mAOD downstream (reflecting the stage of the River Hull).
>
> *Ganstead/Holderness Drain:* 1.5 mAOD upstream (based on topographic elevation); 0 mAOD downstream (reflecting the assumed average water level in the Humber Estuary).

Wells

Wells (as "analytic elements" in MODFLOW) were used to represent groundwater abstractions within the model area. The abstractions represented are listed in Table 3-4. All were represented in Layer 8 (Main Chalk layer).

3.11.3 A Note on the Docks

It was agreed with MMG JV that the docks would not be represented in the model. The docks have walls, so are likely to be largely isolated from the groundwater system, except potentially through their bases (and through the walls if these leak). If the docks are well sealed then they may represent local barriers to shallow groundwater flow, and could be modelled as zones of low hydraulic conductivity. If the docks do interact significantly with groundwater (perhaps being a source of water during low tide and a sink at high tide) then they could be modelled as river cells. Neither of these approaches was taken here. The docks were simply omitted from the model, and the dock areas were assigned the same properties as the surrounding ground.

If the docks act as low permeability barriers then there may be "image well" effects, with greater drawdowns adjacent to the barriers (reflecting an absence of water supply from the barrier, which acts as a no-flow boundary). However, modelled drawdowns in the vicinity of the docks are relatively small (<0.05 m in both the construction and operation scenarios - see Section 4.5 and Section 4.7), so the influence of any barrier effect is unlikely to be very significant.

Figure 3-14 Boundary Conditions in Layer 1

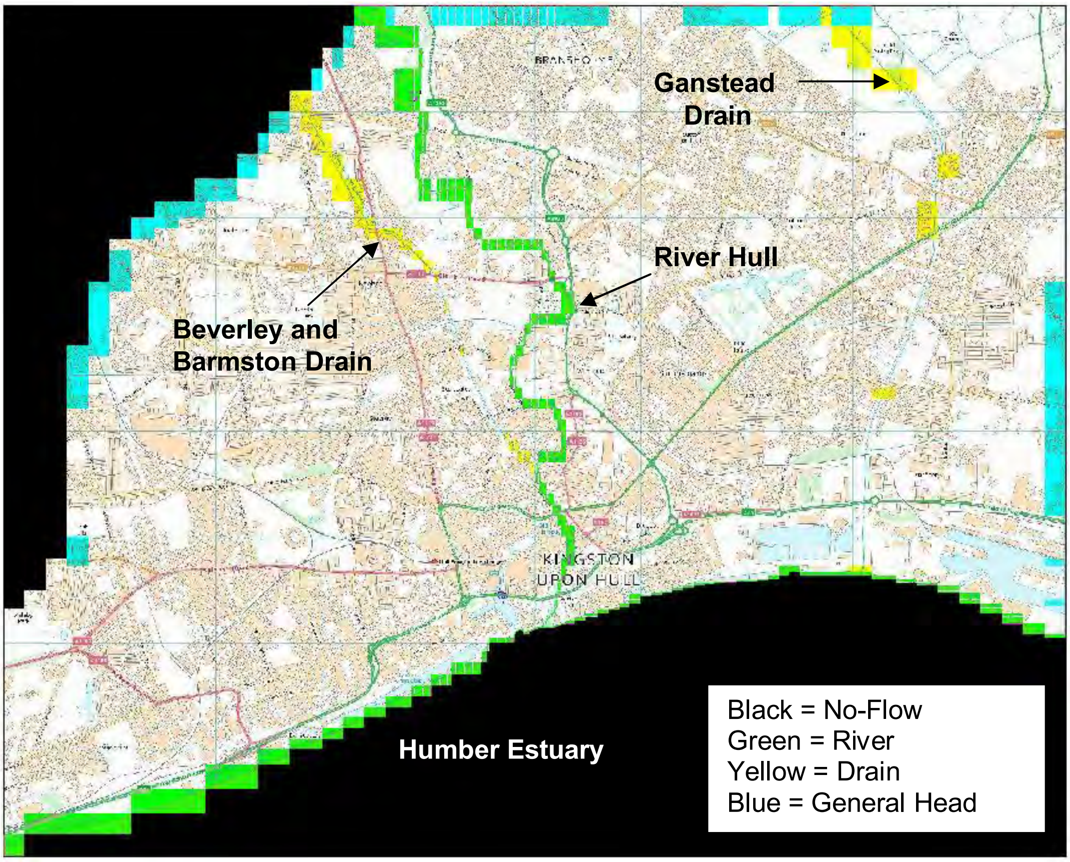


Figure 3-15 Boundary Conditions in Layer 2 (colours as in Figure 3-14)

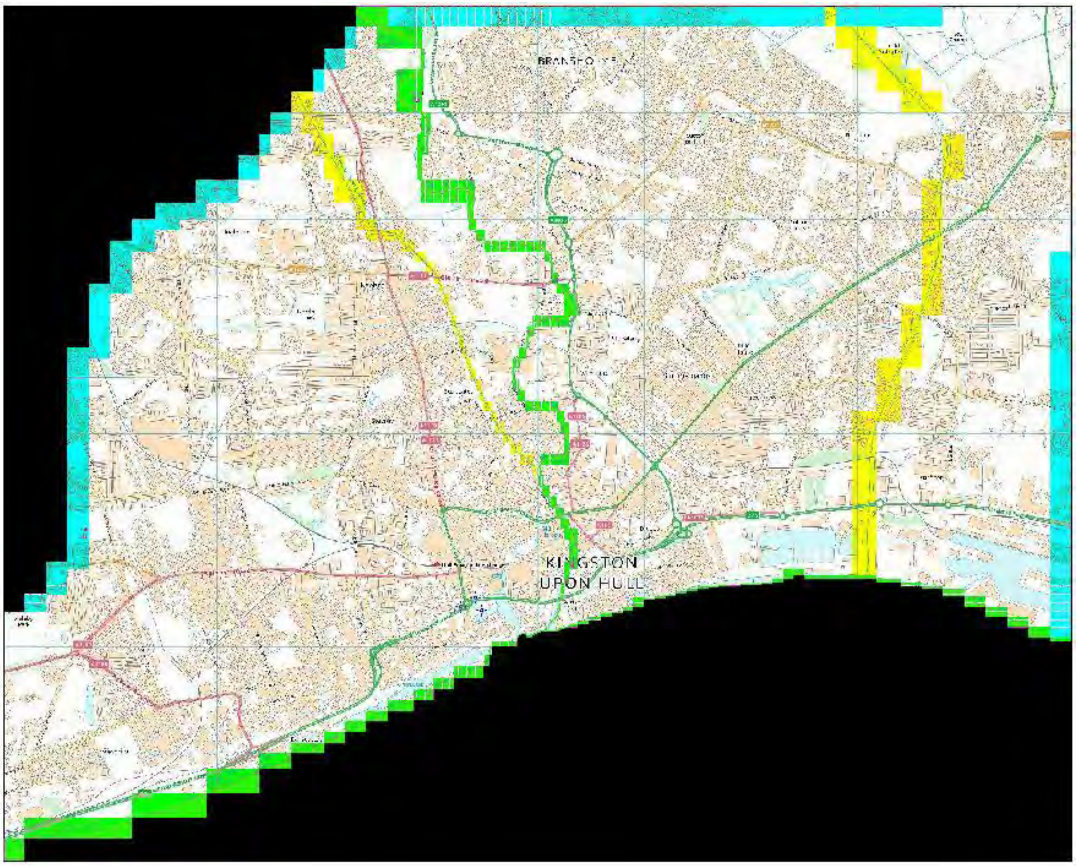


Figure 3-16 Boundary Conditions in Layer 3 (colours as in Figure 3-14)

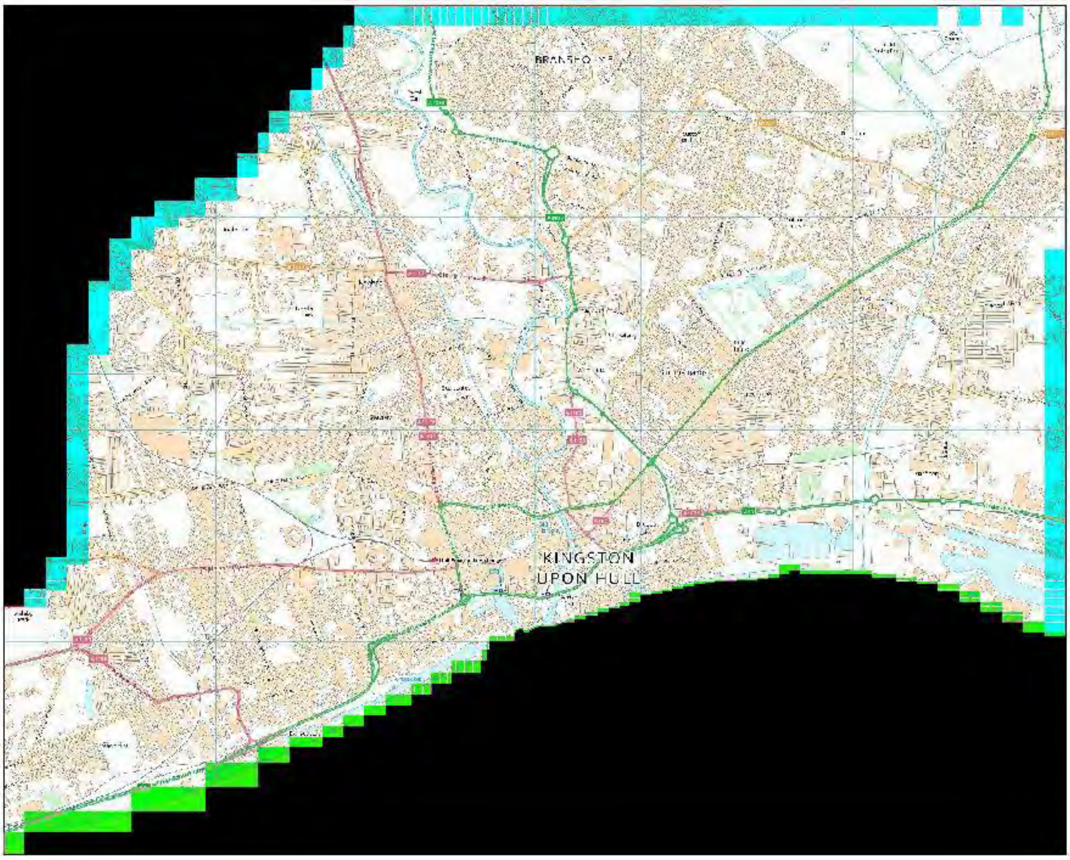


Figure 3-17 Boundary Conditions in Layers 4, 5 and 6 (colours as in Figure 3-14)

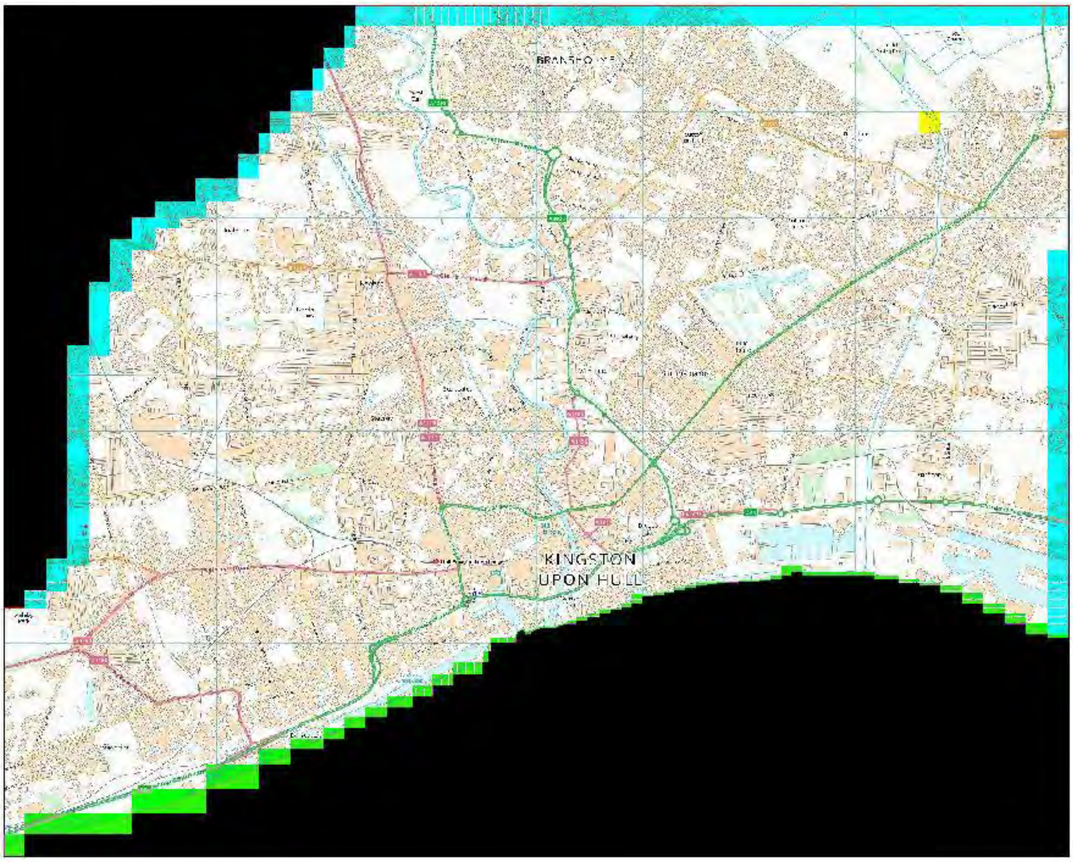


Figure 3-18 Boundary Conditions in Layers 7 and 8 (colours as in Figure 3-14)

Figure 3-19 Boundary Conditions in Layer 9 (colours as in Figure 3-14)

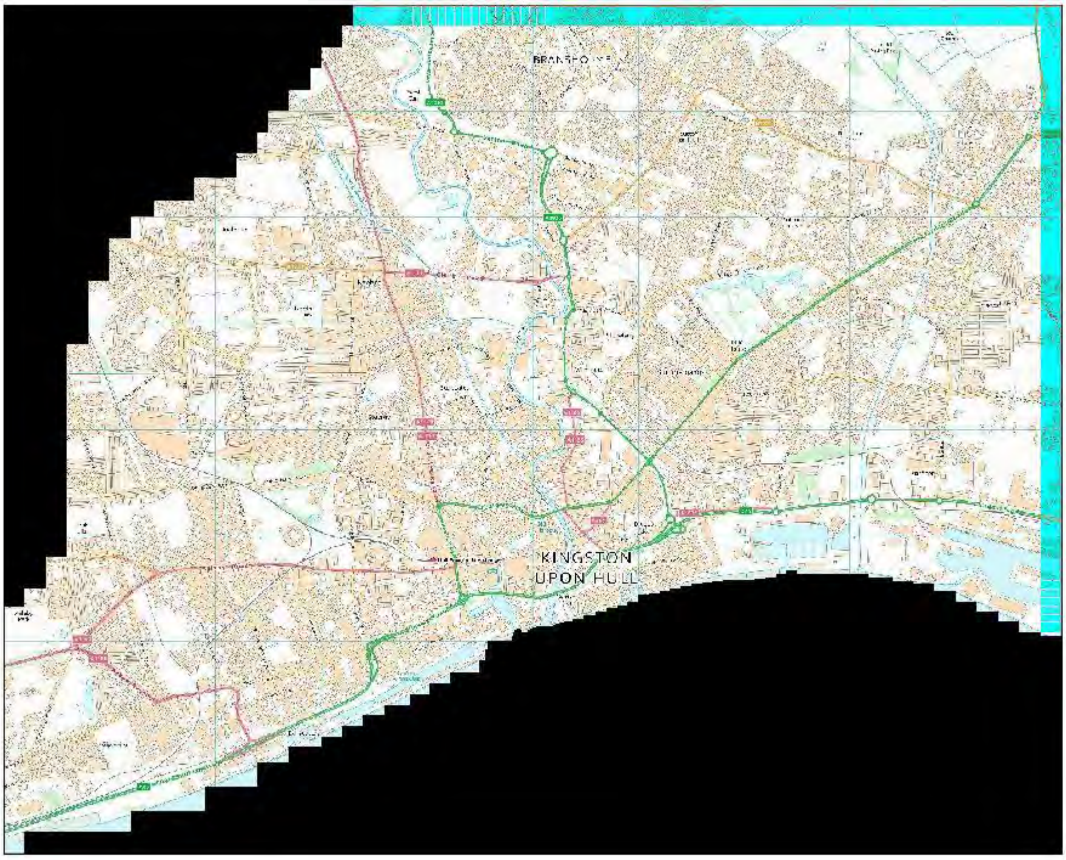


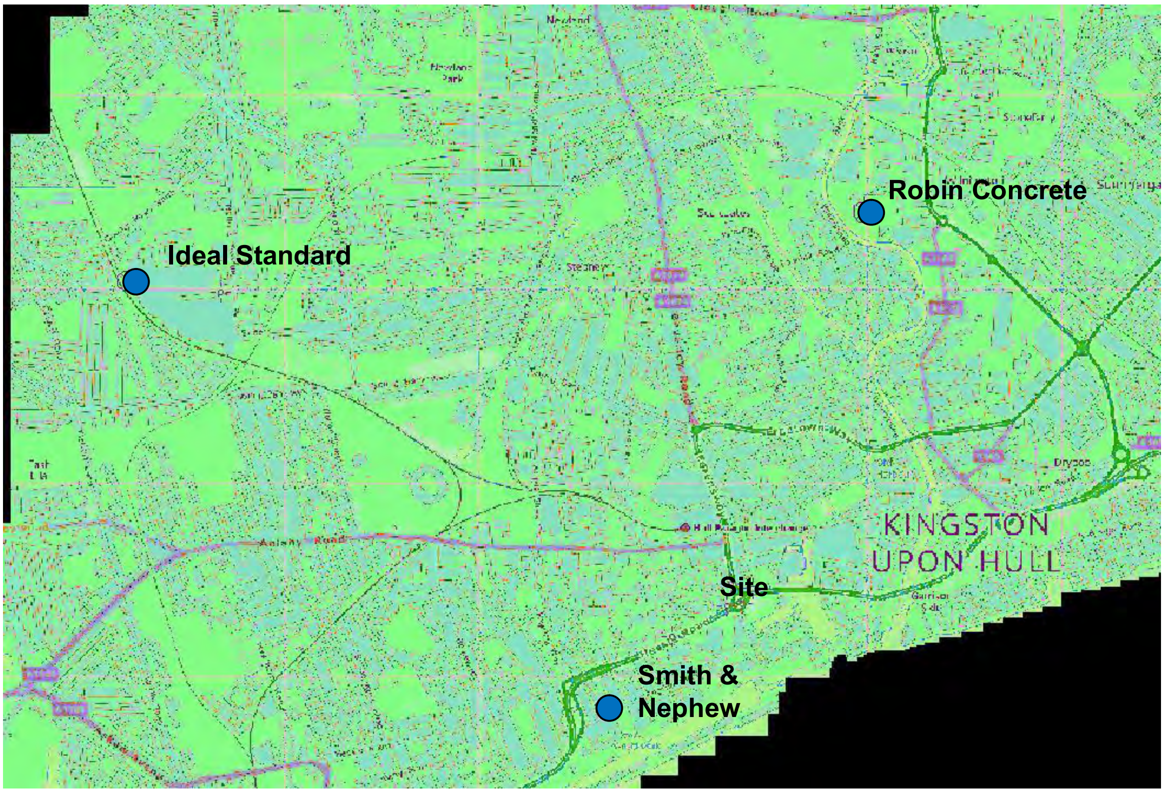
Table 3-3 External Boundary Conditions

Location	Layer(s)	Boundary Type	Explanation
Southern boundary	Superficial deposits (Layers 1 - 6)	River (Reach 0)	Represents the Humber Estuary. Stage = 0 mAOD; bottom elevation = -16.46 mAOD (ESI, 2010); bed thickness = 1 m; K = 1 m/d. Conductance of the order of 10 to 100 m^2/d (ESI, 2013, assumed 100 m^2/d).
	Chalk (Layers 7 and 8)	River (Reach 0)	Represents the Humber Estuary (parameters same as above).
	Chalk (Layer 9)	No-Flow	River Hull assumed to be in contact only with active Chalk (Layers 7 and 8).
Western boundary	Superficial deposits (Layers 1 - 6)	General Head (Reach 0) (locally set to no-flow if General Head boundary head below layer base)	Allows flow across the western boundary within the superficial deposits. Head = 1 mAOD at a distance of 500 m with K = 0.1 m/d and saturated thickness = 1 m.
	Chalk (Layers 7,8,9)	No-flow	Represents a groundwater divide coinciding with the catchment boundaries of major groundwater abstractions to the west of the modelled area.
Northern boundary	Superficial deposits (Layers 1 - 6)	General Head (Reaches 1, 2 and 3) Locally Drain or River where a watercourse flows into the model area - see description of internal boundary conditions; locally no-flow where "general head" below layer base.	Allows flow across the northern boundary. Generally, head = 1 mAOD at a distance of 500 m with K = 0.1 m/d. Saturated thickness = 1 m. Locally head = 5 mAOD at a distance of 100 m with K = 0.1 m/d and saturated thickness = 2 m (reflecting the location of a hill along the central part of the boundary).
	Chalk (Layer 7)	General Head (Reach 7)	Allows flow across the northern boundary. Same as for underlying layers but with saturated thickness = 4 m.
	Chalk (Layers 8 and 9)	General Head (Reach 7)	Allows flow across the northern boundary. Head = 0 mAOD at a distance of 10,000 m (based on groundwater contours in ESI, 2013); K = 20 m/d and saturated thickness = 20 m.
Eastern boundary	Superficial deposits (Layers 1 - 6)	No-flow (in north)	An approximate flow line orientated roughly at right-angles to drains/streams immediately east of the modelled area.
		General Head (Reaches 4, 5 and 6) Locally set to no-flow if General Head boundary head below layer base.	Represents the influence of a stream, Old Fleet, located to the east of the modelled area. Head = 3 m AOD at a distance equal to that of Old Fleet from the model boundary. K = 0.1 m/d.
	Chalk (Layer 7)	General Head (Reach 8)	Allows flow across the eastern boundary. Same as for underlying layers but with saturated thickness = 4 m.
	Chalk (Layers 8 and 9)	General Head (Reach 8)	Allows flow across the eastern boundary. Head = 2 mAOD at a distance of 8,000 m. K = 20 m/d and saturated thickness = 20 m. Chosen to represent, very approximately, an assumed groundwater divide under the Holderness Peninsula. Choice of head informed by topography and groundwater contours of ESI (2013).

Table 3-4 Licensed Abstractions in the Model Area

Easting	Northing	Licence	Name	Annual Quantity [m^3/yr]	Modelled Rate [m3/d]
508660	427850	2/26/32/049	Smith and Nephew Medical Ltd	221686	607.3589041
506200	430050	2/26/32/059	Ideal Standard Manufacturing (UK) Ltd	700 (daily rate)	700
508950	429200	2/26/32/423	Hull Truck Theatre Co Ltd	33600	0 [as water returned to aquifer as part of an open-loop ground source heat scheme]
509981	430400	NE/026/0032/038	Robin Concrete and Waste Disposal Ltd	3500	9.589041096
Notes: Data supplied by the Environment Agency.					

Figure 3-20 Abstraction Boreholes (Layer 8)



3.12 Sensitivity Analysis and Steady-state Calibration

Calibration was undertaken using 20 head targets (Figure 3-21 and Table 3-5). Most were average groundwater levels observed in monitoring boreholes in January 2014. Two were older, dating from July 2004 and December 2005. It should be noted that even the January 2014 data were collected over different weeks as data loggers were moved between boreholes, so the averages are not necessarily representative of the entire month. The logger data contain different numbers of tidal fluctuations and this too is likely to affect the averages. It should also be noted that January 2014 was relatively wet, so the groundwater level targets may be fairly high. The boreholes were chosen so that (where possible) there were multiple head targets in each layer, allowing calibration of the horizontal hydraulic gradient.

Calibration was undertaken manually by varying the model parameters (hydraulic conductivity, recharge, boundary conditions) until an acceptable match was achieved between modelled and observed heads at the target locations. The success of calibration was assessed using the Sum of Squared Residuals (SSR), where the residual is defined as the observed head minus the modelled head.

Sensitivity analysis was undertaken to aid the calibration process. This involved varying a single parameter at a time, running the model, and calculating the SSR. The process was automated using Groundwater Vistas' Autosensitivity tool; this applied a set of pre-specified multipliers to the parameter value and ran the model for each multiplier (see Appendix A).

Sensitivity analysis identified which parameters were most important for calibration. The following parameters were tested:

- Kxy (multipliers x0.01, x0.1, x0.25, x0.5, x1, x2, x5, x10, x100)
- Kz (x0.01, x0.1, x0.25, x0.5, x1, x2, x5, x10, x100)
- Recharge (x0.1, x0.25, x0.5, x1, x2, x5, x10, x20, x50)
- Drain conductance (all drains together) (x0.01, x0.1, x0.25, x0.5, x1, x2, x5, x10, x100)
- River Humber conductance (x0.01, x0.1, x0.25, x0.5, x1, x2, x5, x10, x100)
- River Hull conductance (x0.01, x0.1, x0.25, x0.5, x1, x2, x5, x10, x100)
- General head boundary conductance (x0.01, x0.1, x0.25, x0.5, x1, x2, x5, x10, x100).

Note that no formal analysis of sensitivity to storage was undertaken during the transient calibration (Section 3.16).

Parameter sensitivity in the steady-state model was assessed as follows:

Very high sensitivity (variation in SSR > 5)

Kz Layer 2 and Layer 5

Recharge

River Humber conductance (only at lowermost end, otherwise insensitive)

High sensitivity (variation in SSR 1 to 5)

Kxy Layers 3 and 8

Kz Layers 1, 3, 4 and 10.

Moderate sensitivity (variation in SSR 0.5 to 1)

Kxy Layers 2, 7 and 10

Low sensitivity (variation in SSR < 0.5)

Kxy Layers 1, 4, 5, 6 and 9.Kz Layers 6, 7, 8 and 9.

Drain conductance

River Hull conductance

General head boundary conductance

Overall the calibration was found to be most sensitive to recharge, the vertical hydraulic conductivity of the main aquitards and the horizontal hydraulic conductivity of the main aquifer layers. It was not very sensitive to the boundary conductances. This is likely to be due to the fact that the boundary conductances are in general set much higher than the hydraulic conductivity of the layers. Hence the connection with the boundary cells is very good, and any flow to/from the boundary cell is mainly determined by the hydraulic conductivity of the adjacent aquifer layers, not the conductance of the boundary condition. Another factor may be the low hydraulic gradients, meaning that boundary flows remain low even if boundary conductances are set to be relatively high. In general the calibration was not very sensitive to the conductance of the River Humber. However, when the Humber conductance was reduced to something more similar to that of the aquifer hydraulic conductivity then the sensitivity increased.

The final calibration had a RSS value of 5.62. A better calibration (RSS = 4.14) was achieved with slightly different K values, but these were revised during transient calibration (see Section 3.16). The somewhat worse steady calibration was accepted on the following grounds:

- The two models (steady and transient) need to have consistent K values.
- The steady targets are averages from different time periods and may not be representative, whereas the transient calibration is based on actual measured heads for a specific time interval.
- The steady case has very low hydraulic gradients, making accurate calibration difficult. The transient case represents tidal fluctuations, which give higher hydraulic gradients that are easier to calibrate too.
- The transient calibration is sensitive to K as this affects the rate at which a tidal fluctuation propagates through the model.

In short, the transient calibration of K was preferred over the steady one.

Figure 3-22 summarises the steady-state calibration by plotting model-generated heads against observed heads. Figure 3-23 to Figure 3-29 show the target residuals for each layer; in these plots, red values are too high (compared to the targets) and blue values too low. Note that the groundwater contour labels are also blue.

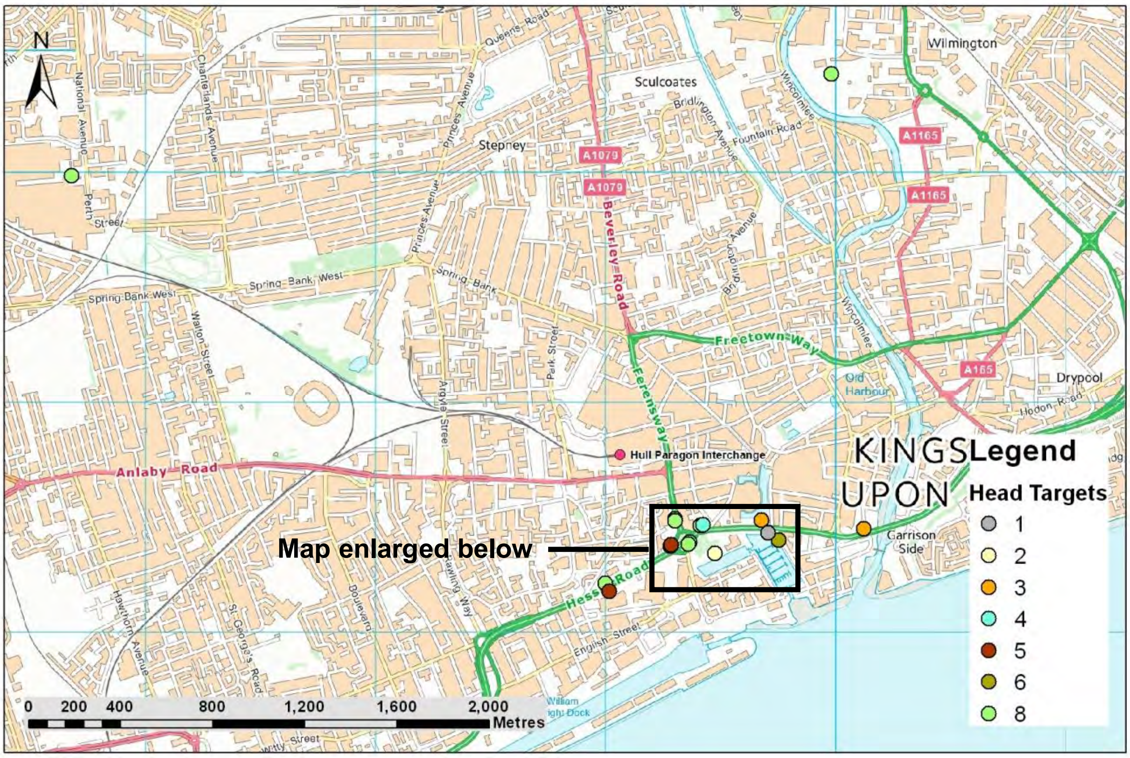
Layer 1 (Made Ground) is dry across much of the site (Figure 3-23); this is consistent with the site investigation data, which recorded most first water strikes within the underlying alluvium (Layer 2). The absence of widespread "flooding" of cells in Layers 1 and 2 is also an indication that modelled groundwater levels in the superficial deposits are likely to be fairly realistic.

Modelled heads in Layer 1 are significantly lower than the target value. However, it is likely that groundwater in the made ground is perched, and has a complex distribution. This state of affairs cannot be represented properly in a saturated flow model.

In general, targets in the other layers were matched reasonably well (allowing for the fact that adjacent targets with a large head difference could only be matched "on average", with one being too high and the other too low). The exception was Layer 3 (Granular Alluvium), where the easternmost target, close to the River Hull, could not be matched at all, even with quite large changes in parameter values. This target is very close to the River Hull and may be affected by tidal fluctuations in river level. The target residuals for Layer 3 increase eastwards, indicating that the hydraulic gradient in this layer does not match that implied by the target heads (Figure 3-25). This gives a negative correlation between modelled head and target head in Figure 3-22.

The target groundwater levels vary over a fairly narrow range of elevation, and some variation appears to reflect local conditions (e.g. perched groundwater in Layer 1). The lower target elevations are relatively well matched by the model, but the higher targets are not as well matched. Overall, the model roughly approximates the target heads in the vicinity of the site, but does not reflect details related to local conditions.

Figure 3-21 Locations of Head Targets for Steady-state Calibration (numbers in legend are later numbers)



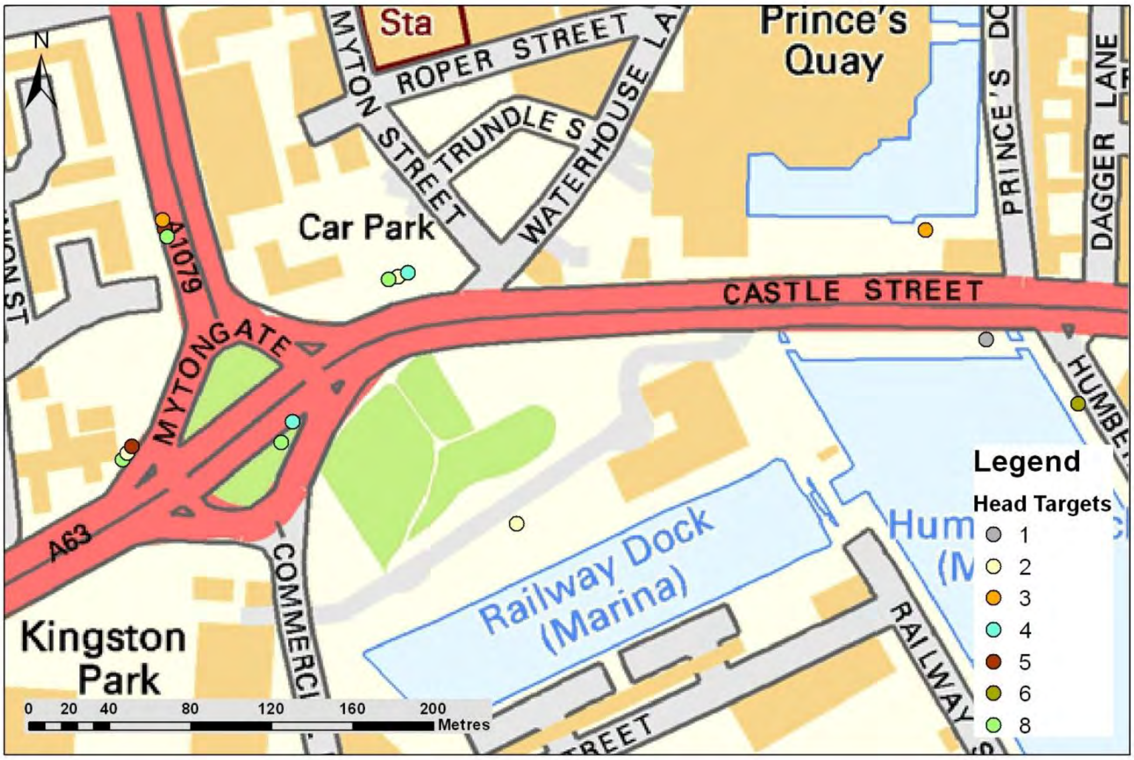


Table 3-5 Steady-state Head Targets

Easting	Northing	Name	Target (mAOD)	Layer
508996.05	428213.669	BH02	0.25	8
509014.973	428176.986	BH03	0.58	5
509279.102	428374.455	BH11	0.27	8
509281.379	428377.592	BH12	1.26	2
509283.484	428380.985	BH13	0.7	4
509362.953	428393.089	BH15	0.51	4
509357.222	428383.089	BH18A	0.16	8
509299.636	428488.775	BH20	-0.11	5
509298.664	428492.972	BH21	0.55	3
509300.893	428484.717	BH22	0.04	8
509473.781	428342.647	BH30	0.6	2
509415.07	428465.029	BH32	0.44	2
509410.215	428463.744	BH33	0.0003	8
509419.888	428467.012	BH34	0.25	3
509675.594	428487.895	BH38	0.68	3
510120.053	428449.443	BH46	1.07	3
509751.034	428402.2	BH42	0.23	6
509705.575	428434.058	BH41A	2.69	1
509980	430430	DJ Broady July 2004 level Target very close to Robin Concrete abstraction, but abstraction is small.	0.4	8
506677	429986	"Dec2005" December 2005 level (from ESI, 2013)	0.3	8

Figure 3-22 Graphical Summary of Steady-state Calibration (values in mAOD)

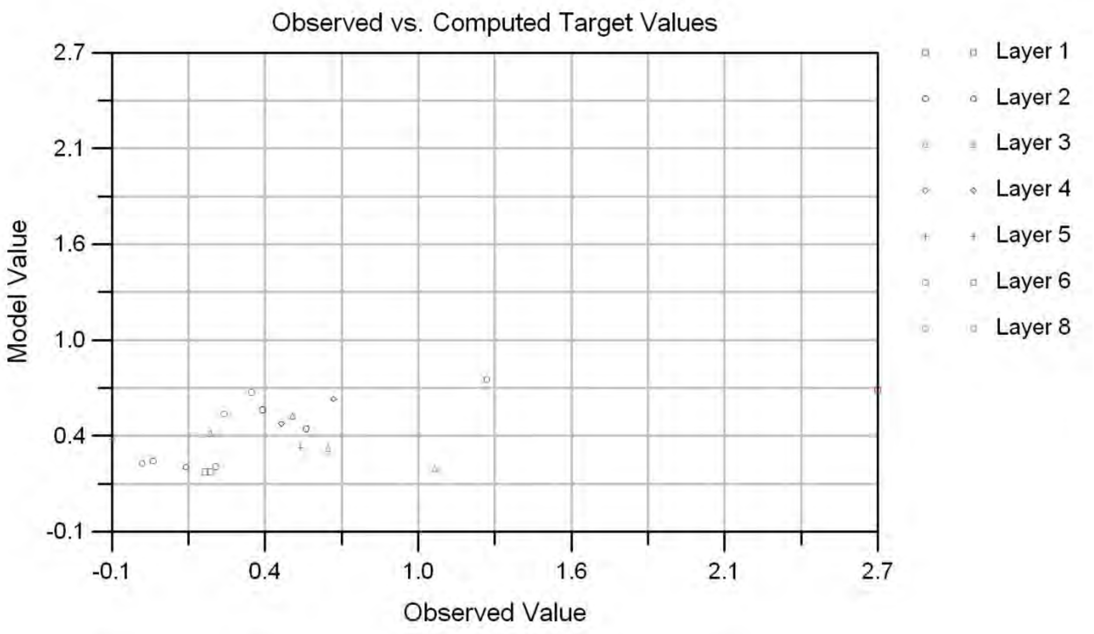


Figure 3-23 Target Residuals in Layer 1 (Made Ground) (figure zoomed into site area)

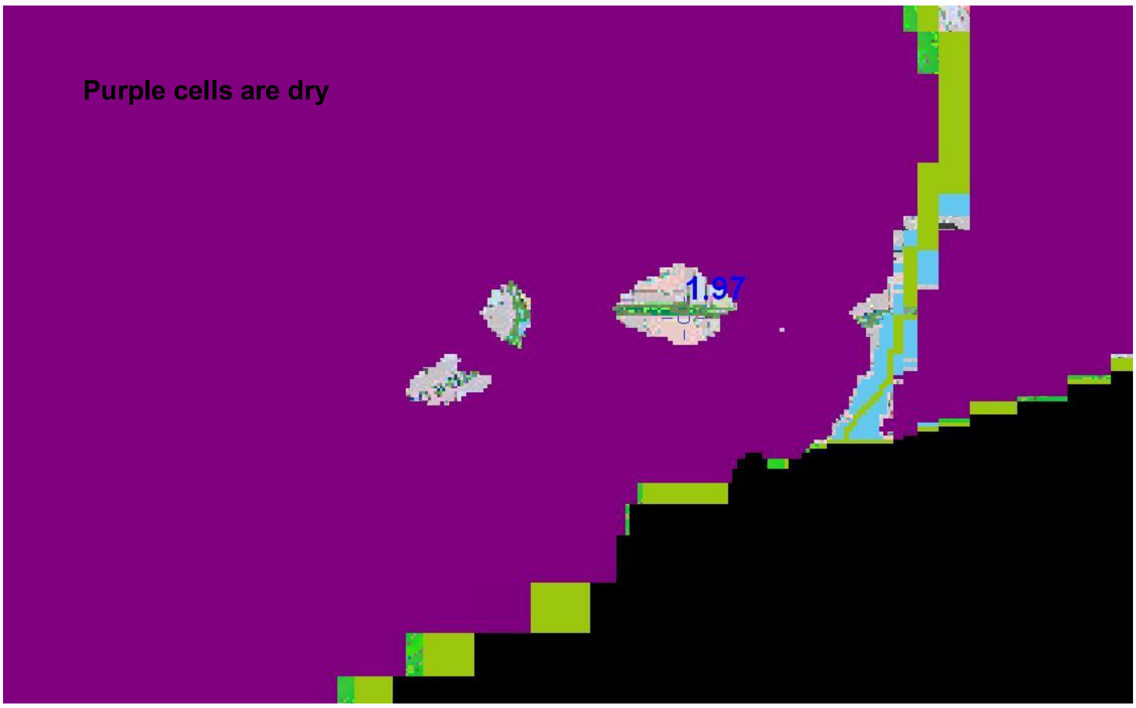


Figure 3-24 Target Residuals in Layer 2 (Cohesive Alluvium)

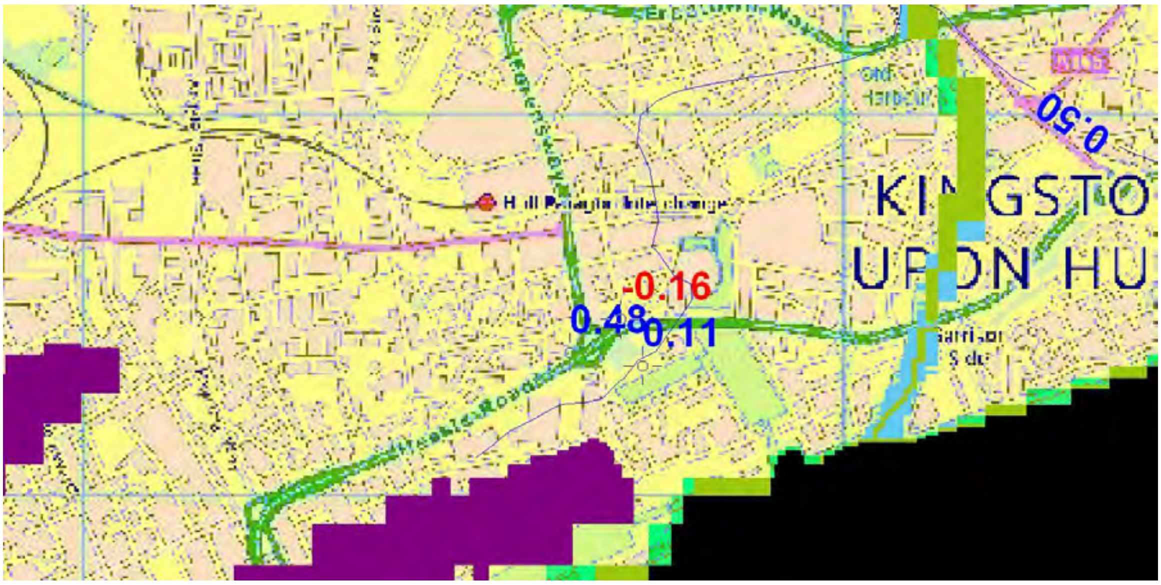


Figure 3-25 Target Residuals in Layer 3 (Granular Alluvium)

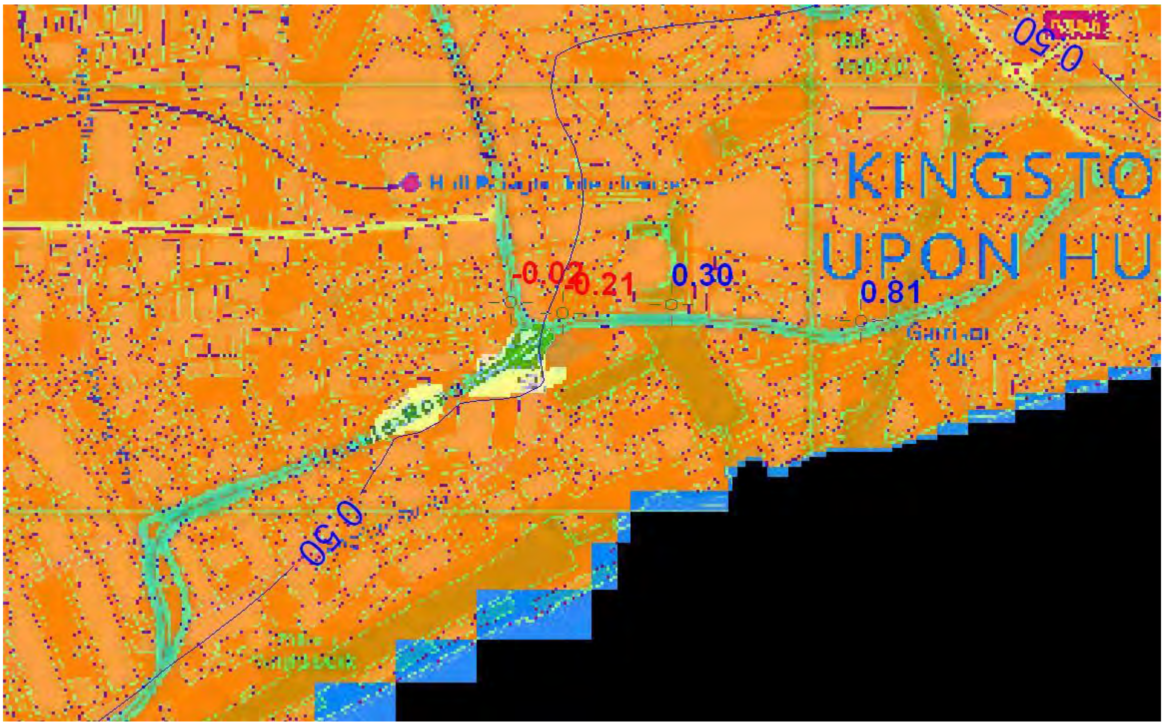


Figure 3-26 Target Residuals in Layer 4 (Glacial Till)

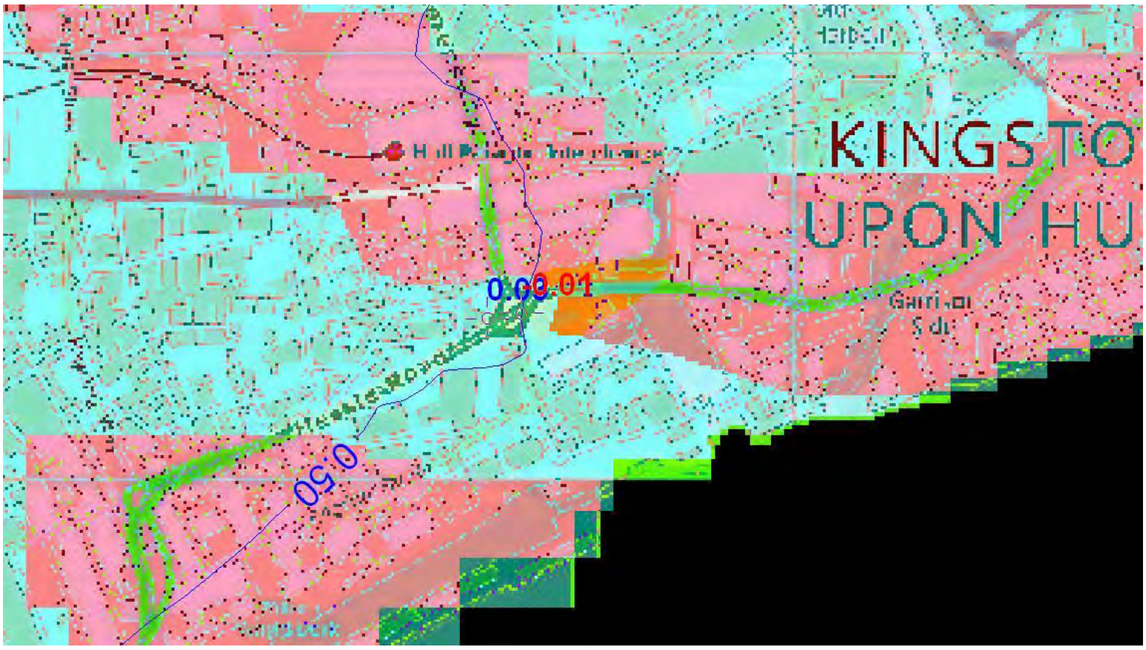


Figure 3-27 Target Residuals in Layer 5 (Glaciolacustrine)

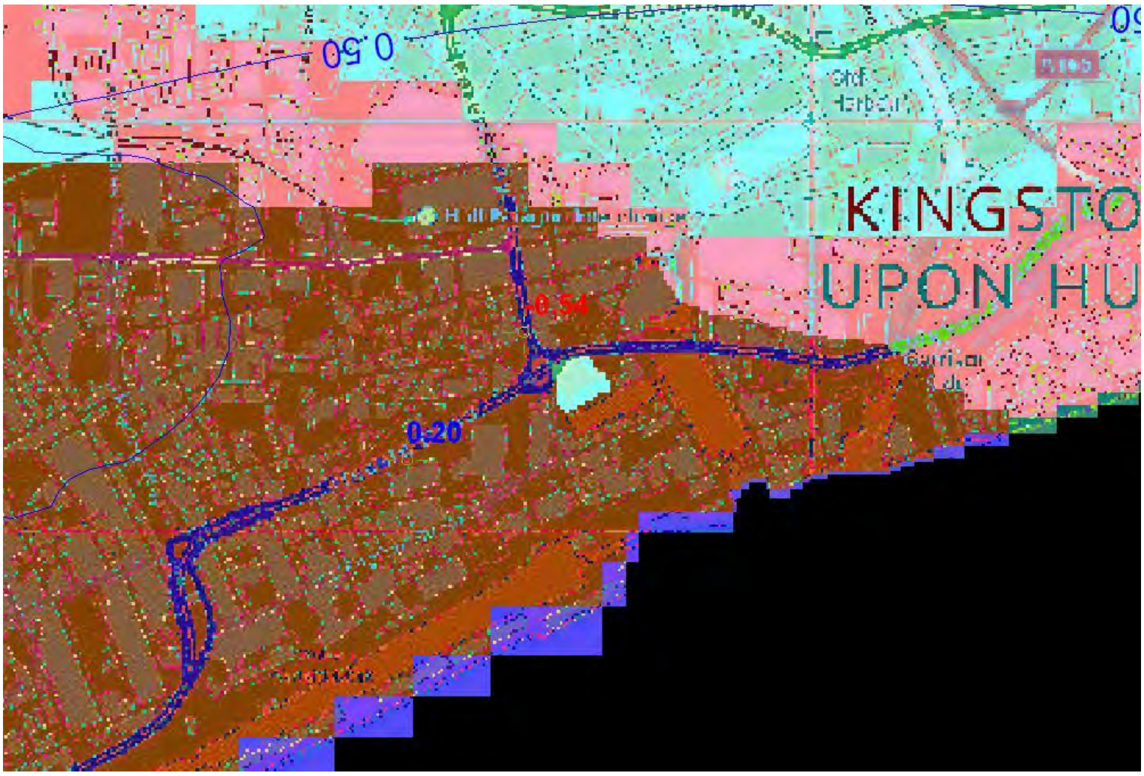


Figure 3-28 Target Residuals in Layer 6 (Fluvioglacial)

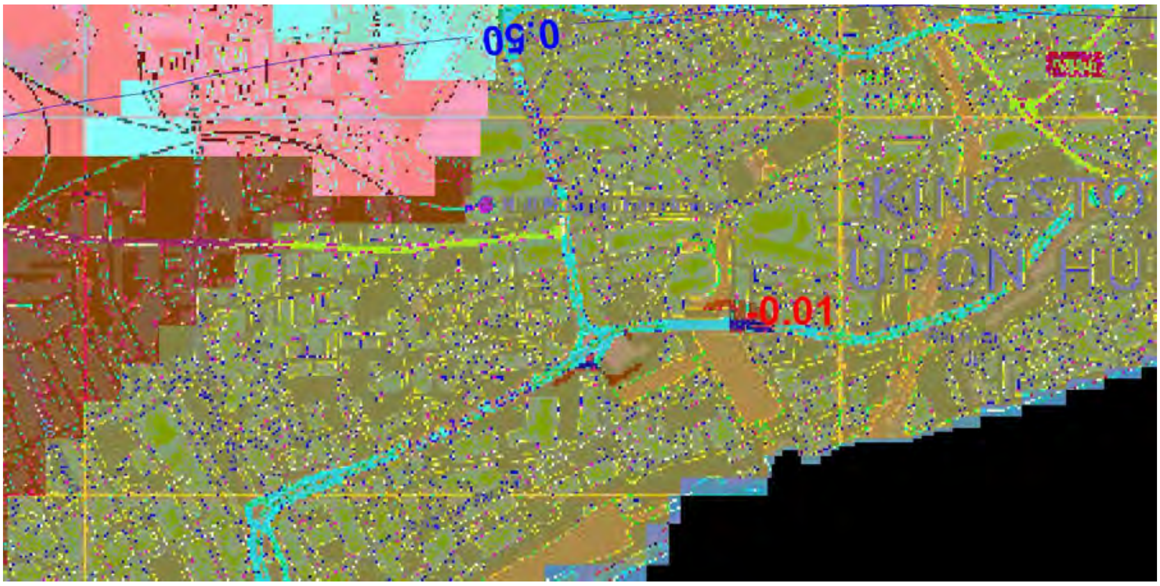
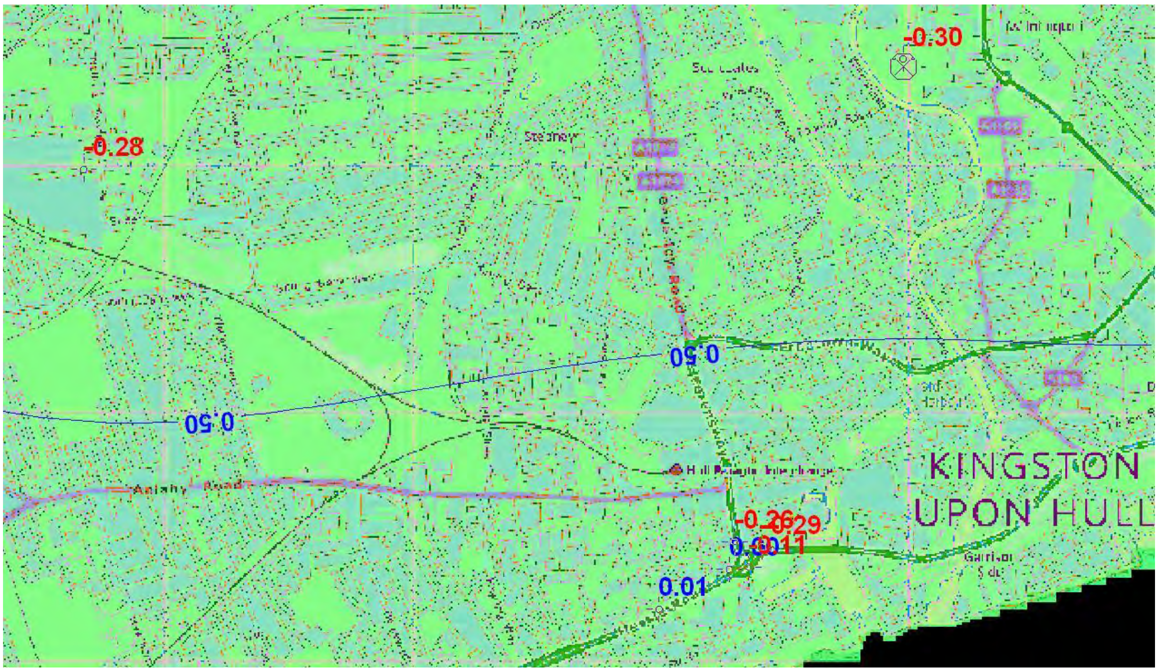


Figure 3-29 Target Residuals in Layer 8 (Main Chalk)



3.13 Steady-state Groundwater Contours

Figure 3-30 to Figure 3-38 show calculated groundwater level contours (in mAOD) for each layer in the steady-state model. In general, groundwater heads fall towards the Humber Estuary. Groundwater contours are seen to be deflected northwards where they cross the River Hull and Ganstead Drain, reflecting the generally "gaining" nature of these watercourses (receiving water from the ground).

Comparison of the plots for the various layers reveals that there is little vertical hydraulic gradient. This is to be expected given the facts that (i) the whole area is relatively close to sea level and (ii) there is not large-scale abstraction from the deeper layers.

Figure 3-30 Steady-State Groundwater Levels Calculated for Layer 1

Figure 3-31 Steady-State Groundwater Levels Calculated for Layer 2

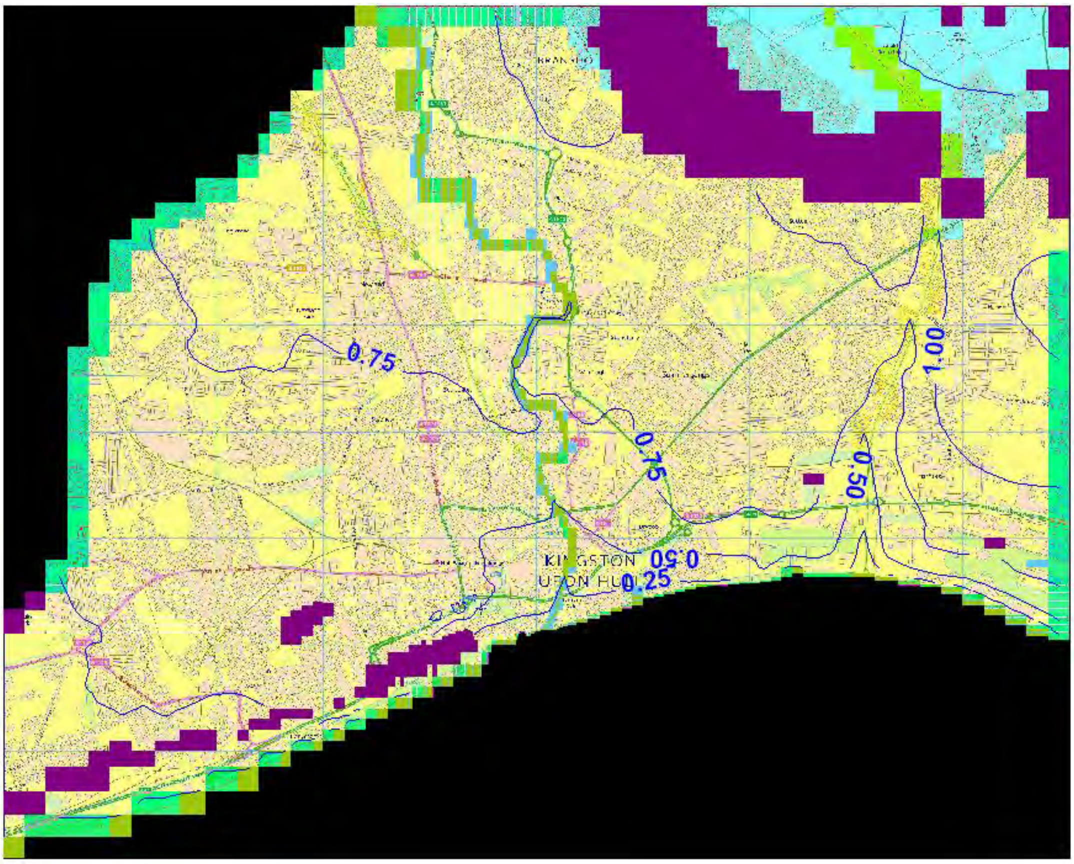


Figure 3-32 Steady-State Groundwater Levels Calculated for Layer 3

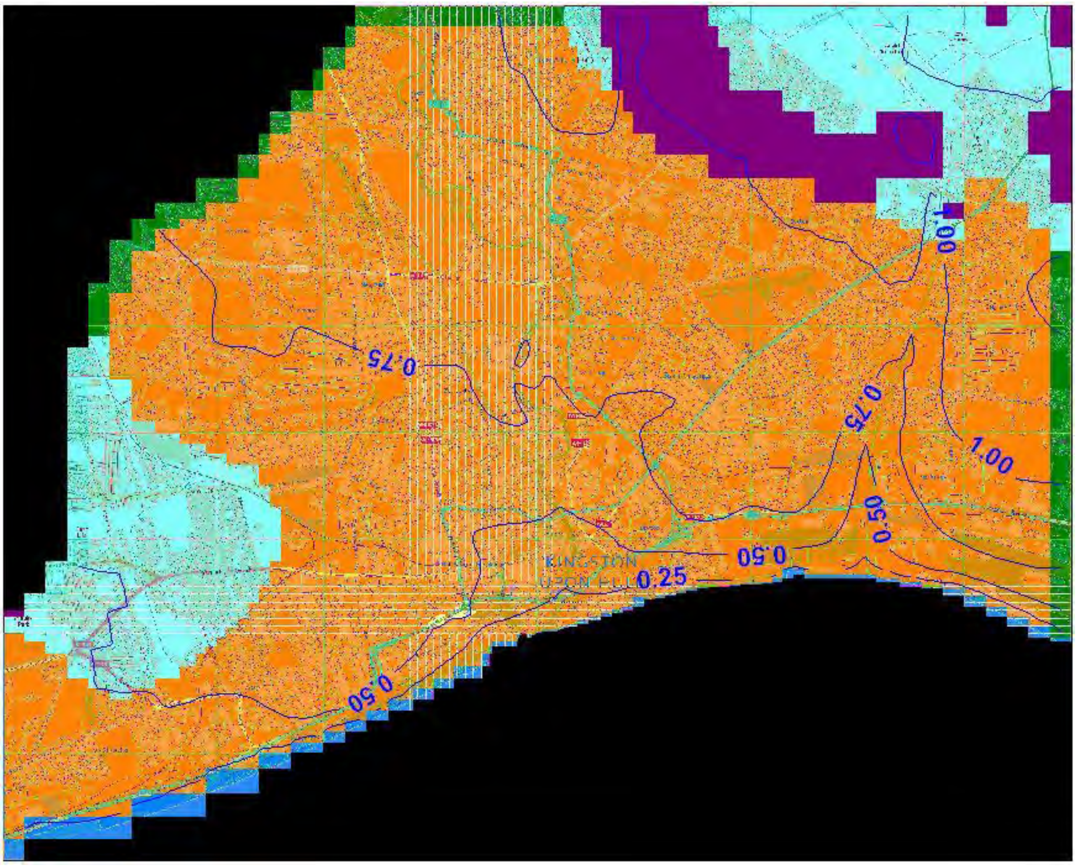


Figure 3-33 Steady-State Groundwater Levels Calculated for Layer 4

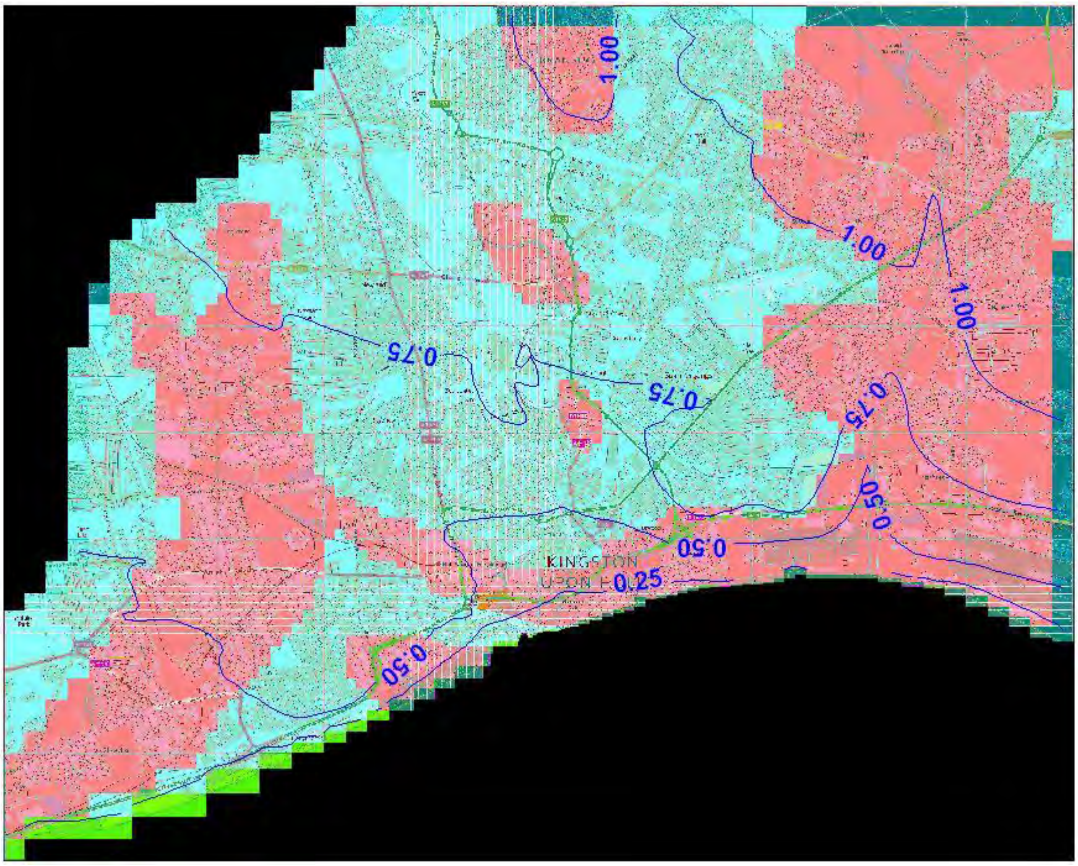


Figure 3-34 Steady-State Groundwater Levels Calculated for Layer 5

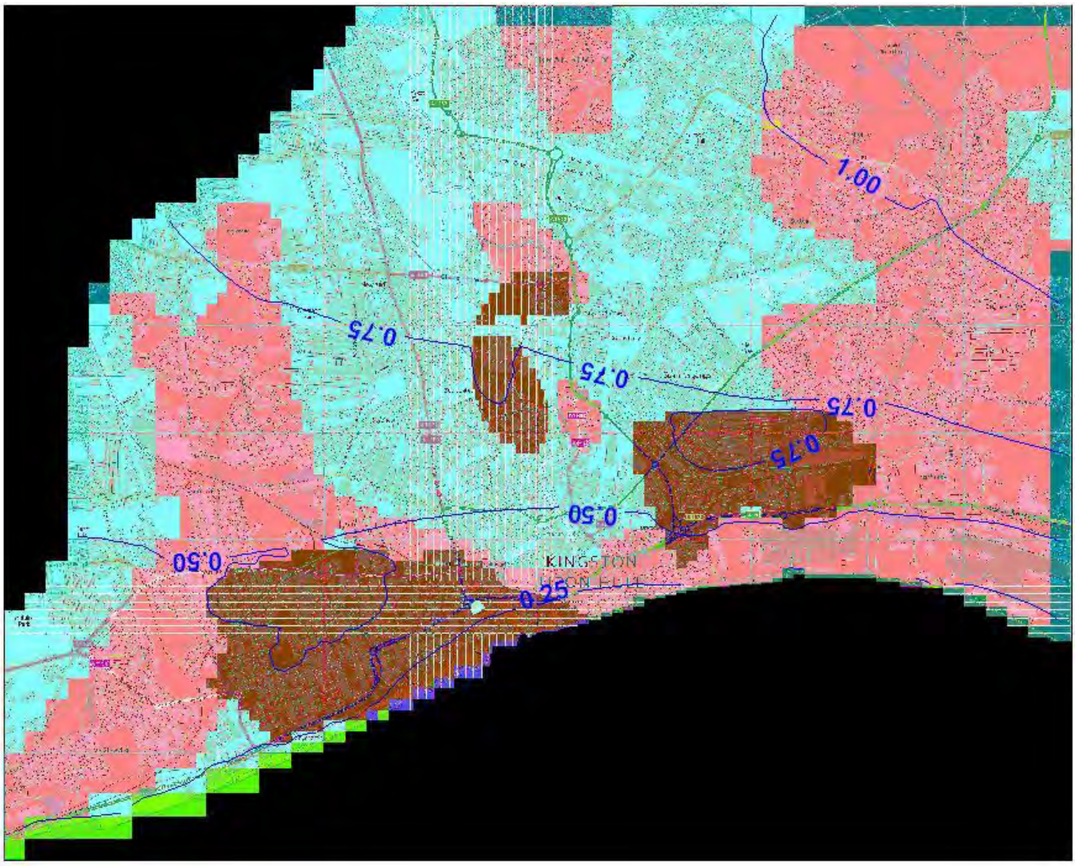


Figure 3-35 Steady-State Groundwater Levels Calculated for Layer 6

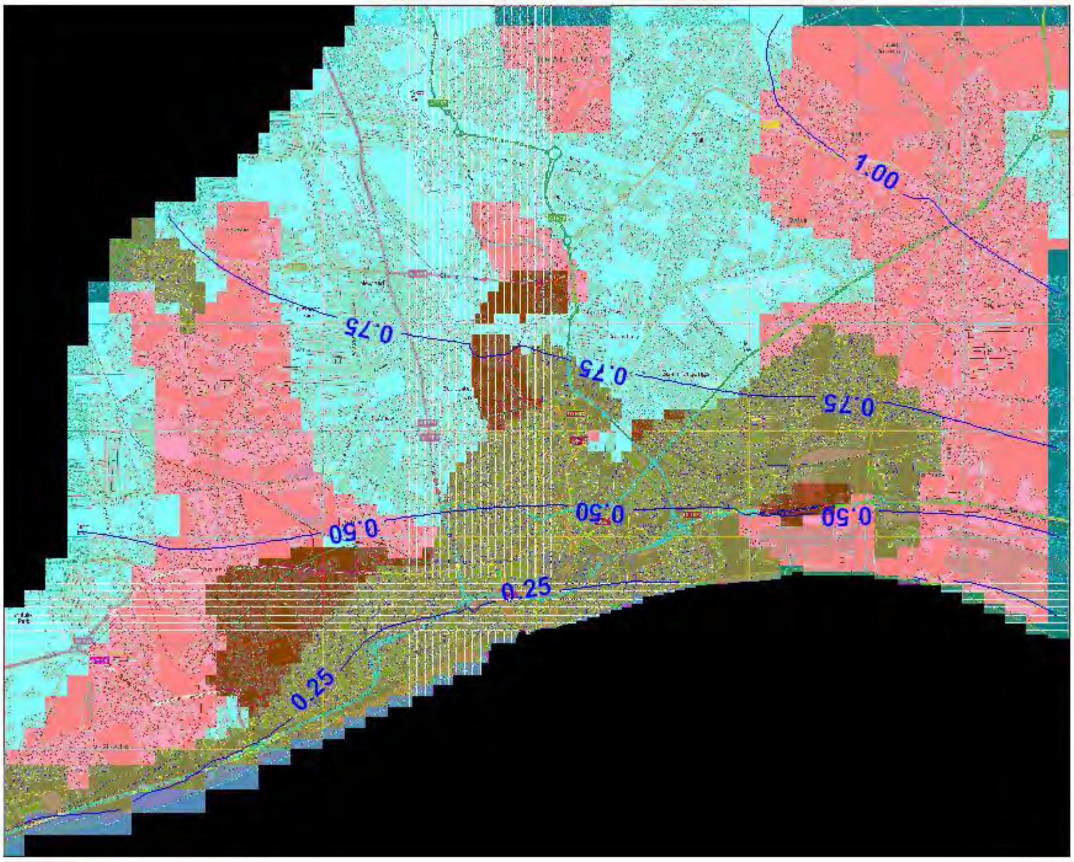


Figure 3-36 Steady-State Groundwater Levels Calculated for Layer 7

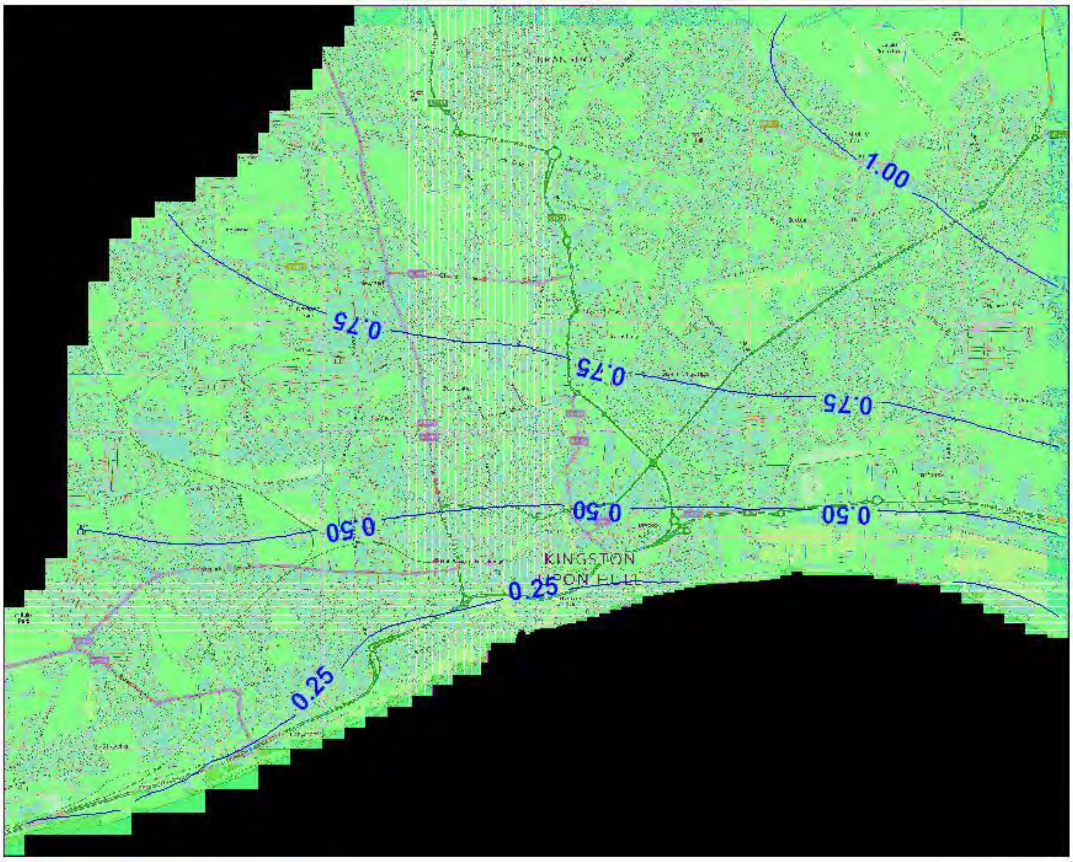


Figure 3-37 Steady-State Groundwater Levels Calculated for Layer 8

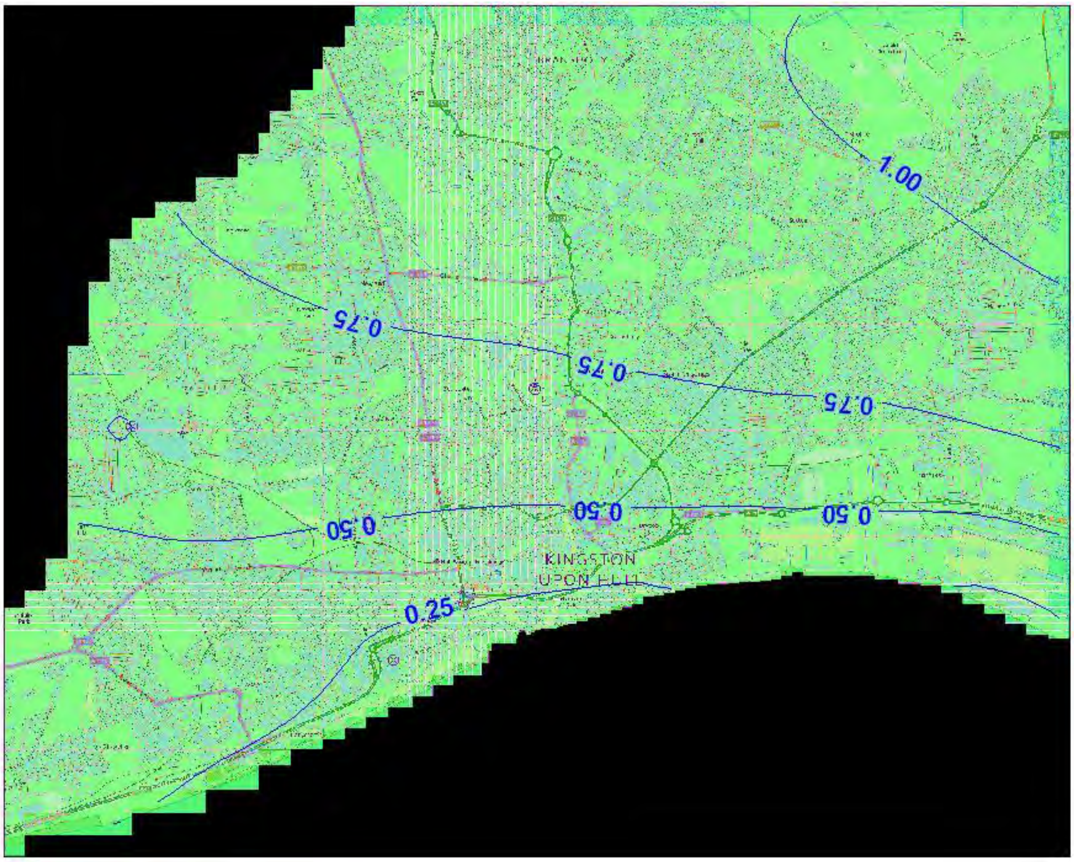
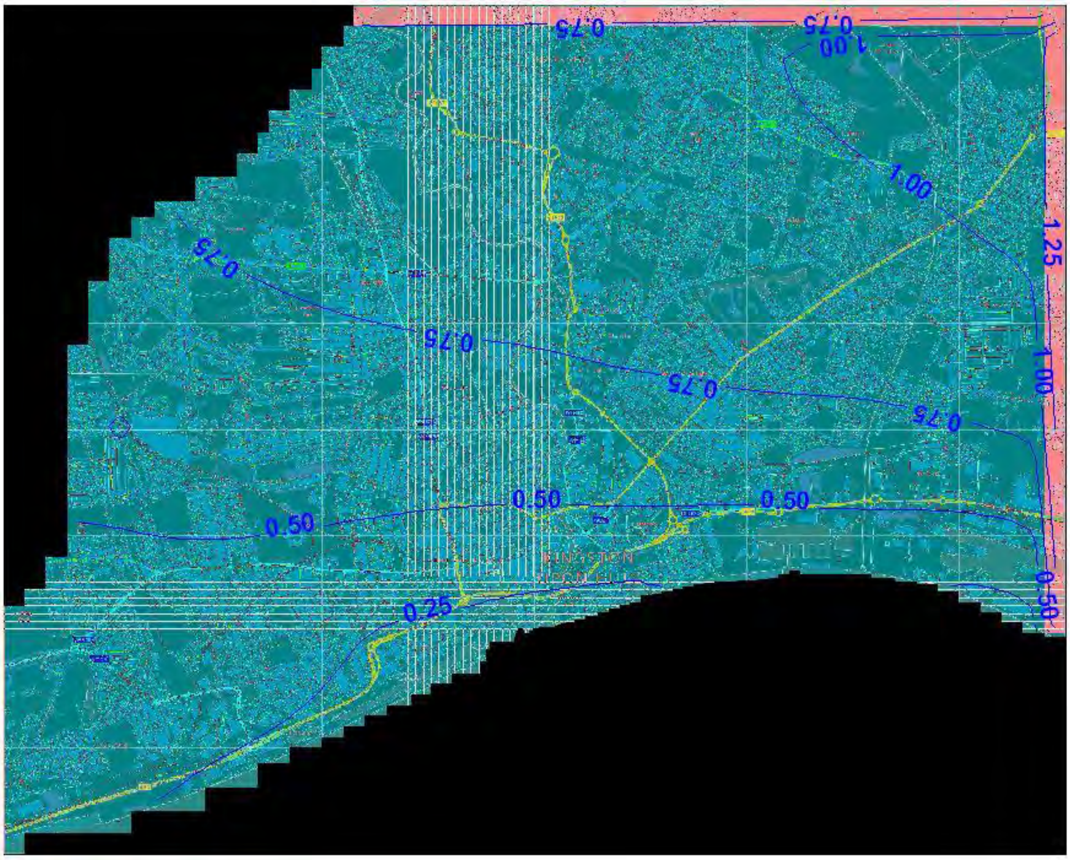


Figure 3-38 Steady-State Groundwater Levels Calculated for Layer 9



3.14 Steady-state Water Balance

Figure 3-39 and Figure 3-40 show the mass balance calculated for the whole steady-state model. The main input is recharge, and the main output is to rivers, including the Humber Estuary.

Figure 3-39 Mass Balance Plot for Whole Steady-State Model

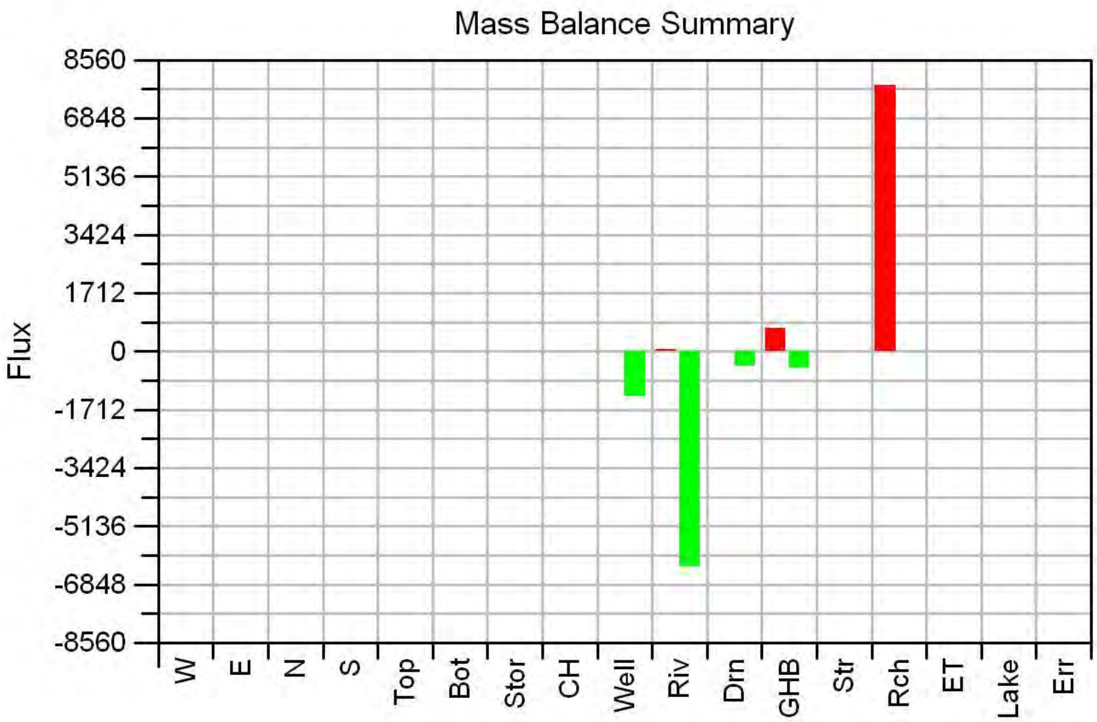
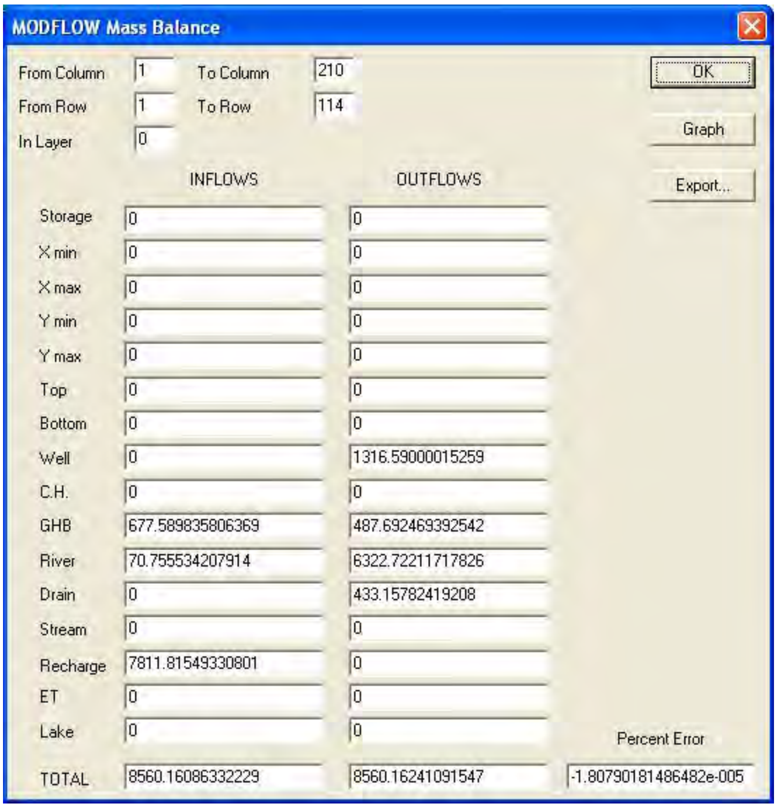


Figure 3-40 Mass Balance Table for Whole Steady-State Model



MODFLOW Mass Balance

From Column: 1 | To Column: 210
From Row: 1 | To Row: 114
In Layer: 0

OK | Graph | Export...

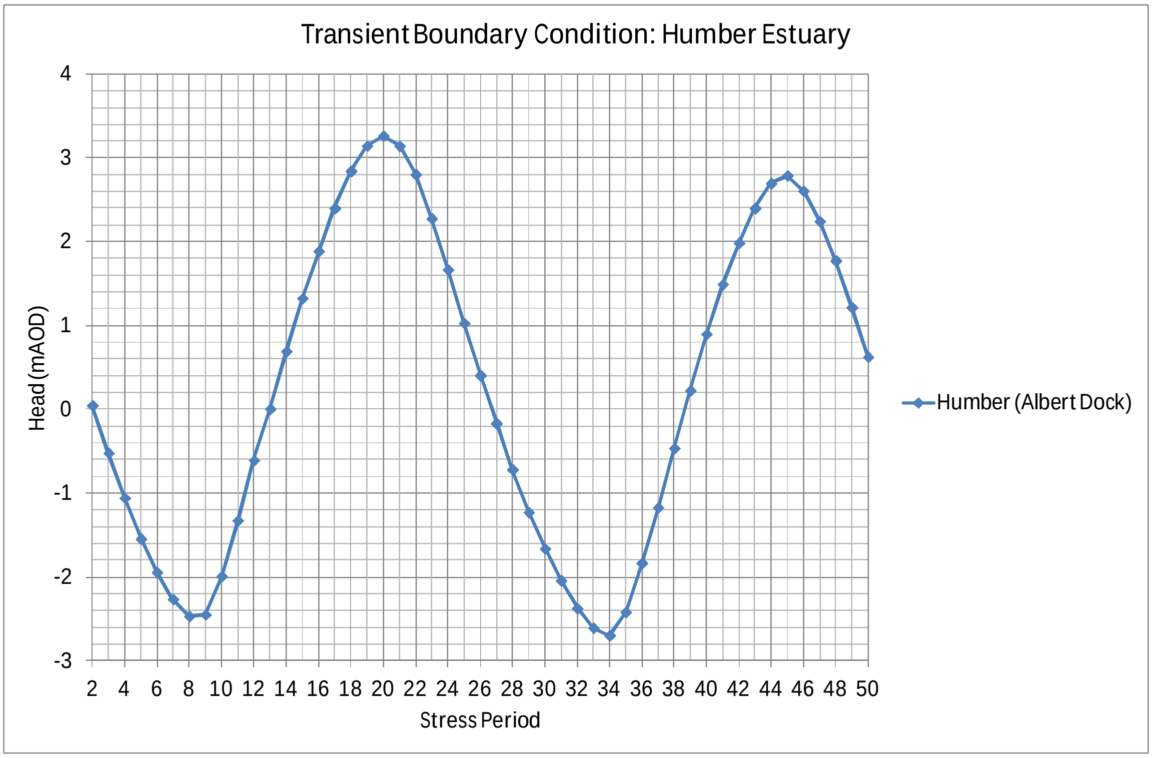
	INFLOWS	OUTFLOWS
Storage	0	0
X min	0	0
X max	0	0
Y min	0	0
Y max	0	0
Top	0	0
Bottom	0	0
Well	0	1316.59000015259
C.H.	0	0
GHB	677.589835806369	487.692469392542
River	70.755534207914	6322.72211717826
Drain	0	433.15782419208
Stream	0	0
Recharge	7811.81549330801	0
ET	0	0
Lake	0	0
TOTAL	8560.16086332229	8560.16241091547

Percent Error: -1.80790181486482e-005

3.15 Boundary Conditions for Transient Simulation

Only one transient boundary condition was defined: the river boundary representing the Humber Estuary. River stage was defined for each stress period using actual levels from Albert Dock for the one-day period 20/01/2014 11:30 - 21/01/2014 11:30. The time series included two tidal peaks of slightly different height (Figure 3-41).

Figure 3-41 River Stage Variations represented in the Transient River Humber Boundary Condition



3.16 Transient Calibration

Transient calibration was undertaken using groundwater level records from five boreholes monitored using automatic dataloggers during 20/01/2014 11:30 - 21/01/2014 11:30.

BH15: Layer 4 (Glacial Till)

BH18A: Layer 8 (Chalk)

BH20: Layer 5 (Glaciolacustrine)

BH21: Layer 3 (Granular Alluvium)

BH30: Layer 2 (Cohesive Alluvium/Peat)

Calibration was undertaken by varying storage properties (controlling the amplitude of tidal fluctuations) and K (controlling the lag/phase of tidal fluctuations). Calibration plots are provided in Figure 3-42 to Figure 3-46.

Good calibrations were obtained for BH15, BH18A, BH21 and BH30. The last looks "off" on the plot, but the head difference is small and the whole trace is fairly flat.

For BH20 the modelled head is out by 1 m compared to the target, but the amplitude and phase of tidal fluctuation are matched fairly well. This target borehole shows heads about 1 m below those recorded from other boreholes in the superficial deposits (Figure 2-2), including in the glaciolacustrine layer. This suggests that BH20 is being influenced by some local factor not represented in the model, although the model is adequately representing groundwater levels in the superficial deposits on a broad scale.

Figure 3-42 Transient Calibration: BH15 (Layer 4, Glacial Till)

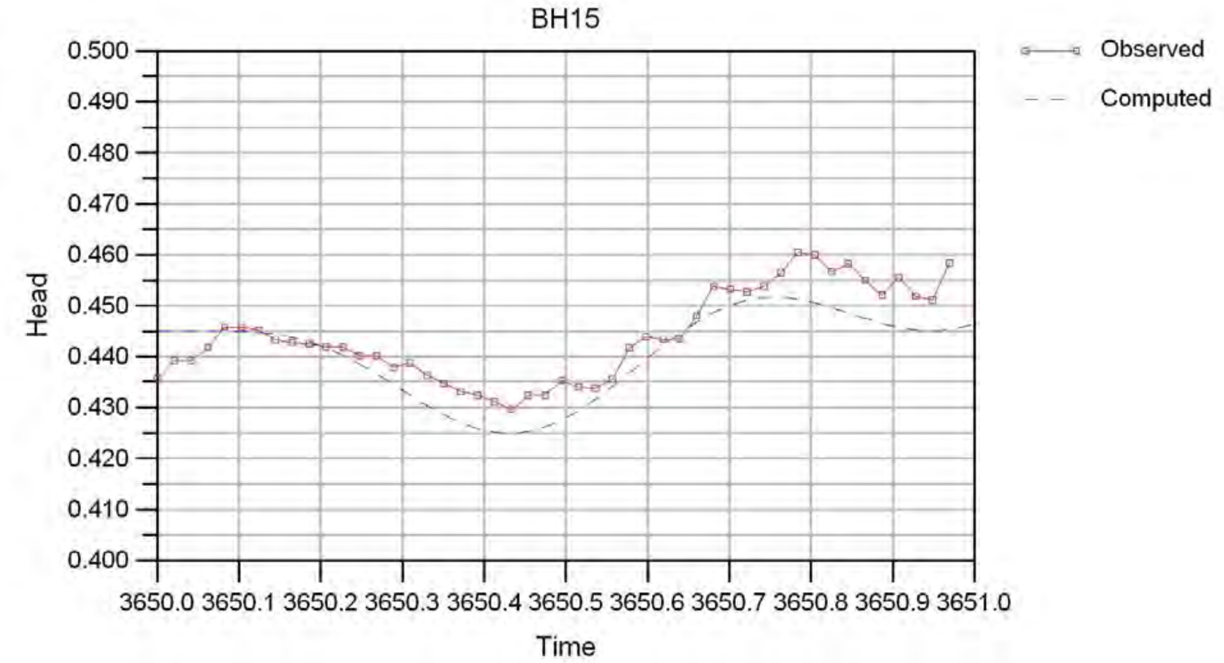


Figure 3-43 Transient Calibration: BH18A (Layer 8, Chalk)

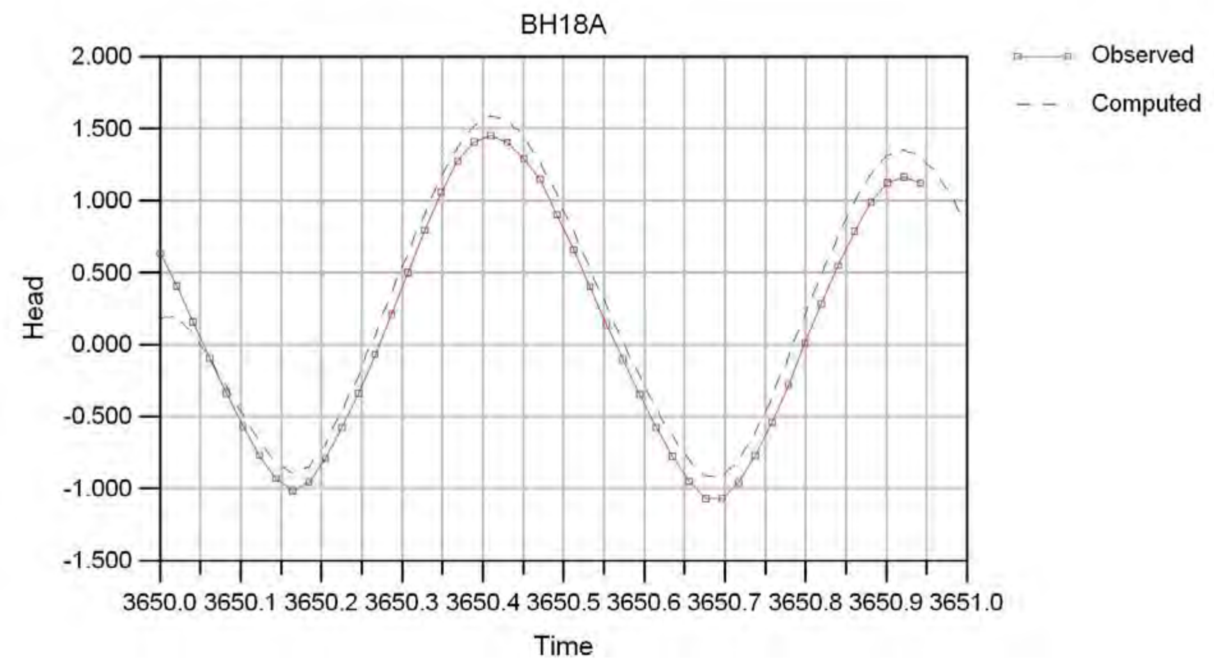


Figure 3-44 Transient Calibration: BH20 (Layer 5, Glaciolacustrine)

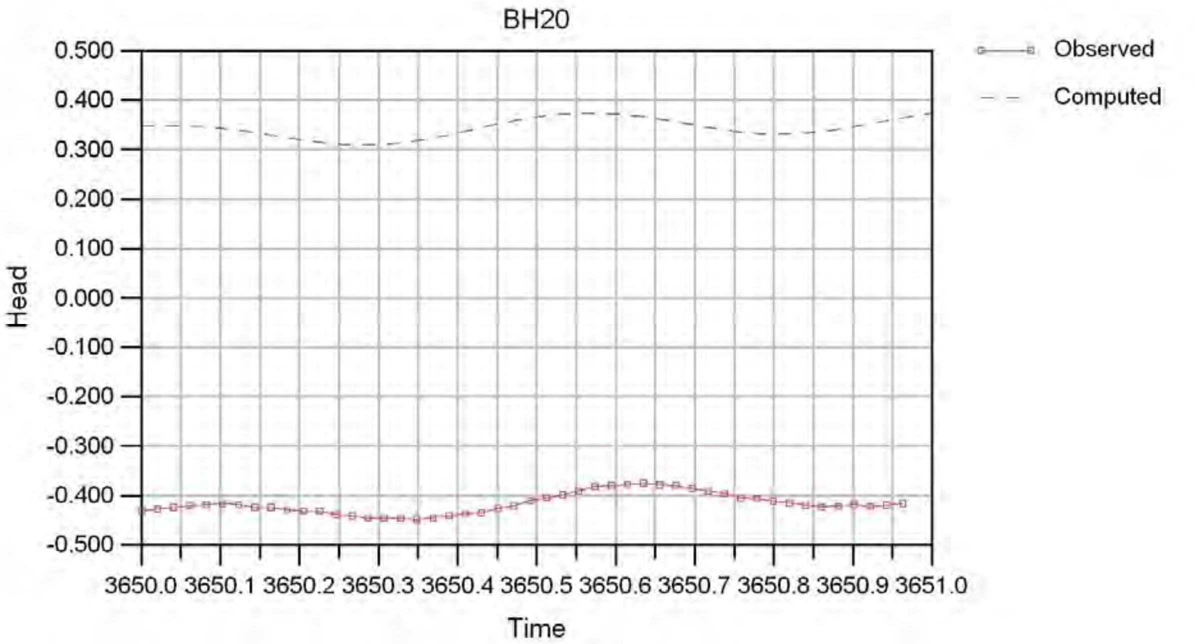


Figure 3-45 Transient Calibration: BH21 (Layer 3, Granular Alluvium)

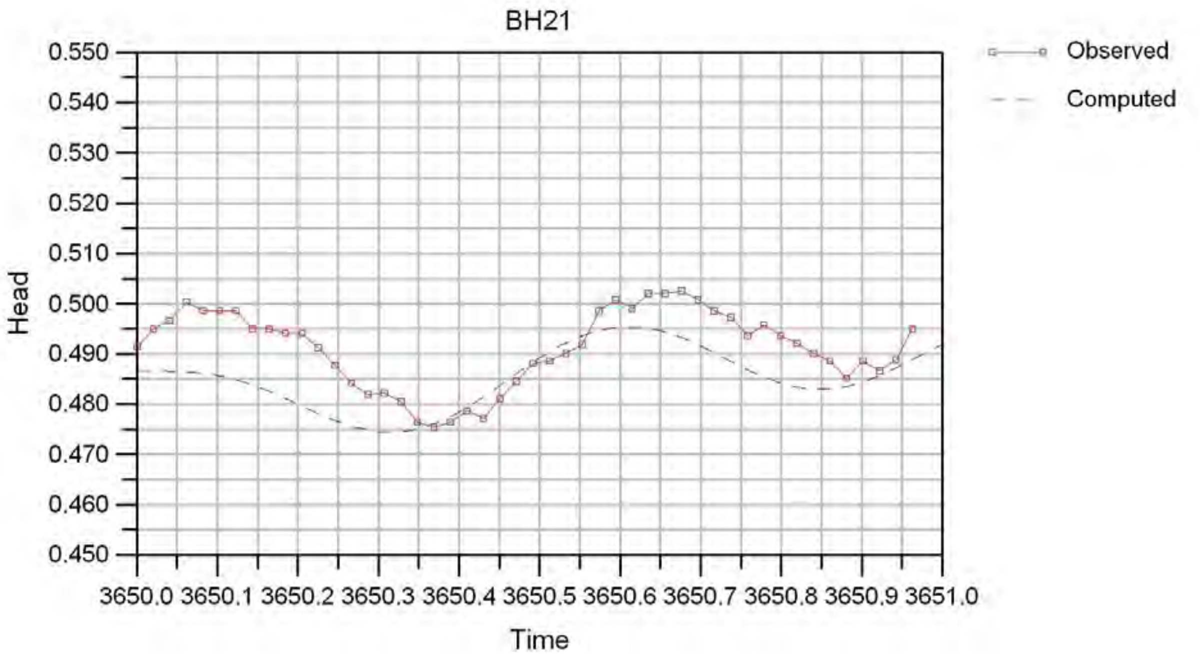
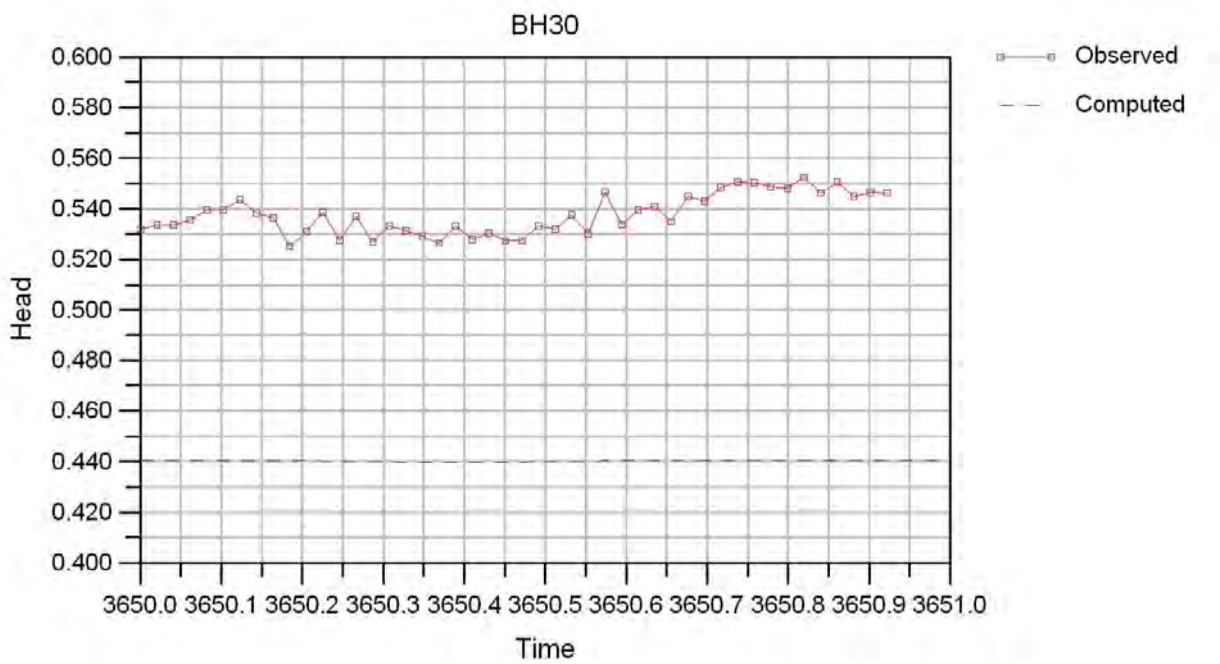


Figure 3-46 Transient Calibration: BH30 (Layer 2, Cohesive Alluvium/Peat)



4 Groundwater Modelling: Scenarios

4.1 Introduction

After the baseline steady-state and transient models had been calibrated, a representation of the road scheme was added so that its impact on groundwater levels and flows could be predicted.

4.2 Representation of the Scheme in the Model

4.2.1 Construction Phase

During the construction phase the scheme will consist of an excavation with secant pile walls but an open base. This situation was represented in the model as follows:

- Secant pile walls:
  - Represented as wall cells 0.9 m thick with K = 8.6 x 10^{-5} m/d.
  - Placed in Layers 1, 2, 3, 4, 5, 6 and 7 (as the piles will extend about 4 m into the Chalk).
  - Walls closed off at the ends to form a "box" (as planned for the piling).
- Excavation:
  - Entered as drain cells (within walled area) into Layer 1 and Layer 2.
  - Drain stage (representing base of excavation) varies from 3 mAOD to -6 mAOD. Width = 20 m (width of road). Bed thickness = 0.5m and K = 1 m/d. K higher than for cohesive alluvium.

The drain cells were used to remove water from the model, representing dewatering of the excavation. The grid cells in the area of the scheme are 10 m x 10 m, so two rows of drain cells were used to represent the road (locally three, to prevent the road from becoming too thin when crossing between rows).

Figure 4-1 shows the walls and drain cells in Layers 1 and 2; Figure 4-2 shows the walls in Layers 3, 4, 5, 6 and 7.

Figure 4-1 Wall Boundaries and Interior Drain Cells (Layers 1 and 2) - inset shows view without base mapping

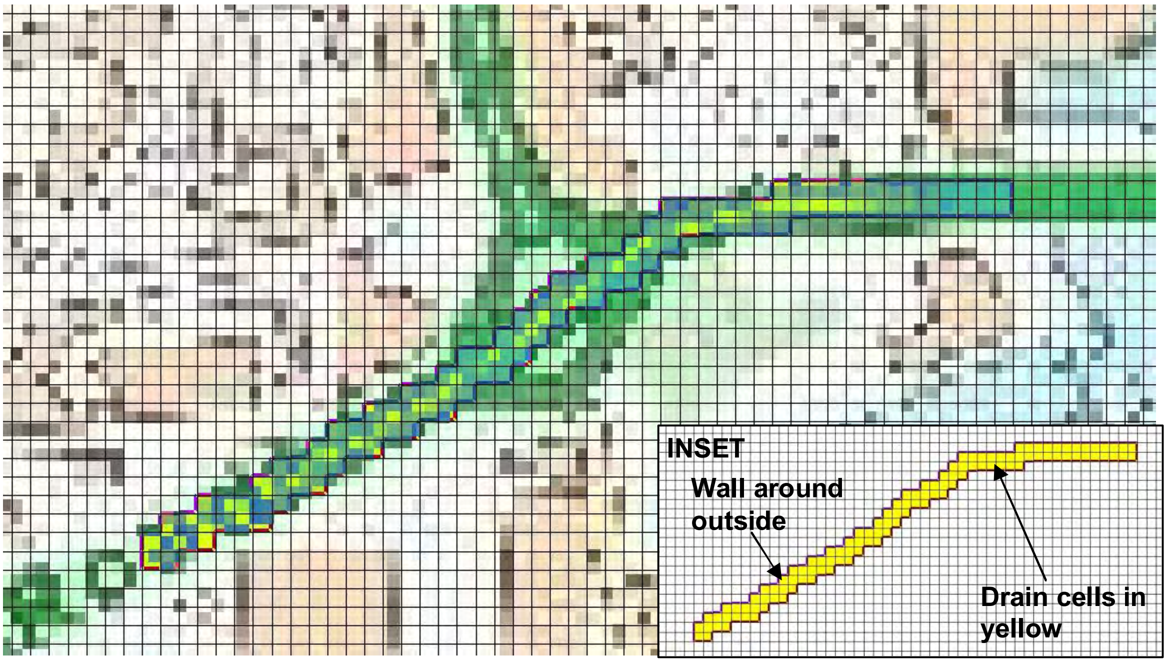
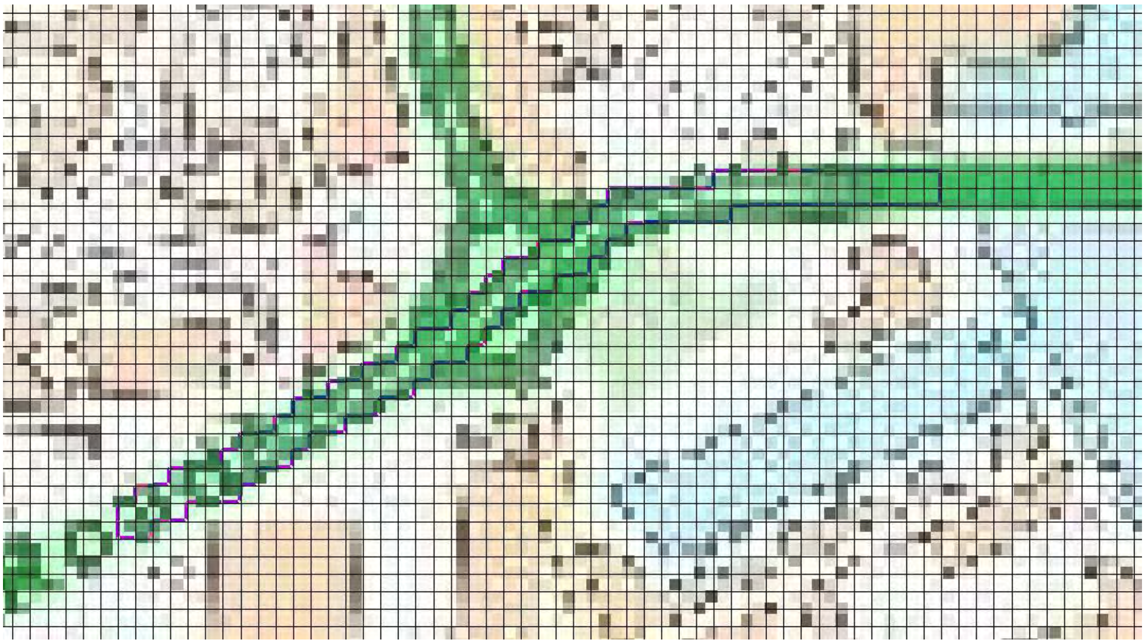


Figure 4-2 Wall Boundaries only (Layers 3, 4, 5, 6 and 7).



4.2.2 Operation Phase

During the operation phase a basal slab will rest in the bottom of the excavation (anchored to the Chalk bedrock by tension piles) and will be overlain by the road construction itself.

Initially, representation of the operational scheme in the model was as for the construction phase but drains were given a bed thickness of 0.8 m and a K of 8.6 x 10^{-5} m/d to reflect the presence of the basal concrete slab. Also, drain stages were adjusted to reflect the constructed road profile (drain level was taken as road top minus 1 m to allow for drainage being below road deck). Tension piles were not represented in the model for reasons explained below.

When the model was run it was found that drawdowns within the road cutting were underestimated, with heads being predicted several metres above the level of the road. For this reason the representation of the scheme in the model was modified in the following ways:

- The horizontal hydraulic conductivity (Kx=Ky) of cells in Layer 1 and Layer 2 within the road cutting (i.e. within the boundary defined by the walls) was set very high, at 1000 m/d. This was to reflect the presence of efficient road drainage. In contrast, the vertical hydraulic conductivity (Kz) was set very low, at 8.6 x 10^{-5} m/d, reflecting the presence of the basal concrete slab. Overall, this highly anisotropic situation allowed rapid horizontal movement of water within the cutting (representing runoff and road drainage), but restricted vertical flow (representing the barrier effect of the basal slab). Within the model, these new hydraulic conductivities were defined as K Zone 11.
- The conductance of the drain cells was increased by a factor of 10,000,000 so as to represent the presence of efficient road drainage. The cells were assigned the following properties: length = width = 10 m (reflecting model grid size within the area of the scheme), thickness = 0.8 m, K = 860 m/d and stage = 1 m below road deck (to reflect road drainage).

These changes had the effect of ensuring that any water that entered the cutting was rapidly removed by the drains. This increased drawdowns within the cutting so that they were realistic, i.e. at the level of road drainage (assumed 1 m below road deck) in the deepest part of the cutting. Changes were not made to the steady-state operation scenario as in that case the drawdowns were already realistic, being close to the base of the excavation. Changes were also not made to the transient operation scenario as in the transient case the main interest was in the impact of the scheme on tidal fluctuations in groundwater head.

Note that in the operation phase steady-state simulation an alternative to using highly conductive drain cells would have been to use constant head cells with heads set 1 m below road deck level.

Numerical Modelling Experiment to Investigate the Impact of Tension Piles on Hydraulic Conductivity

A numerical modelling experiment was undertaken to assess the impact of tension piles on the hydraulic conductivity (K) of the ground. The experiment was undertaken for: (i) a low permeability cohesive superficial deposits layer (minimal K contrast between layer and piles) and (ii) a high permeability Chalk or granular alluvium layer (maximum contrast between layer and piles).

The design assumes that individual tension piles will have a diameter of 600 mm (0.6 m) and a low permeability (K assumed to be 8.6 x 10^{-5} m/d). A diameter of 0.6 gives a circular cross-sectional area of 0.28 m^2. The square root of this area yields the side length of a square of equal area, namely 0.53 m, which is approximately 0.5 m.

A very simple one-layer model was created in MODFLOW with 0.5 m square grid cells. The model domain was rectangular and measured 18.5 m (east-west) by 20 m (north-south). The layer was 4 m thick (corresponding to the piled thickness of Chalk) and was isotropic with K = 75 m/d (representing the Chalk), K = 0.05 m/d (representing cohesive alluvium or till) or K = 0.01 m/d (representing glaciolacustrine deposits). These layer K values were the Kxy values from the main model (Kz was not used as there was no vertical component of flow). The Chalk K value of 75 m/d was higher than the K values used for the granular superficial deposits in the main model, but within the range typical for sand and gravel (Brassington, 2007). It was therefore taken to represent both the Chalk and granular superficial deposits. Piles were represented by assigning lower K values (8.6 x 10^{-5} m/d) to individual 0.5 x 0.5 m cells in a 4 m x 5 m grid pattern. A constant head of 1 mAOD was specified along the northern edge, and a constant head of 0 mAOD was specified along the southern edge. The top of the model was set at -20 mAOD and the layer specified as confined. Pile spacing was assumed to be closest perpendicular to the direction of flow. This was a conservative assumption as it maximised the impact of the piles on flow.

The model was run with the piles represented and with them removed (K of piles made equal to K in rest of layer). Groundwater head contours were plotted for both scenarios (see Figure 4-3, Figure 4-4, Figure 4-5 and Figure 4-6). Comparison of these plots showed the impact of piling on heads to be slight, with groundwater readily flowing around the piles. The greatest effect was seen at the edges of the model, reflecting the presence of an adjacent no-flow boundary (here flow that would have diverged symmetrically around a pile is all forced to flow around the same side). A higher hydraulic gradient (4m head drop over the model) was tried, and this gave similar results. As the effect of the tension piling was seen to be small, tension piles were not represented in the main model for the operation scenarios.

In theory, piling in a low permeability cohesive superficial deposits layer should have less impact on flow than piling in a higher permeability Chalk or granular superficial deposits layer as there is less contrast in K between the piles and surrounding material; in other words, there should be less deflection of groundwater contours and flow vectors. In the case under consideration the difference is slight as even the glaciolacustrine deposits (K = 1 x 10^{-2} m/d) are much more permeable than the piles (K = 8.6 x 10^{-5} m/d).

The model runs simulating piling in a low permeability cohesive superficial deposits layer suggest that horizontal flow in such a layer is unlikely to be significantly affected by the piling. It should be borne in mind that in horizontal multilayer aquifer-aquitard systems (with a K contrast of two orders of magnitude or more between aquifers and aquitards), regional groundwater flow is generally near-horizontal in the aquifer layers and near-vertical in the aquitard layers; this reflects the tangent law of refraction (Freeze and Cherry, 1979). In the case of steady-state vertical flow, i.e. parallel to the long axes of the piles, the head contours and flow vectors will not be deflected but the flow rates will be lower along the piles themselves (due to lower K).

Figure 4-3 Tension Piling Simulation for Cohesive Superficial Deposits: Scenario with Piles in Place (upper part is vertical N-S cross-section; lower part is plan view; head contours in metres above datum; arrows are flow vectors)

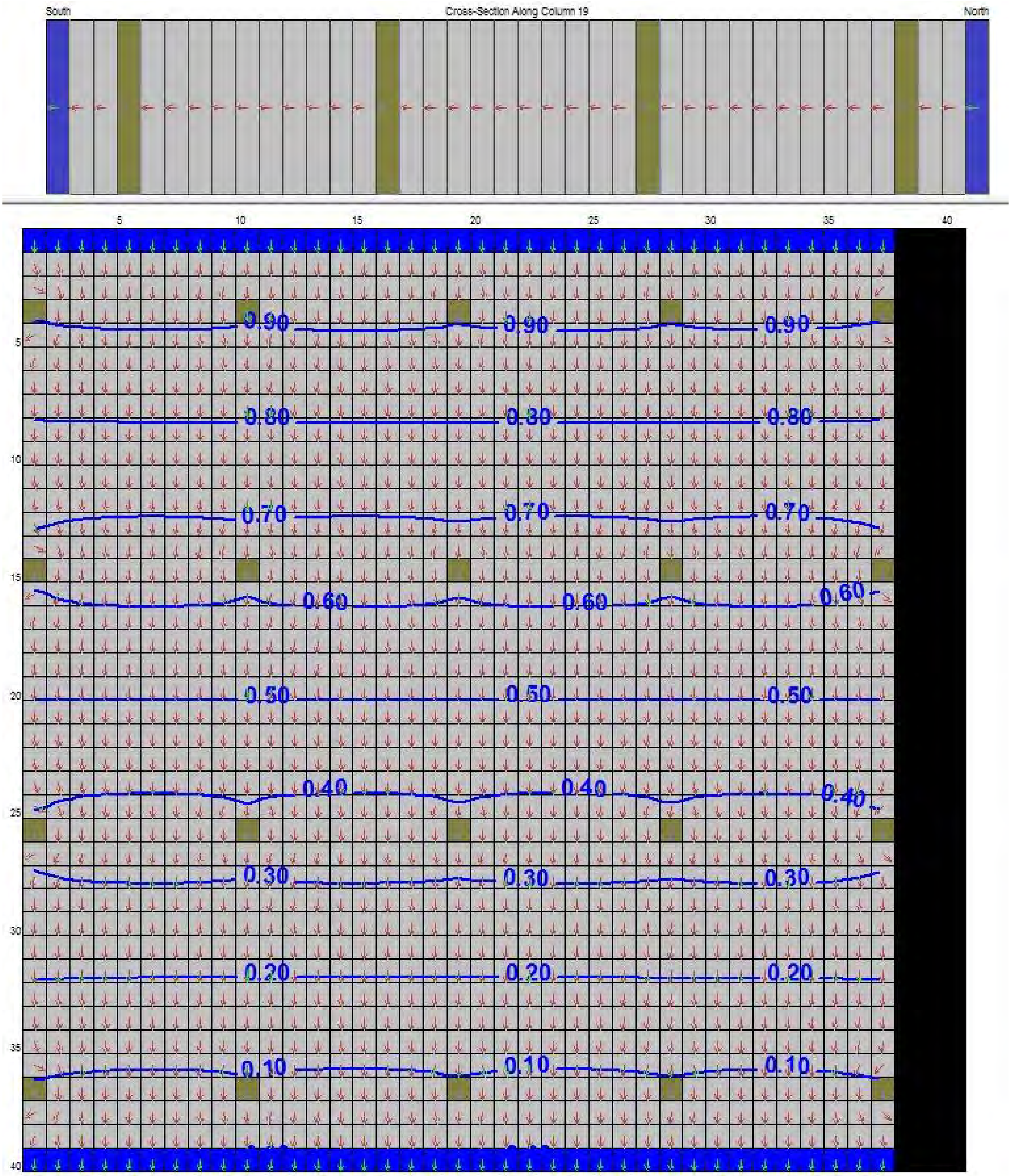


Figure 4-4 Tension Piling Simulation for Cohesive Superficial Deposits: Scenario with No Piles (as for Figure 4-3 but darker grey cells show where piles have been removed)

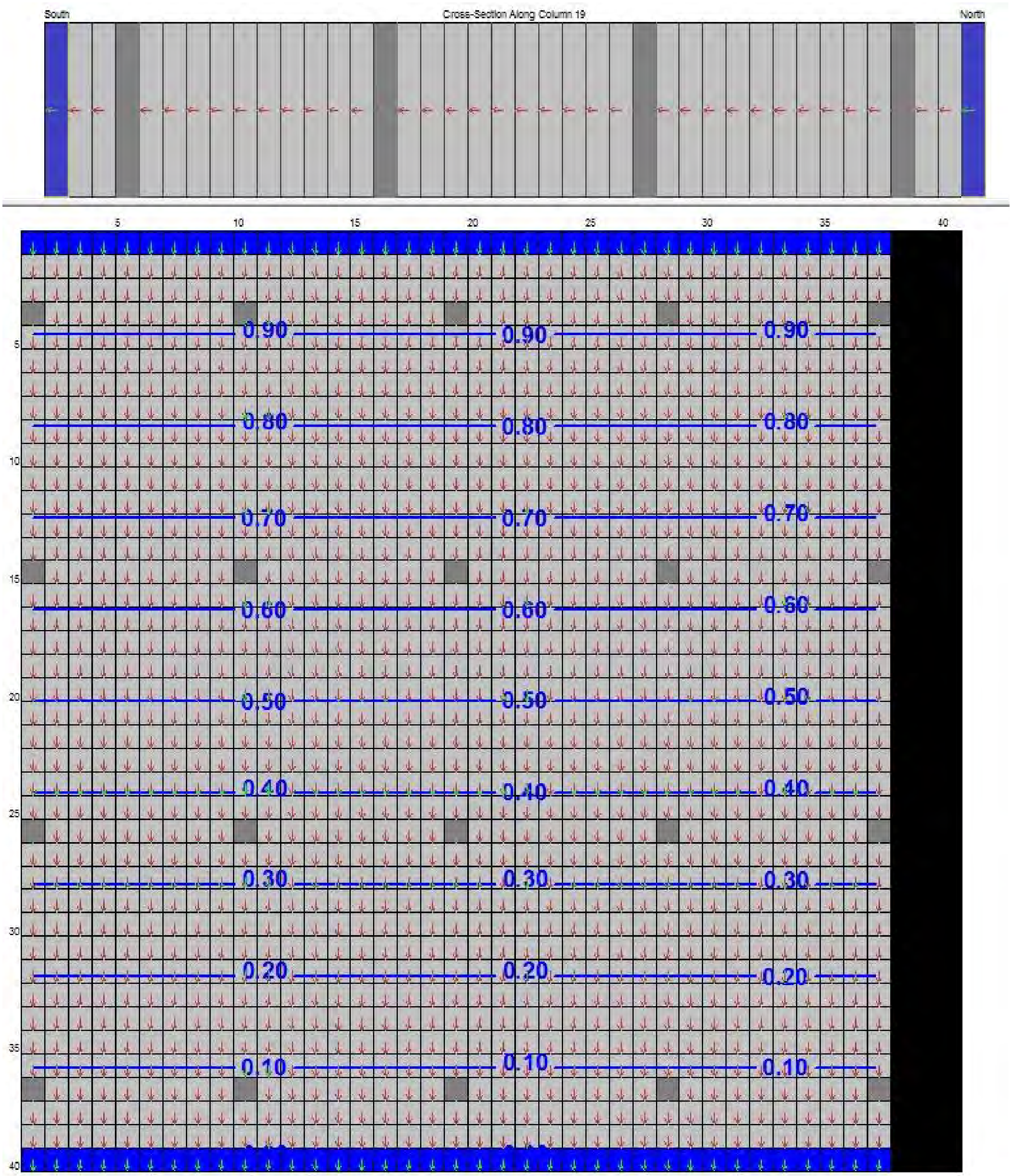


Figure 4-5 Tension Piling Simulation for Chalk: Scenario with Piles in Place (upper part is vertical N-S cross-section; lower part is plan view; head contours in metres above datum; arrows are flow vectors)

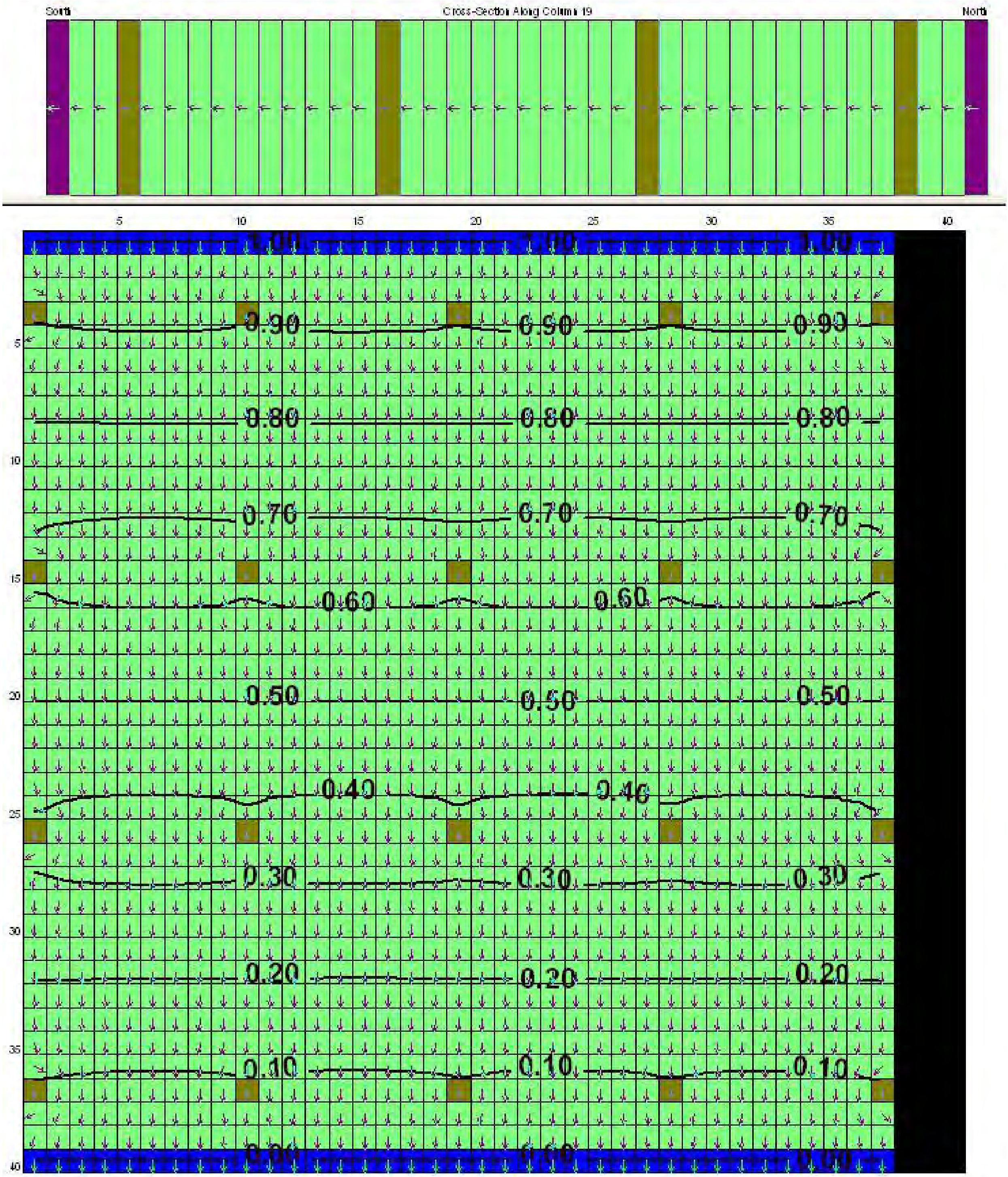
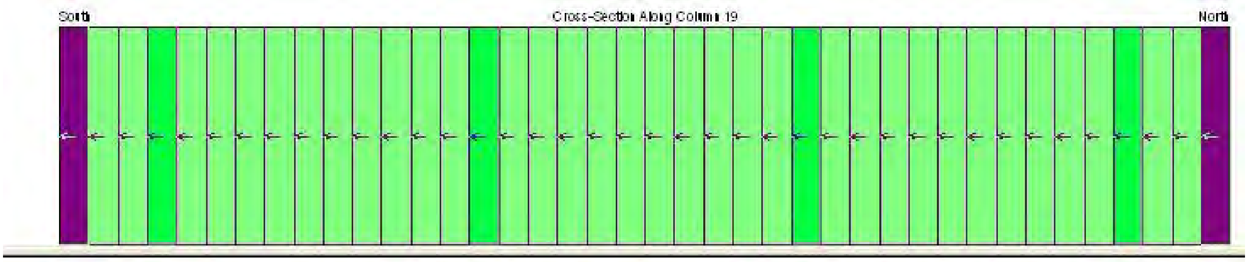
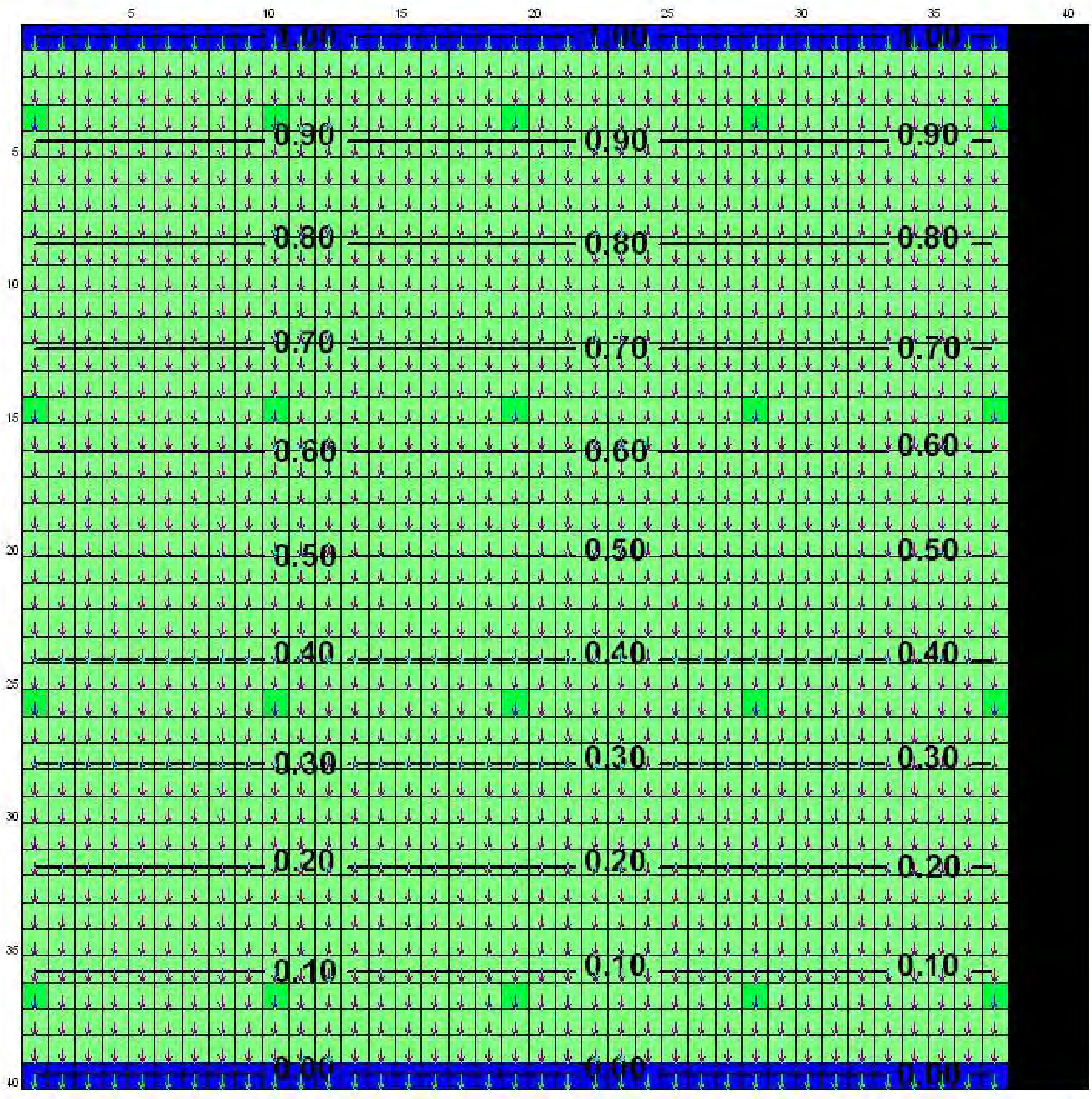


Figure 4-6 Tension Piling Simulation for Chalk: Scenario with No Piles (as for Figure 4-5 but darker green cells show where piles have been removed)





4.3 Monitoring Heads within the Transient Model

For the transient runs (baseline, construction and operation), 12 monitoring points, in the form of fictitious monitoring wells, were added to the model in order to observe the effects of the scheme on groundwater levels (Figure 4-7). The monitoring points were placed in Layer 2 (Cohesive Alluvium), Layer 3 (Granular Alluvium) and Layer 8 (Chalk).

Figure 4-8 and Figure 4-9 show baseline hydrographs for the monitoring points. A strong tidal signal is apparent in the Chalk (Figure 4-8) and a weaker one is visible at two monitoring points within the Granular Alluvium (BHNN3 and BHSS3; Figure 4-9).

Figure 4-7 Monitoring Points for Transient Model Runs (i = Layer = 2, 3, 8)

4.4 A Note on the Representation of the Scheme in the Transient Model Runs

Initially the 20 m wide road scheme was represented by wall cells spaced one cell (10 m) apart, rather than two cells (20 m) apart, although the drain cells had a width of 20 m specified within their conductance. This representation was satisfactory in terms of representing drainage impacts, but was not considered conservative in terms of the potential "damming" impact of the walls on groundwater. For this reason the steady-state scenario model runs were repeated with a wider spacing of walls more closely approximating the actual width of the road. The results of these runs are the results presented in this chapter. However, the transient runs were undertaken with the narrow representation only. This was considered justified as comparison of the steady-state runs with wide and narrow versions of the road showed very little difference in groundwater levels and drawdown patterns.

Figure 4-8 Baseline Hydrographs for Monitoring Points within the Transient Model: Chalk

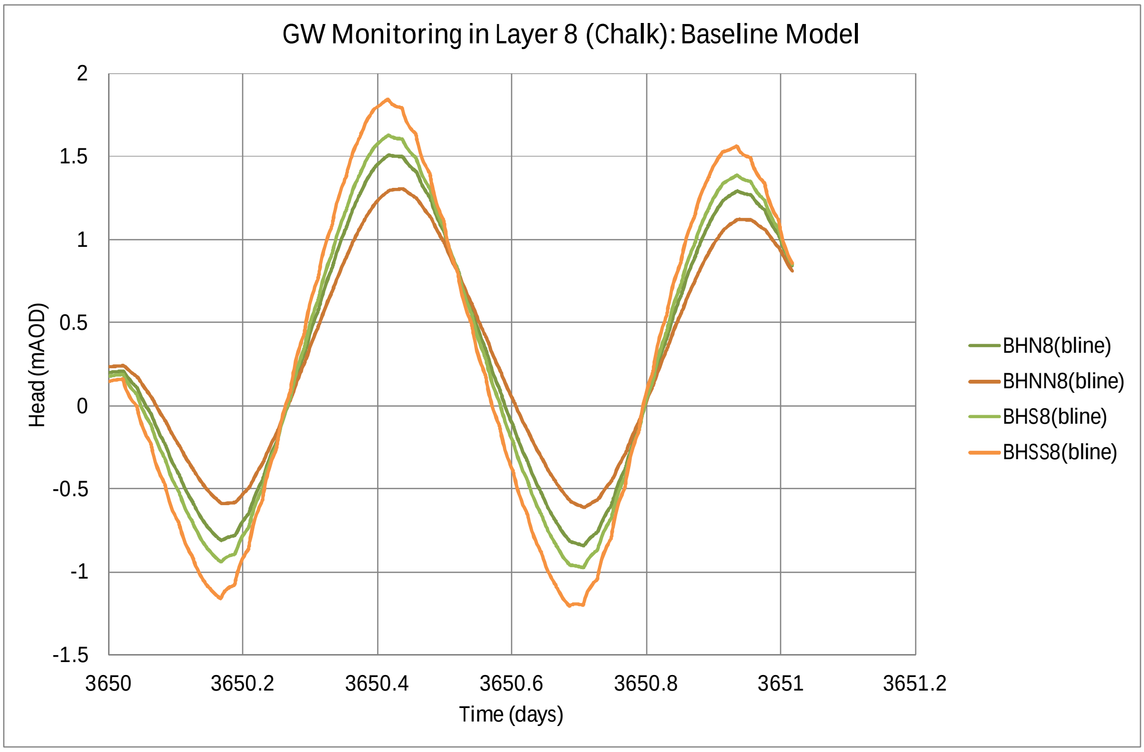
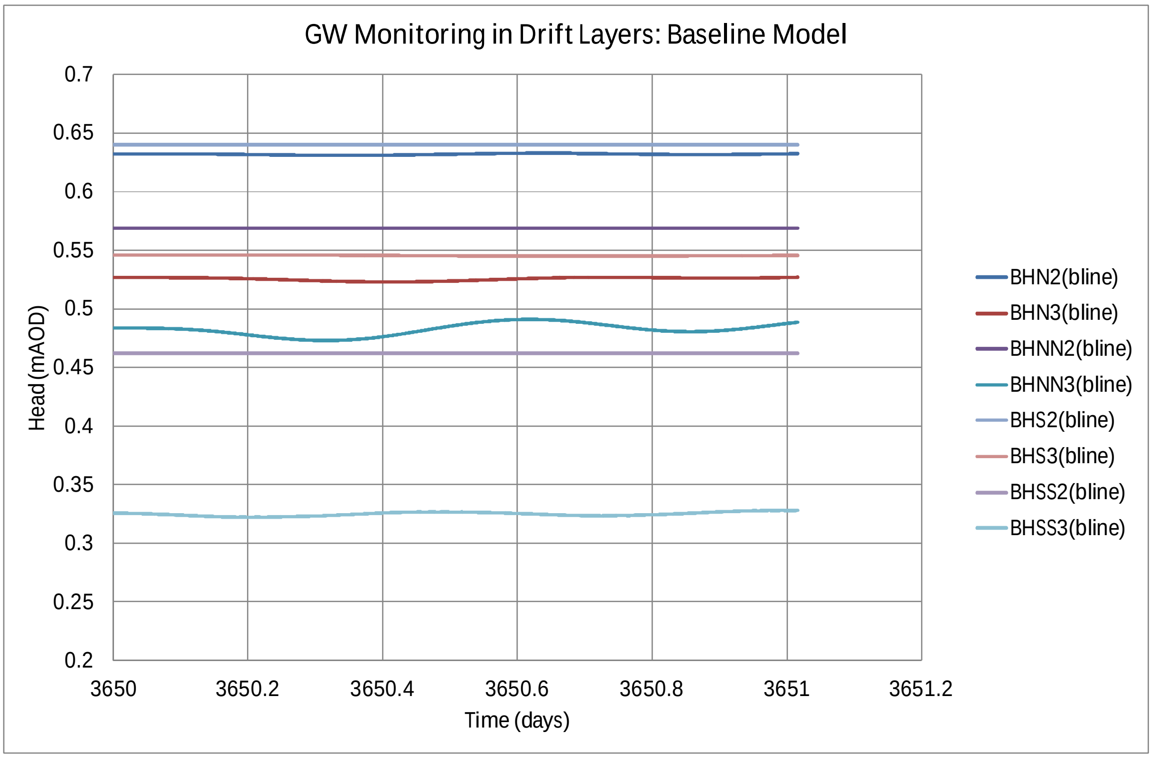


Figure 4-9 Baseline Hydrographs for Monitoring Points within the Transient Model: Superficial Deposits



4.5 Construction Scenario: Steady-State Model Run

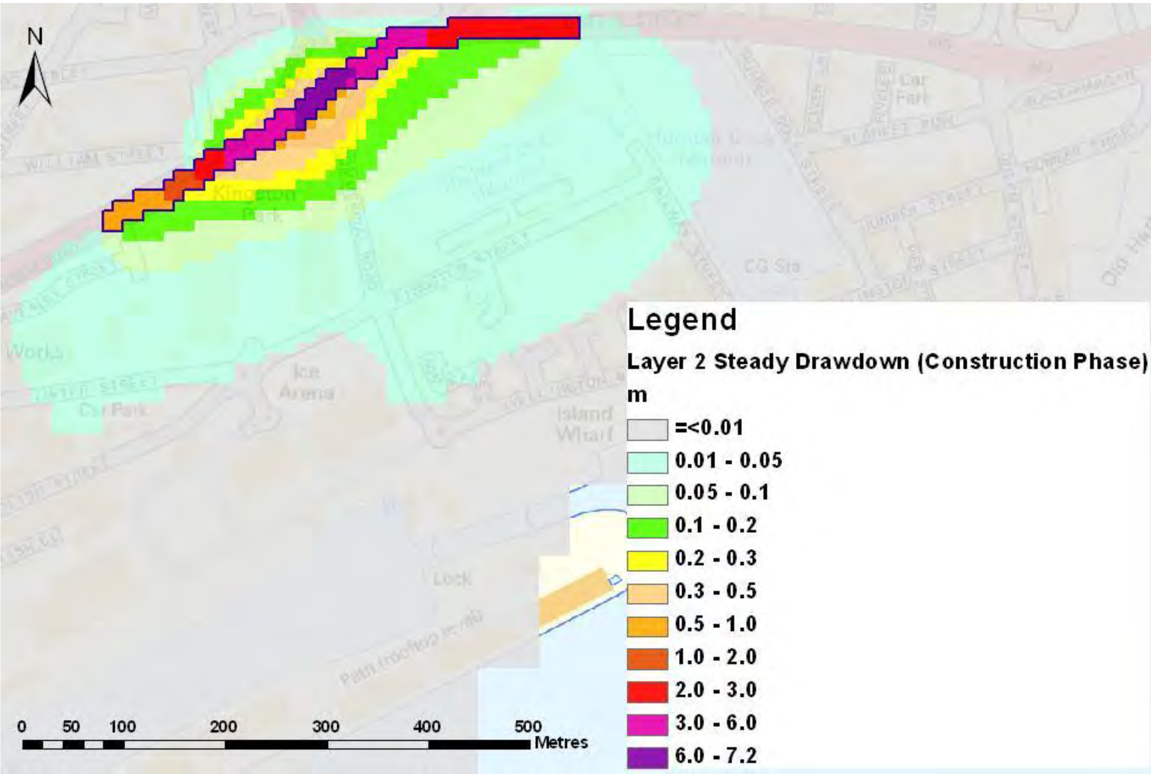
The model was run in steady-state with the construction phase of the scheme represented as described in Section 4.2.1. Figure 4-10 shows the calculated pattern of drawdown of groundwater heads (relative to the baseline scenario) for Layer 2, the Cohesive Alluvium. This is the layer that showed the largest drawdowns.

The main features of Figure 4-10 are:

- The area of significant drawdown is fairly localised around the scheme.
- As expected, drawdowns are greatest within the secant-piled boundary, with the maximum drawdowns occurring in the central part of the scheme where the excavation is deepest.
- The extent of drawdown is greater down-gradient (south of the scheme) than upgradient (north of the scheme), presumably reflecting a slight "damming" of groundwater by the secant pile walls. However, it should be noted that hydraulic gradients across the structure reflect the boundary heads selected for the modelling. No measurable gradients have been identified from groundwater level monitoring within the construction footprint. Drawdown from an initially flat water table or potentiometric surface (zero gradient) would yield a more symmetrical pattern than that shown, with more drawdown along the northern edge of the road cutting.

Steady-state inflow to the excavation, as measured by the total flow to the drain cells, is 13.4 m^3/d. This represents a worst case, i.e. maximum, inflow rate (based on the model parameters) because the steady-state simulation assumes instantaneous drainage to maximum dredge level. In reality, dewatering will take place gradually as excavation progresses, and hydraulic gradients within the zone of influence will be lower, giving lower inflow rates. Also, dewatering will lower the saturated thickness of the layer(s) contributing flow; this will lower transmissivities and therefore will also lower inflow rates.

Figure 4-10 Modelled Steady-State Drawdown (in Layer 2) for Construction Scenario



4.6 Construction Scenario: Transient Model Run

Figure 4-11, Figure 4-12 and Figure 4-13 compare groundwater level hydrographs for the baseline and the construction phase. Note that the Chalk hydrographs plot on top of each other as there is no change between baseline and construction. Hydrographs for the superficial deposits show small (<0.35 m) changes in average groundwater level between the baseline and construction scenarios. The points south of the scheme all show a fall in average groundwater level from baseline to construction, whereas points north of the scheme show a mixed response (some points show a rise and others a fall).

Figure 4-11 Baseline and Construction Phase Hydrographs for Monitoring Points within the Transient Model: Chalk

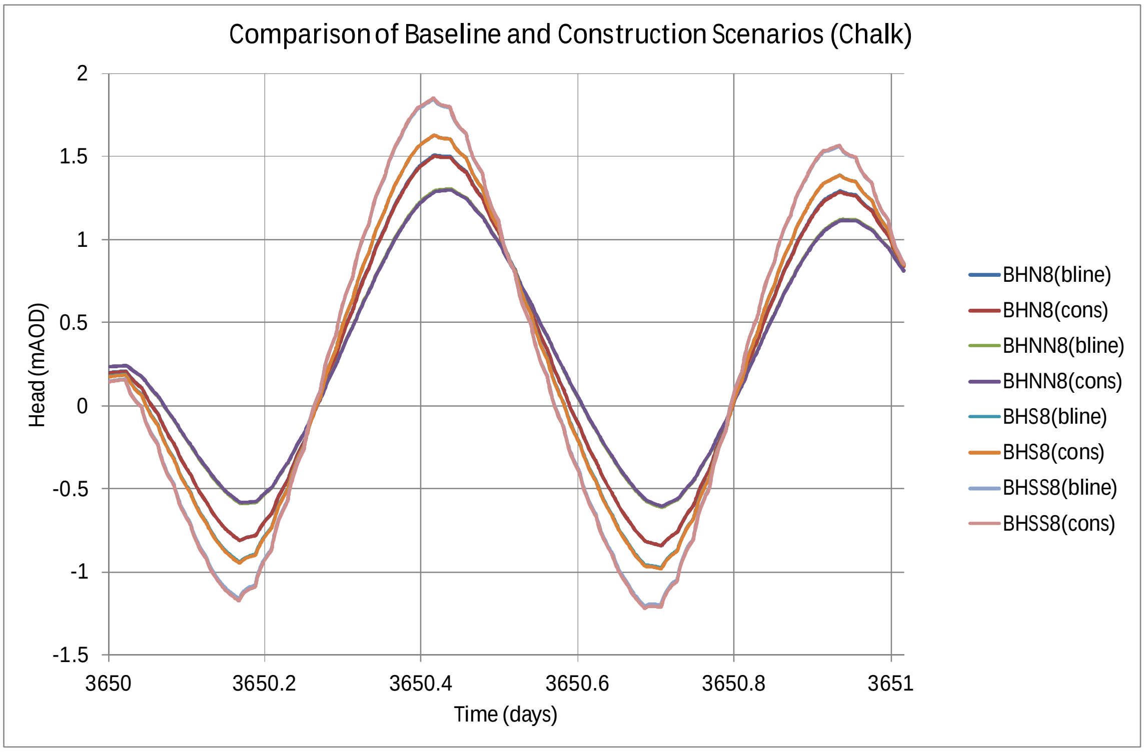


Figure 4-12 Baseline and Construction Phase Hydrographs for Monitoring Points within the Transient Model: Superficial Deposits North of Scheme

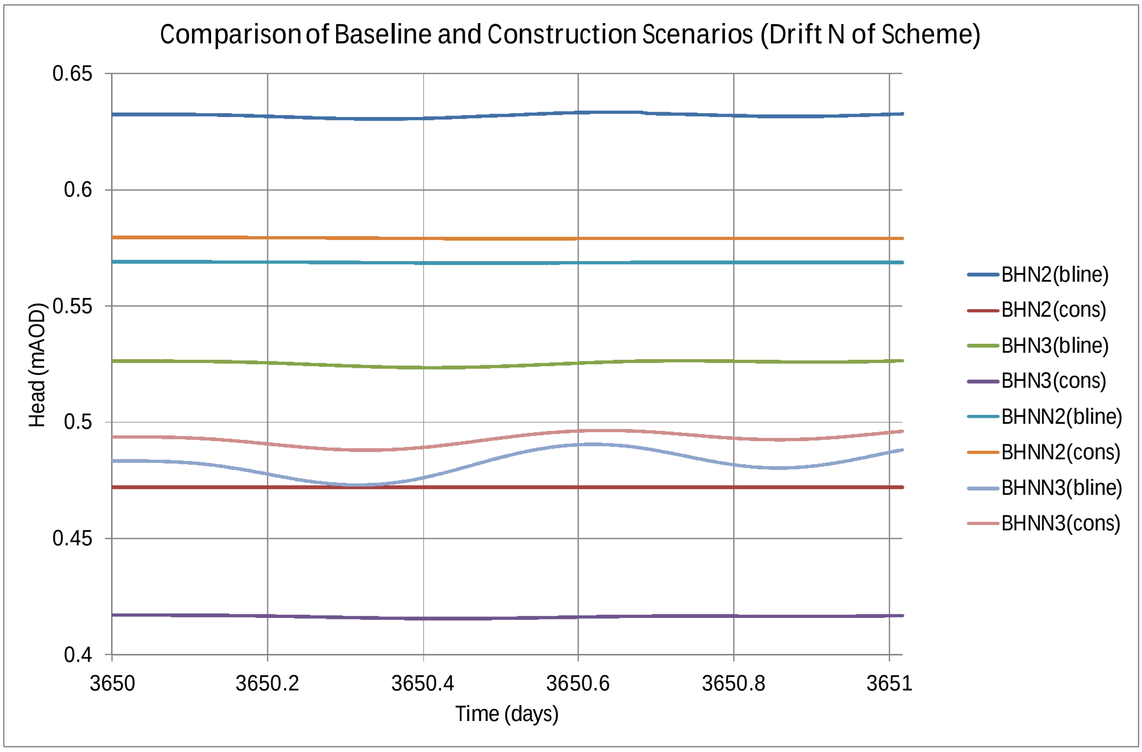
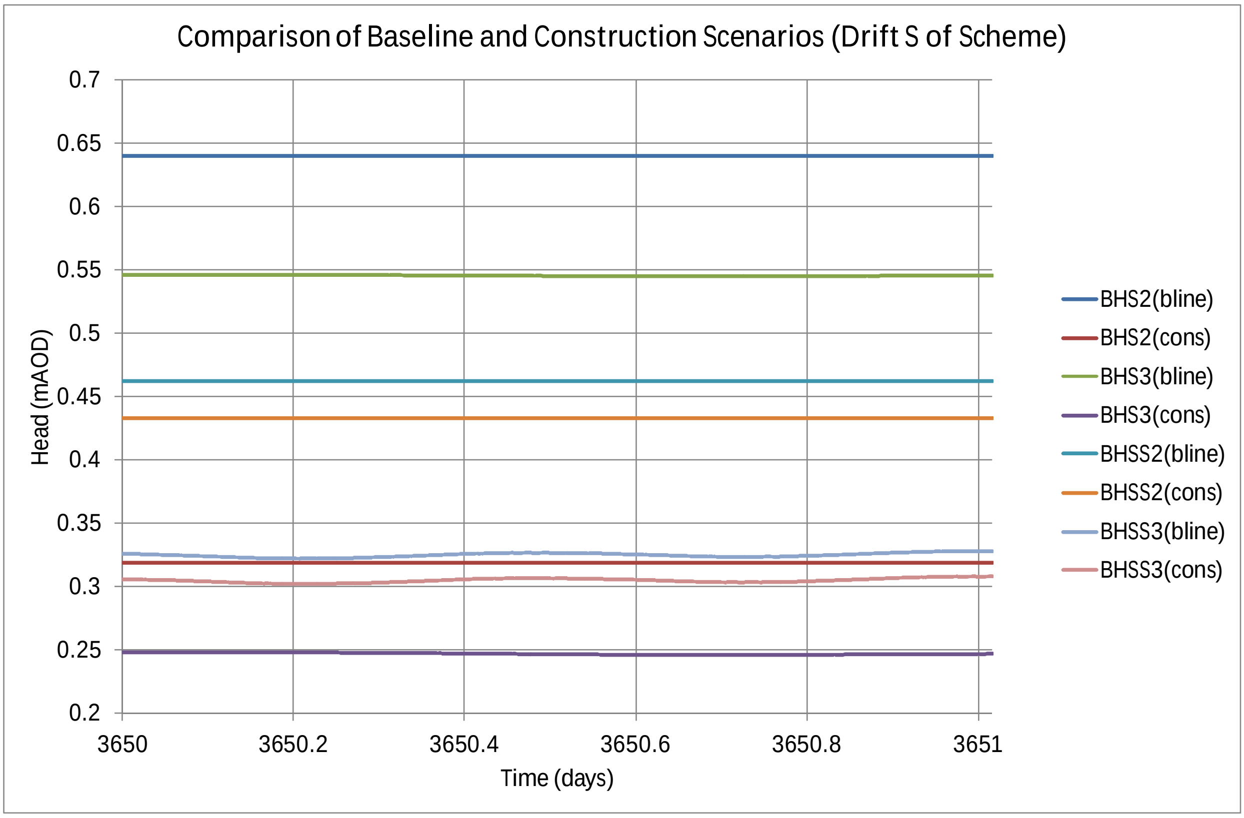


Figure 4-13 Baseline and Construction Phase Hydrographs for Monitoring Points within the Transient Model: Superficial Deposits South of Scheme



4.7 Operation Scenario: Steady-State Model Run

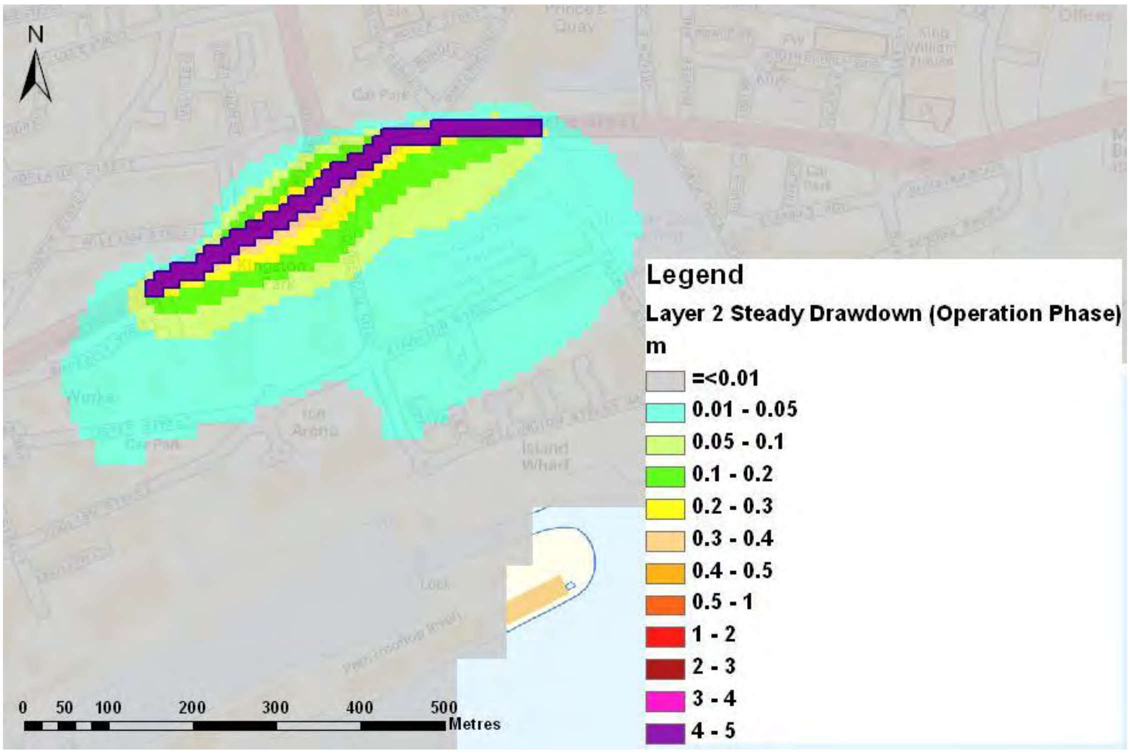
The model was run in steady-state with the operation phase of the scheme represented as described in Section 4.2.2. Figure 4-14 shows the calculated pattern of drawdown of groundwater heads (relative to the baseline scenario) for Layer 2, the Cohesive Alluvium. This is the layer that showed the largest drawdowns.

The main features of Figure 4-14 are:

- The area of significant drawdown is very localised around the scheme.
- Drawdowns are significantly lower than for the construction phase.
- Drawdowns are greatest (between 4.4 and 4.8 m relative to the baseline model) within the secant-piled boundary.
- The extent of drawdown is greater down-gradient (south of the scheme) than up-gradient (north of the scheme), presumably reflecting "damming" of groundwater by the secant pile walls.

Steady-state inflow to the excavation, as measured by the total flow to the drain cells, is about 7 m^3/d. This low rate of inflow reflects the presence of a concrete basal slab (represented by low vertical hydraulic conductivity) as well as the secant pile walls.

Figure 4-14 Modelled Steady-State Drawdown (in Layer 2) for Operation Scenario



4.8 Operation Scenario: Transient Model Run

Figure 4-15, Figure 4-16 and Figure 4-17 compare groundwater level hydrographs for the baseline and the construction phase. Note that the Chalk hydrographs plot on top of each other as there is no change between baseline and construction. Hydrographs for the superficial deposits show small (<0.1 m) changes in average groundwater level between the baseline and operation scenarios. The points south of the scheme all show a fall in average groundwater level from baseline to operation, whereas points north of the scheme show a mixed response (some points show a rise and others a fall).

Figure 4-15 Baseline and Operation Phase Hydrographs for Monitoring Points within the Transient Model: Chalk

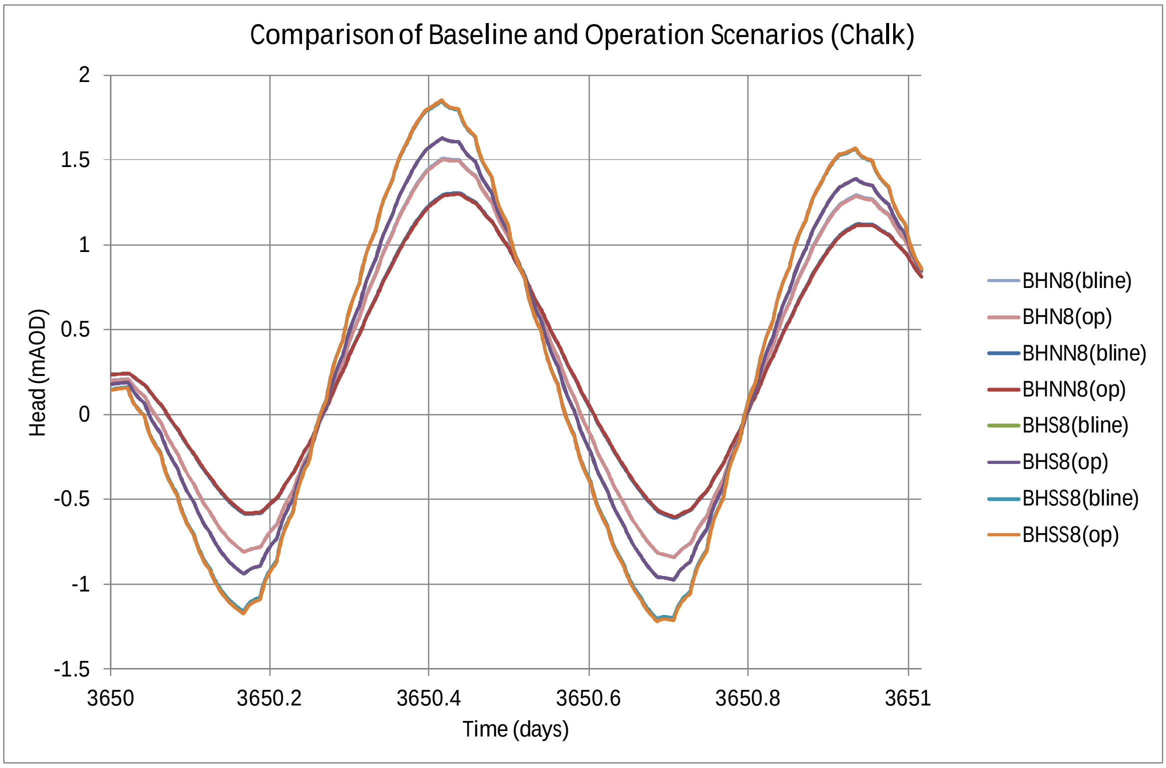


Figure 4-16 Baseline and Operation Phase Hydrographs for Monitoring Points within the Transient Model: Superficial Deposits North of Scheme

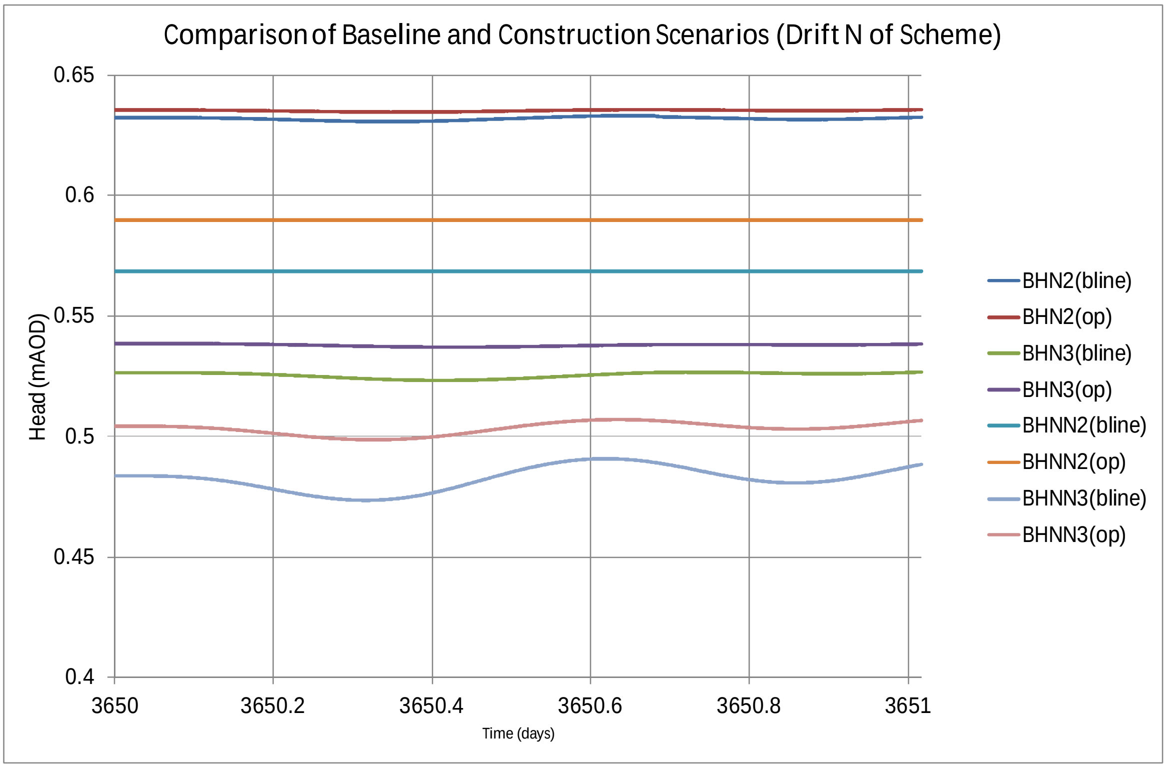
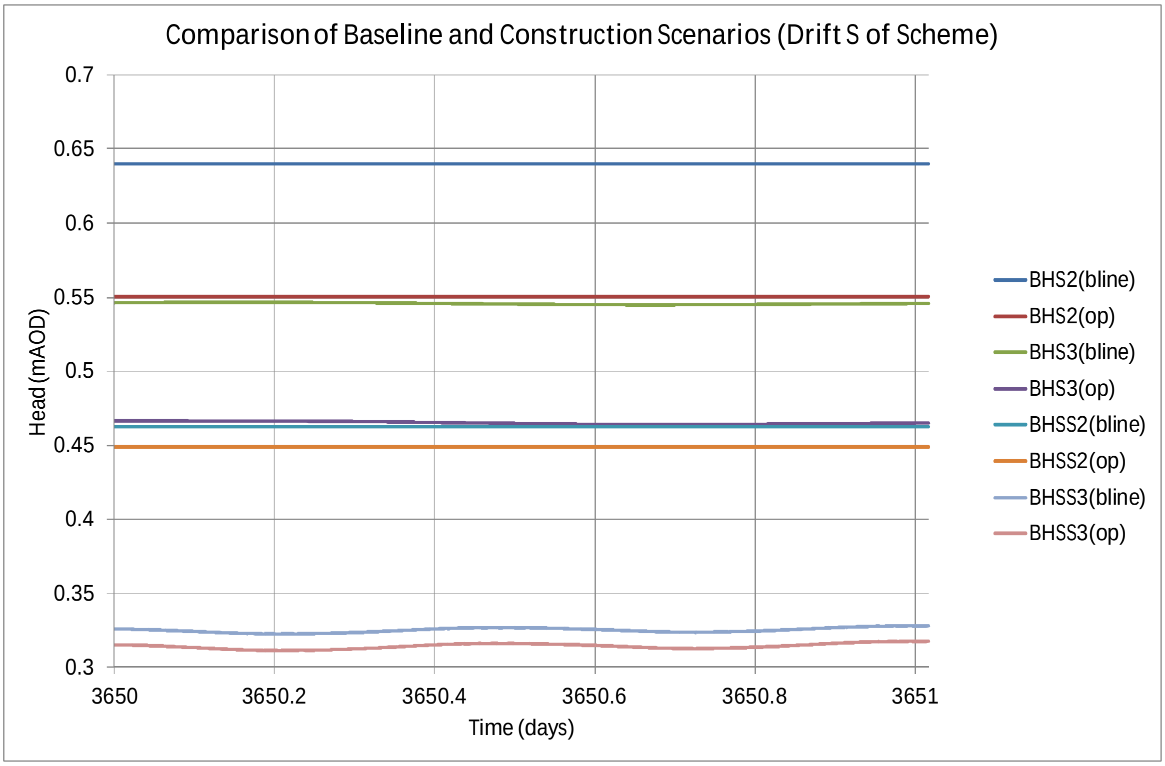


Figure 4-17 Baseline and Operation Phase Hydrographs for Monitoring Points within the Transient Model: Superficial Deposits South of Scheme



5 Discussion of Modelling Results

5.1 Summary of Modelling Results

The steady-state modelling results suggest that the proposed road scheme will lead to a local lowering of average groundwater levels within the superficial deposits. This lowering is predicted to be greatest during the construction phase, and within the secant pile walls. The zone of predicted significant drawdown extends further southwards than northwards, reflecting a slight "damming" of groundwater flow by the scheme. The main quantitative predictions from the steady-state model (which represents average groundwater levels) are:

- Construction phase:
    - Maximum drawdown of 7.2 m in the central part of the scheme, inside the secant pile walls
    - Drawdown immediately outside the secant pile walls less than 0.6 m.
    - Inflow of groundwater to open-based excavation = 13.4 m^3/d.
- Operation phase:
    - Maximum drawdown of 4.8 m in the central part of the scheme, inside the secant pile walls.
    - Drawdowns immediately outside the secant pile walls less than 0.4 m.
    - Inflow of groundwater to road construction = 7 m^3/d.

It should be emphasised that the modelled construction phase scenario is a worst case and that the slight damming effect reflects a regional hydraulic gradient that is a function of the model boundary heads rather than measured heads. No measurable gradients have been identified from groundwater level monitoring within the construction footprint (other than that due to the tidal effect), although this footprint is linear and much smaller than the area of the model, thereby making any low regional gradient hard to detect. Drawdown from an initially flat water table or potentiometric surface (zero gradient) would yield a more symmetrical pattern, i.e. with more drawdown along the northern edge of the road cutting, and there would not be a damming effect.

The transient modelling results, which allow for tidal effects, show slight drawdowns in average groundwater level for the superficial deposits south of the scheme. North of the scheme there are also slight changes, but not always drawdowns. No significant impact is predicted on groundwater heads in the Chalk aquifer, and the tension piling is not predicted to have a significant effect on groundwater flow (regardless of whether piling is into the Chalk or into cohesive superficial deposits).

5.2 Limitations of the Model

The numerical model presented in this report is a highly simplified version of the complex hydrogeological system that exists beneath Hull. The quantitative predictions of the modelling should be considered in the light of the following limitations:

- Within the model the complex geology (especially within the superficial deposits) is simplified into a small number of layers with assigned "bulk" hydraulic properties based on limited information. The BGS Lithoframe model (BGS, 2013), itself a simplification of the geology, had to be further simplified before it could be incorporated into the original MMG JV model that informed the JBA model.
- The active Chalk (Layers 7 and 8) is given a single value for each of Kxy and Kz. In reality, Kx, Ky and Kz are likely to vary spatially as a function of the frequency of fracturing and the orientation and aperture of the fractures. In the model, Kz for the Chalk is two orders of magnitude lower than Kxy. This may well not be realistic (ESI, 2013, assumed Kxy=Kz), although sensitivity analysis showed the results to be insensitive to changes in Kz.
- The Made Ground (Layer 1) is represented as strongly anisotropic. This may not be realistic, except where there is strongly-developed layering. The aquifer properties of this layer were adjusted to match targets that may reflect perched groundwater; as such,

they may not be realistic. However, as Layer 1 is dry across much of the model area, the aquifer properties specified will have had little impact on the results.

- The interactions between groundwater and surface water bodies are imperfectly known and are very roughly represented in the model. The tidal nature of the River Hull is not represented.
- External boundary conditions are very roughly approximated and (with the exception of the southern boundary in the transient model) they are not based on monitoring data. Although most are distant from the area of interest (reducing their influence) they will, nevertheless, have exerted an influence on the solution, particularly on regional hydraulic gradients and flow patterns. The Humber Estuary is very close to the area of interest, so the representation of this boundary (based on a limited understanding of the interaction between groundwater and surface water) will have exerted a significant effect on the results.
- The southern boundary in the deep Chalk (Layer 9) is represented as no-flow; this gives rise to vertical flow at the edges of the model: something that is unlikely to be realistic. However, the deep Chalk layer has a low hydraulic conductivity and so the flows are relatively minor compared to those in the overlying Chalk layers (Layer 8 and Layer 7).
- The model does not consider variations in groundwater density resulting from saline intrusion. Density variations will affect hydraulic heads and hydraulic gradients.
- The calibration of the steady-state model is based on average groundwater levels from different time periods in an area where groundwater levels vary with the tide. Horizontal hydraulic gradients are low, and difficult to calibrate to. Overall, the calibration of the steady-state model is very approximate. It is also likely to be non-unique insofar as different combinations of recharge and hydraulic conductivity could have given the same head distribution.
- The transient calibration is based on a single day's worth of water level data, and the transient simulations also represent just one day. The Chalk responds rapidly to tides in the Humber Estuary. However, it is possible that the slow release/uptake of water from/to low permeability superficial deposits is not properly represented in a model representing this short a time period.
- The docks are not represented at all in the model, either as surface water bodies or as barriers to flow. It may be that they affect groundwater levels by acting as sources and/or sinks or by acting as barriers to groundwater flow. These effects are not represented in the model.
- The road scheme is represented very approximately. The model cells along the road in Layer 1 and Layer 2 have the hydraulic properties of the natural ground. It would be more realistic to give them high hydraulic conductivity to represent rapid runoff and pipe flow associated with the road and its drainage system. However, this is unlikely to have had a large effect as the secant pile walls have a much lower hydraulic conductivity than the natural ground.

6 Conclusions and Recommendations

6.1 Conclusions

Numerical groundwater modelling suggests that the proposed road scheme will modify local groundwater levels within the superficial deposits.

Steady-state simulations predict a lowering of average groundwater levels in the vicinity of the Scheme. This lowering is predicted to be greater during the construction phase than during the operation phase, and greater within the secant pile walls than outside. Maximum drawdown is predicted in the central part of the scheme, within the walls.

The zone of predicted significant drawdown extends further southwards than northwards, reflecting slight "damming" of groundwater flow by the scheme. However, this effect reflects a regional hydraulic gradient that is a function of the model boundary heads rather than measured heads. Monitoring data collected from the construction footprint (which is small compared to the modelled area, linear and orientated perpendicular to the modelled hydraulic gradient) do not provide evidence of a consistent regional hydraulic gradient, and the picture is complicated by tidal fluctuations. The main quantitative predictions from the steady-state model (which represents average groundwater levels) are:

- Construction phase (modelled as a worst case scenario):
  - Maximum drawdown of 7.2 m in the central part of the scheme, inside the secant pile walls
  - Drawdown immediately outside the secant pile walls less than 0.6 m.
  - Inflow of groundwater to open-based excavation = 13.4 m^3/d.
- Operation phase:
  - Maximum drawdown of 4.8 m in the central part of the scheme, inside the secant pile walls.
  - Drawdowns immediately outside the secant pile walls less than 0.4 m.
  - Inflow of groundwater to road construction = 7 m^3/d.

Transient simulations (including tidal fluctuations) reveal a similar picture, with the scheme causing small changes in groundwater level within the superficial deposits in the vicinity of the scheme (although some small rises in groundwater level are predicted up-gradient of the scheme). No significant impact on groundwater heads or flows is predicted for the Chalk Aquifer. In particular, the tension piles are not considered likely to have a significant impact on groundwater flow (regardless of whether piling is into the Chalk or into cohesive superficial deposits).

The model is a simplification of a complex natural system, and is subject to considerable uncertainty. The limitations of the model should be borne in mind when using the results.

6.2 Recommendations

It is recommended that the predicted steady-state drawdowns be examined by a geotechnical engineer and the potential for settlement assessed, taking into account the uncertainty in the modelling results.

It is recommended that additional groundwater monitoring points be installed outside the construction footprint. This would aid understanding of hydraulic gradients and also help model calibration. It is also recommended that the following additional modelling work be considered:

- Representing the docks in the model (following a study of their likely influence on the local hydrogeology).
- Improving the steady-state calibration.
- Increasing the length of the transient calibration and prediction periods and assessing any long-term storage effects.
- Decreasing the anisotropy of the Chalk to make it more realistic.

- Representing variable water density within the model (although given the other uncertainties in the model this would not necessarily make a large difference when considering a small geographical area such as the footprint of the road scheme).
- Representing the Gromtmij pumping test, to see if the model can reproduce the time-drawdown data from the test. This would be a form of model validation.
- If jet grouting is proposed, representing this in the model by modifying the hydraulic properties of the ground beneath the scheme (if the hydraulic conductivity of jet grouted ground is significantly different from that of natural ground). However, it is understood that jet grouted ground has a similar hydraulic conductivity to the alluvium, so it may not need to be represented explicitly in the model.
- A nearby graveyard will require excavating during construction of the scheme. The model could be used to predict the hydrogeological impacts of excavation and dewatering. However, it is considered more beneficial to do this after a ground investigation has been undertaken and the local geology and hydrogeology are better understood.
- The model could potentially be used to investigate potential water quality issues (e.g. using the reactive transport code MT3DMS in concert with MODFLOW).

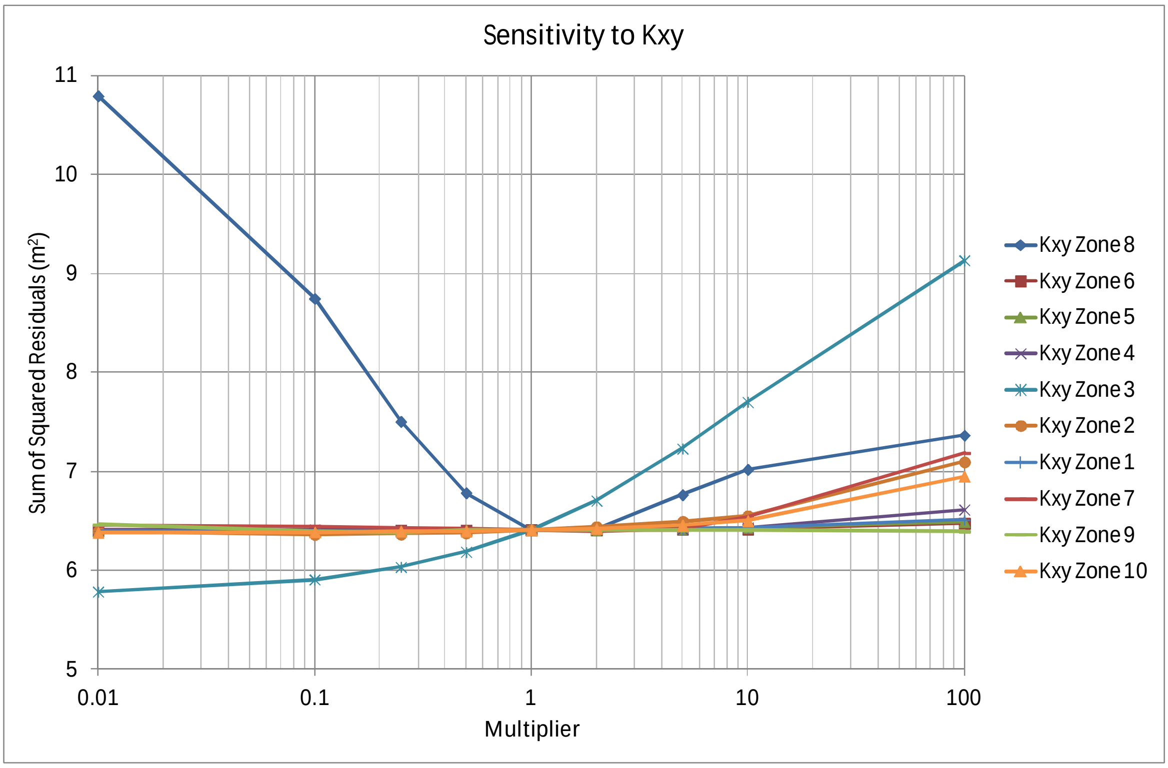
Appendices

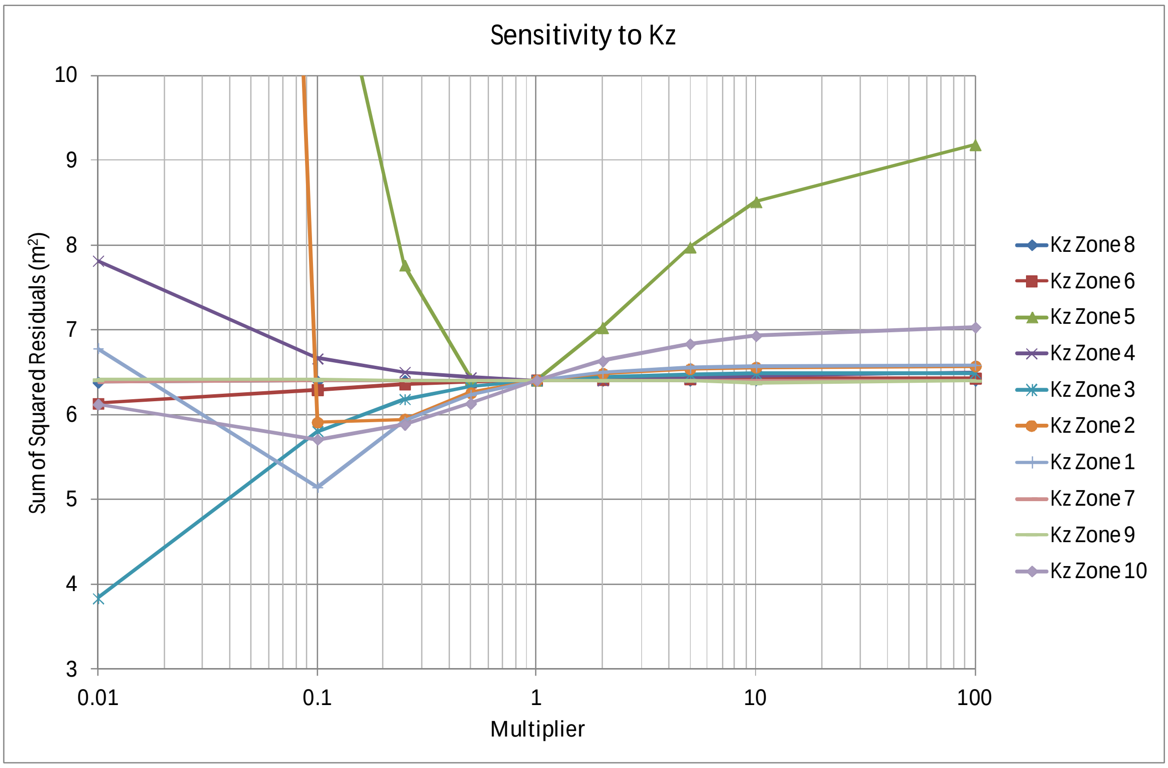
A Sensitivity Analysis

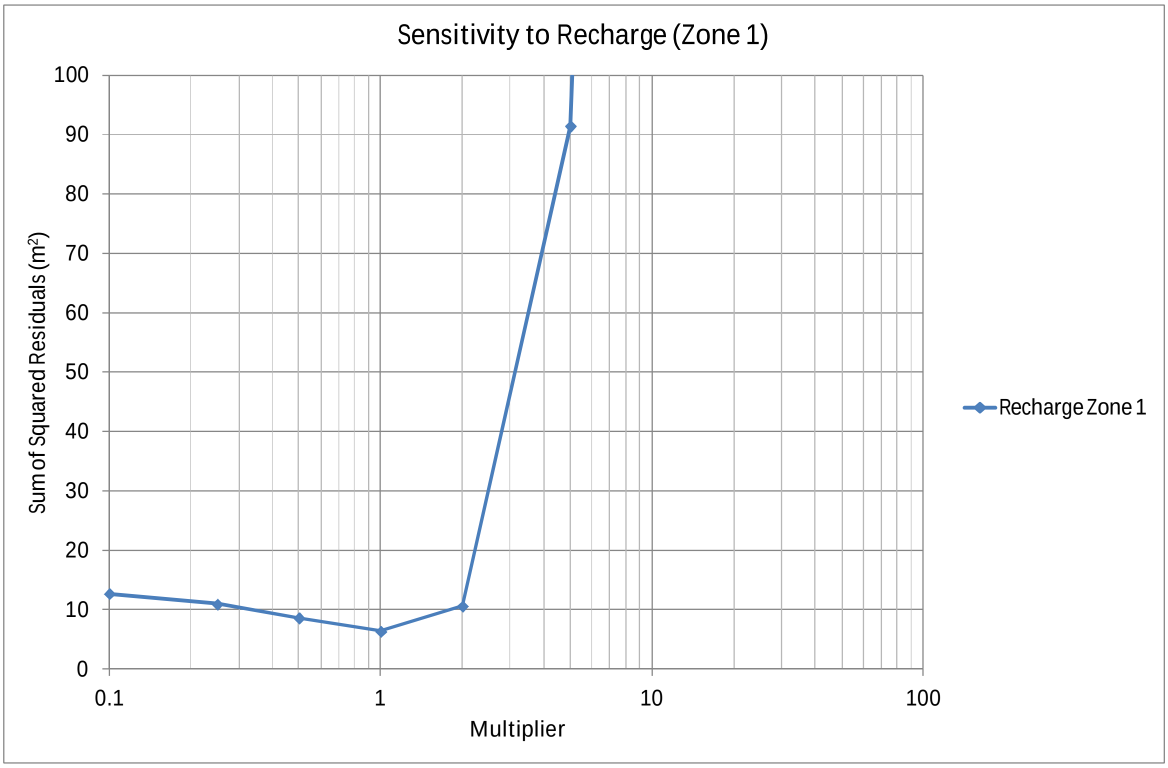
A.1 Sensitivity Analysis Plots

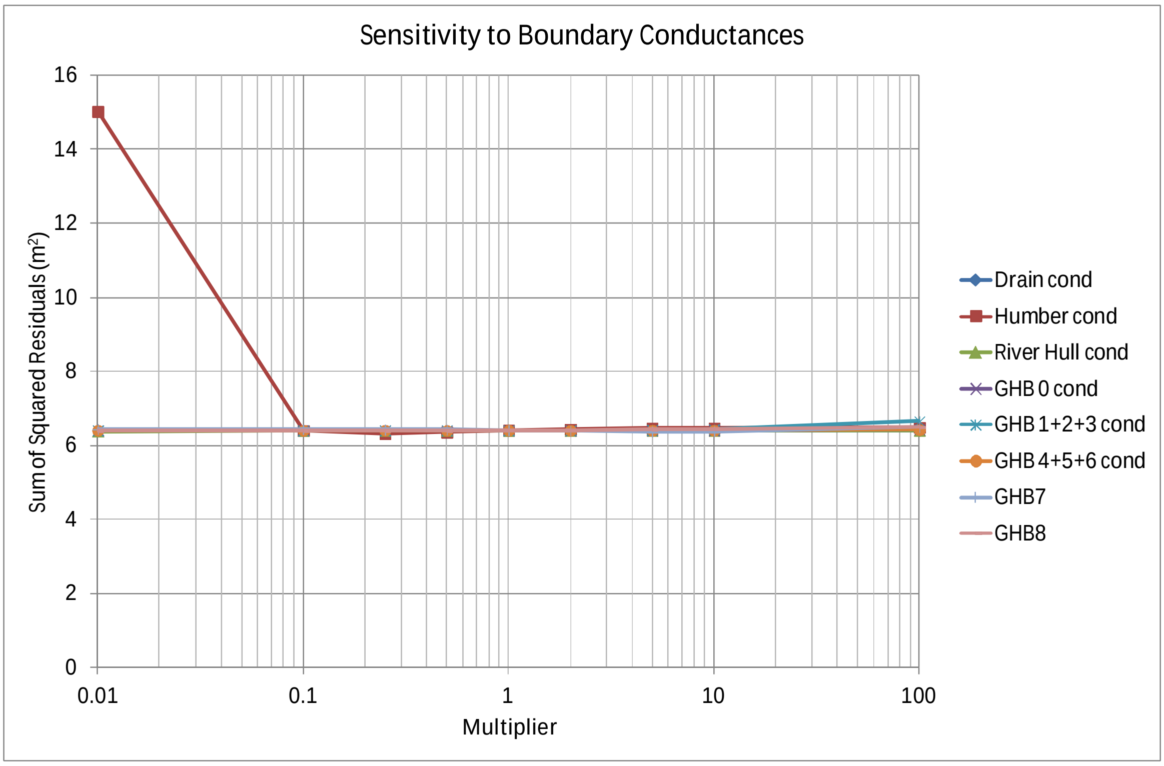
The plots below show the results of the sensitivity analysis in graphical form. Note that the Kxy and Kz values corresponding to a multiplier of 1 were the initial values used, and not the final values given in Table 3-1. These initial values were as follows:

Zone	Kxy [m/d]	Kz [m/d]
1	0.1	0.01
2	0.1	0.01
3	1	0.1
4	0.01	0.01
5	0.01	0.001
6	1	0.1
7	50	50
8	50	50
9	0.01	0.01
10	0.1	0.01




Sensitivity to Kz
10
9
8
7
6
5
4
3
0.01
0.1
1
10
100
Multiplier
Sum of Squared Residuals (m²)
Kz Zone 8
Kz Zone 6
Kz Zone 5
Kz Zone 4
Kz Zone 3
Kz Zone 2
Kz Zone 1
Kz Zone 7
Kz Zone 9
Kz Zone 10


Sensitivity to Recharge (Zone 1)
100
90
80
70
60
50
40
30
20
10
0
0.1
1
10
100
Multiplier
Sum of Squared Residuals (m²)
Recharge Zone 1


Sensitivity to Boundary Conductances
Sum of Squared Residuals (m²)
16
14
12
10
8
6
4
2
0
0.01
0.1
1
10
100
Multiplier
Drain cond
Humber cond
River Hull cond
GHB 0 cond
GHB 1+2+3 cond
GHB 4+5+6 cond
GHB7
GHB8

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A63 Castle Street Improvements, Hull
Environmental Statement

Volume 3 Appendix 11.7
ROAD DRAINAGE AND THE WATER ENVIRONMENT – GROUNDWATER MODELLING REPORT

TR010016/APP/6.3
HE514508-MMSJV-EWE-S0-RP-LE-000006
6 September 2018

A63 Castle Street Improvements, Hull

Environmental Statement

Appendix 11.7 Groundwater modelling report

Revision Record						
Rev No	**Date**	**Originator**	**Checker**	**Approver**	**Status**	**Suitability**
P01.1	02.05.18	C Ball	H Carlyle	J McKenna	S0	For review
P01	31.07.18	C Ball	H Carlyle	J McKenna	Shared	S4
P02	06.09.18	C Ball	H Carlyle	J McKenna	Shared	S4

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1. Introduction

1.1 Background

1.1.1 JBA was commissioned to develop a numerical groundwater flow model to inform the Environmental Statement for the A63 Castle Street Improvements in May 2013, as reported in Volume 3, Appendix 11.6 Groundwater modelling report. The model was used to predict the likely impacts of the Scheme on groundwater levels and flows. The 2014 model was based on the conceptual hydrogeological model and preliminary design prepared by MMG JV[1].

1.1.2 Arup and Balfour Beatty have since been commissioned to take the Scheme forward through detailed design and construction, and a value engineered design has now been produced.

1.1.3 The groundwater flow model has been updated based on updated construction details as provided in the Approval In Principle Reports[2],[3],[4],[5], general arrangement drawings[6] and additional GI data[7],[8],[9].

1.1.4 Additional regional groundwater level monitoring has also been reviewed to assess the model's suitability.

1.1.5 This model update should be read in conjunction with Volume 3, Appendix 11.6 Groundwater modelling report. Full details of the groundwater model are not reproduced in this report.

1.2 Summary of modelling scope

1.2.1 The aims of the groundwater modelling are summarised below.

Construction Phase

[1] From April 2016 Grontmij was rebranded Sweco and MMG JV became MMS JV (Mott Macdonald Sweco Joint Venture)

[2] Arup (2016) A63 Castle Street, Underpass – Approval in Principle, Issue 2

[3] Arup (2016) A63 Castle Street, Holiday Inn Retaining Wall – Approval in Principle, Issue 2

[4] Arup (2016) A63 Castle Street, Porter Street Footbridge – Approval in Principle, Issue 2

[5] Arup (2016) A63 Castle Street Improvement, Hull – Ground Investigation Report, Document Reference HE514508-ARP-SGT-S0-RP-CG-00010. Issue P01, 3 November 2016

[6] Arup (2016) Drawing HE514508-ARP-SSP-SO-ML_PS-SK-CB-000001: Pumping Station General Arrangement at Proposed Location

[7] ESG (2016) A63 Garrison Road, Castle Street Improvement, Hull, Factual Report on Ground Investigation. Report No A5066-15A. For Balfour Beatty Limited and Ove Arup & Partners

[8] ESG (2016) Princess Quay Footbridge, A63 Castle Street Improvement, Hull, Factual Report on Ground Investigation. Report No A5066-15. For Balfour Beatty Limited and Ove Arup & Partners

[9] ESG (2016) Trinity Burial Ground, A63 Castle Street Improvement, Hull, Factual Report on Ground Investigation. Report No A5049-15. For Balfour Beatty Limited and Ove Arup & Partners

1.2.2 Quantify the volume of groundwater likely to flow into the underpass cutting through the diaphragm walls and open base during construction (to inform dewatering plans).

1.2.3 Confirm the likely changes in groundwater levels in the superficial deposits and Chalk, to inform the assessment of impacts on groundwater receptors identified in the Groundwater Report.

Operation Phase

1.2.4 Quantify the volume of groundwater likely to flow into the cutting and subsequently its drainage system through the diaphragm walls and basal slab during operation (to inform drainage plans).

1.2.5 Confirm the likely changes in groundwater levels in the superficial deposits and Chalk, to inform the assessment of impacts on groundwater receptors identified in the Groundwater Report.

1.2.6 Confirm the likely impact of tension piles on groundwater levels and flows.

2. Conceptual model update

2.1 Hydrogeological conceptual model update

2.1.1 The additional GI undertaken in 2016 confirmed the conceptual model presented in Volume 3, Appendix 11.6 Groundwater modelling report.

2.1.2 Additional groundwater level monitoring is available within the model area from YW saline intrusion boreholes between February and December 2017 as well as continued monitoring from the Chalk GI boreholes.

2.1.3 The additional groundwater level data has been used to verify agreement with modelled heads and hydraulic gradients within the steady state model. Source protection zones were updated by the Environment Agency since the model was originally built in 2014, with the study area now falling within the SPZ3 for a group of public water supply boreholes to the west and northwest of Hull. This suggests that the hydraulic gradients with the Environment Agency's regional groundwater model (which the 2014 groundwater flow model was based upon) may have been updated.

2.1.4 The 2014 steady state model was based on average tidal levels.

2.1.5 presents modelled groundwater heads in the uppermost Chalk layer (Layer 7) for the 2014 model baseline scenario along with average groundwater levels observed in the saline intrusion and GI boreholes.

2.1.6 This shows that modelled and observed Chalk heads within the Scheme Site Boundary and the immediate area are generally in agreement, with modelled groundwater heads within +/- 0.4m.

2.1.7 Figure 11.7.2 presents groundwater level monitoring undertaken at all Chalk GI boreholes at low tide and high tide on 28 February 2017. This shows there to be a northwards hydraulic gradient at high tide and a southwards hydraulic gradient at low tide, in agreement with the 2014 transient model, and as discussed in Volume 3, Appendix 11.6 Groundwater modelling report.

Figure 11.7.1: Baseline modelled and observed heads

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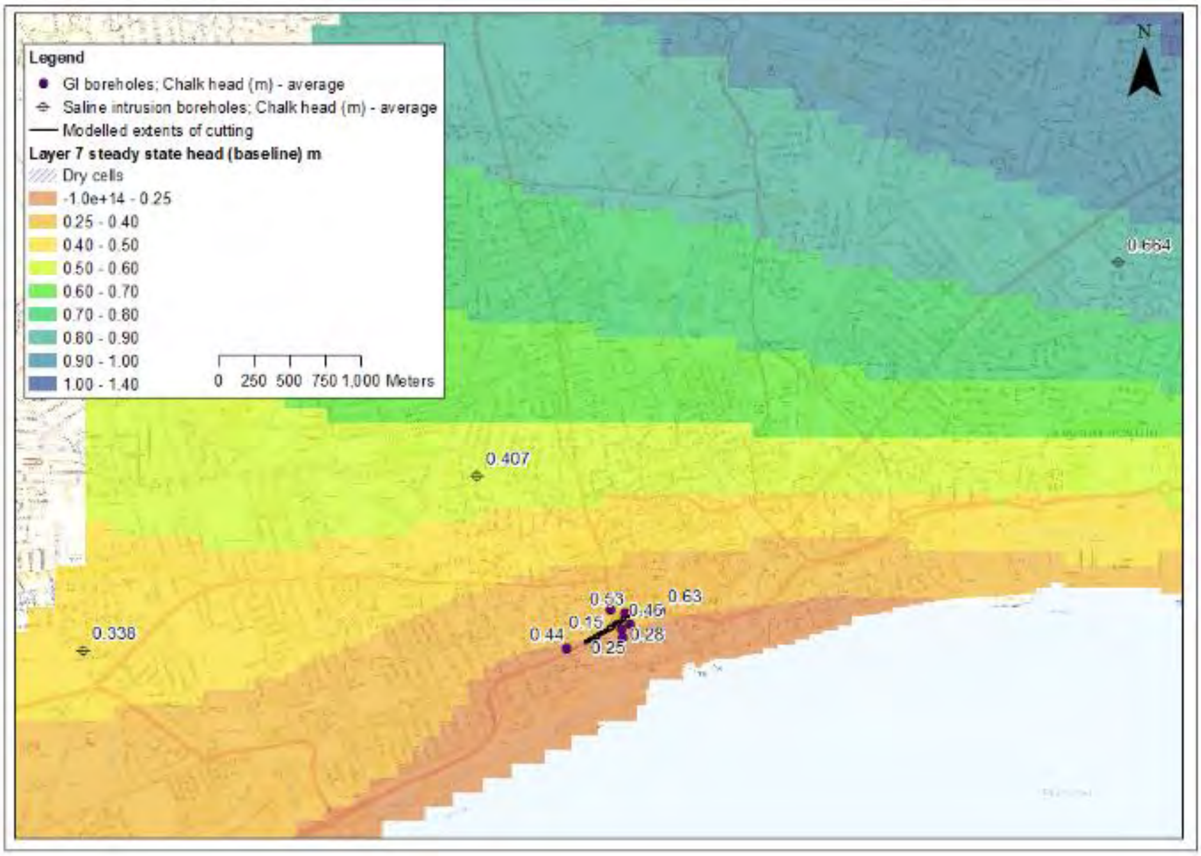
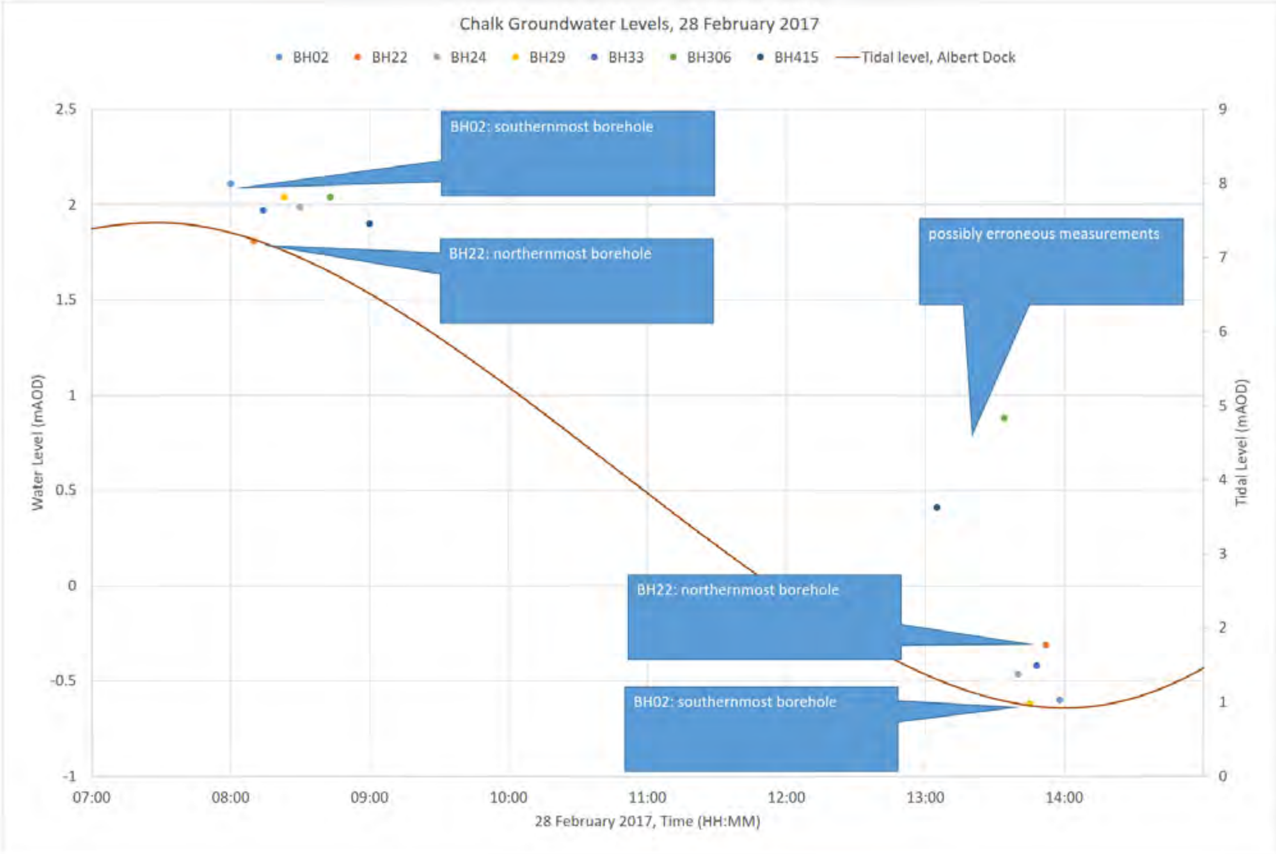


Figure 11.7.2: Chalk groundwater levels, 28 February 2017



2.2 Scheme design

Overview

2.2.1 The following changes to the proposed design are based on information provided in the Approval in Principle (AIP) reports.

Underpass

2.2.2 The secant piles previously forming the retaining walls of the underpass cutting have been replaced with diaphragm walls.

2.2.3 These diaphragm walls are created in sections, with inter-joining water bars to ensure Class 1 'water tightness' (leakage limited to be a small amount, some surface staining or damp patches are acceptable) in accordance with BS EN1992-3:2006. This has been interpreted as creating an impermeable barrier for the purposes of the model.

2.2.4 The diaphragm walls extend to a maximum depth of -27.5m AOD directly below the Mytongate Junction, which is 5.5m below the top of the Chalk. To the west of the junction, the diaphragm walls extend to a depth of -20m AOD (2m below the top of the Chalk), while to the east of the junction, they extend to a depth of -22m AOD (1m below the top of the Chalk). The diaphragm wall thickness is 1m.

2.2.5 The diaphragm walls extend from chainage 1310m to 1670m. The secant piled wall extents were greater than this, from around chainage 1280m to 1750m.

2.2.6 The underpass will be constructed using a top down method. Once the diaphragm walls are in place, the jet grout layer is injected from current ground level to the required depth and thickness, reinforced concrete bored tension piles are bored through the pre-installed jet grout layer and further guide walls and additional piles for the Mytongate Bridge piers are constructed. The ground is then excavated to expose the underpass ground slab formation level (i.e. the top of the jet grout layer), allowing for installation of temporary props where required. This top down method potentially removes the need for dewatering.

2.2.7 The jet grout layer is primarily installed for ground stabilisation purposes, but may also locally reduce the permeability of the ground. This has been interpreted as having the same permeability as the surrounding ground (i.e. the cohesive alluvium) for the purposes of the model.

2.2.8 A waterproof coating will be applied to the ground slab in accordance with DMRB Volume 2, Section 3 Parts 4 and 5, BA 47/99 and BD 47/99: Water proofing and surfacing of concrete bridge decks[10]. This has been interpreted as creating an impermeable barrier for the purposes of the model.

Underpass tension piles

2.2.9 The underpass tension piles extend to a maximum depth of -27m AOD, which is slightly less than the maximum depth of the diaphragm walls. As the diaphragm walls are roughly perpendicular to the hydraulic gradient and would create a potentially more significant barrier to flow than the tension piles, the latter have not been included in the groundwater model. Moreover, the model grid spacing is too coarse in comparison to the cross-sectional area of the tension piles, which means that the latter cannot be appropriately represented in the model. A separate, 2d model constructed as part of the 2014 investigation demonstrated that the tension piles would not have an adverse impact on groundwater levels and flows. This is discussed in detail in Volume 3 Appendix 11.6 Groundwater modelling report.

Holiday Inn retaining wall

2.2.10 The Scheme includes a sheet piled retaining wall to separate the westbound diverge slip road and the Holiday Inn.

2.2.11 The maximum predicted toe depth for the retaining wall is -18.5m AOD. Note that this depth is only indicative and toe levels are to be optimised along the wall's length during the detailed design stage.

---

[10] The Highways Agency, Scottish Government, Welsh Assembly Government Llywodraeth Cynulliad Cymru and the Department for Regional Development Northern Ireland (1999) Design Manual for Roads and Bridges Volume 2 Highway Structures: Design (Sub Structures and Special Structures) Materials, Section 3 Materials and Components, Part 4 BD47/99 Waterproofing and Surfacing of Concrete Bridge Decks

2.2.12 The retaining wall extends from chainage 1610m to 1760m, and so broadly coincides with the eastern end of the underpass.

2.2.13 The retaining wall was not considered in the 2014 model.

Pumping station

2.2.14 The pumping station is to include a 14.09m diameter, cylindrical, below ground tank. The pumping station is to be formed by secant piled walls to a depth of -27.5m AOD, with tension piles to a toe level of -28.5m AOD.

2.2.15 The pumping station is to be located in the grassed area around 10m to the southeast of the Mytongate Junction, at an approximately equivalent chainage of 1520m.

2.2.16 The pumping station was not considered in the 2014 model. Depths and location are sufficiently similar to the placement of the diaphragm walls and therefore no additional structure is included in the model to represent the pumping station.

Bridge tension piles

2.2.17 The design of the Porter Street and Princes Quay pedestrian, cycle and disables user bridges include tension piles into the Chalk. Limited details on the tension piles, based on the concept design, have been used to model the potential impact of the general arrangement of bridge tension piles. Two 1m diameter reinforced concrete bored piles are to be placed with a 3m spacing approximately every 13.78m along the 62.46m bridge ramps. The sets of piles are orientated approximately parallel to groundwater flow, except those located at either end of the bridge itself.

2.2.18 Pile depths have not been confirmed, and an assumed depth of -27.5m AOD has been used.

3. Modelling approach

3.1 Numerical modelling code

3.1.1 The model was produced using the United States Geological Survey (USGS) code MODFLOW with the Groundwater Vistas v6 interface[11]. The 2014 model used Groundwater Vistas Version 6.53, Build 8[12], whereas the model update uses Version 6.94, Build 12.

3.2 Baseline model

3.2.1 As discussed in Section 2, the conceptual model has not been changed from that presented in the 2014 model report and therefore no changes have been made to the baseline model design, calibration or sensitivity analysis.

3.3 Construction and Operation phase scenarios

3.3.1 Parameter updates made to the Construction and Operation Phase model scenarios are summarised in Table 11.7.1, along with a justification for the changes made.

Table 11.7.1: Construction and Operation Phase model parameter updates

Model element	Parameter	2014 model value	Model update value	Model scenario	Justification
Wall Cells	Wall thickness	0.9m	1m	Construction & Operation phases	Diaphragm wall thickness up to 1m.
	Hydraulic conductivity, K	8.6×10^{-5} m/d	0 m/d	Construction & Operation phases	Diaphragm walls 'impermeable'
	Spatial Extents	-	As per 2014 model	Construction & Operation phases	Although underpass extents are reduced, wall cells remain unchanged to reflect inclusion of Holiday Inn retaining wall
Chalk (Layer 7)	Bottom elevation	4m below bottom elevation of Layer 6	5.5m below bottom elevation of Layer 6	Construction & Operation phases	Diaphragm walls extend up to 5.5m into the top of the Chalk.
Hydraulic conductivity, K zone 11	Kz	8.6×10^{-5} m/d	0 m/d	Operation phase only	Waterproof coating applied to ground slab.

[11] McDonald, M.G. and Harbaugh, A.W., 1984. A modular three-dimensional finite-difference groundwater flow model. U.S. Geological Survey Open-File Report 83-875, 528pp

[12] ESI (2011) Guide to using Groundwater Vistas Version 6

3.3.2 Only the steady state model scenarios have been updated. The 2014 transient model results were used only to confirm that the model accurately reproduced variations in groundwater levels as a result of tidal impacts. This has not changed.

3.4 Bridge tension pile model

3.4.1 The 2014 tension pile model was amended to assess the potential impact of the bridge piles. Parameter updates were made to all modelled scenarios, as summarised in Table 11.7.2.

Table 11.7.2: Tension pile model parameter updates

Model Element	Parameter	2014 Model Value	Model Update Value	Justification
Model grid	Dimensions	18.5m x 20m	100m x 100m	Dimensions to allow for spatial extents of the bridge
	Grid spacing	0.5m	1m	Grid spacing equivalent to diameter of piles
Layer 1	Bottom elevation	-24m	-25m	Assumed bridge pile depth
Northern constant head boundary	head	1m	0.5m	To better match the hydraulic gradient of the 2014 baseline model (hydraulic gradient: 0.005m)
Pile hydraulic conductivity, K	K zone 2	8.6×10^{-5} m/d	0 m/d	To represent concrete (as assumed in updated model construction and operation scenarios).
Surrounding ground hydraulic conductivity, K	K zone 1	-	10 m/d	Additional hydraulic conductivity value to assess the potential impact of piles within the granular alluvium. Values of 0.05 m/d, 0.01 m/d and 75 m/d were also used to represent the cohesive alluvium, glaciolacustrine deposits and Chalk respectively, as per the 2014 model.

3.4.2 The hydraulic gradient across the model was initially set at 0.0003 (to match the hydraulic gradient simulated baseline model) by adjusting the northern constant head boundary to 0.05m. However, this caused convergence issues and therefore this northern constant head boundary was subsequently set to 0.5m.

3.4.3 Sensitivity analysis was run on the Chalk scenario for higher hydraulic gradient values of 0.01m and 0.04m. This was done by adjusting the northern constant head boundary to 1m and 4m.

4. Model update results

4.1 Construction and Operation Phase scenarios

4.1.1 Table 11.7.3 summarises the model results for both the Construction and Operation phases. Figure 11.7.3 to Figure 11.7.10 inclusive present drawdown for layers 2, 3, 7 and 8 for the Construction and Operation Phase scenarios. The nearest buildings to the Scheme (William Booth House and the Whittington and Cat public house) are also shown in Figure 11.7.3 and Figure 11.7.7. Figure 11.7.11 and Figure 11.7.12 present mass balance results for Layer 2 for the cutting area, showing inflows into the underpass (modelled as a drain).

Table 11.7.3: Model results – Construction and Operation Phase scenarios

	Construction Phase			Operation Phase		
Receptor	Layer 2 (cohesive alluvium)	Layer 3 (granular alluvium)	Layer 7 (uppermost Chalk)[1]	Layer 2 (cohesive alluvium)	Layer 3 (granular alluvium)	Layer 7 (uppermost Chalk)[1]
Drawdown (m)						
Within underpass cutting (max value)	7.01	1.70	0.04	4.82	0.48	<0.01
Outside cutting – S (max value)	0.13	0.13	0.04	0.13	0.13	0.03
Outside cutting – N (max value)	-0.12	-0.14 *	-0.02	-0.13	-0.14 *	-0.03
William Booth House (N of cutting)	-0.04	-	-	-0.04	-	-
Whittington & Cat public house (S of cutting)	0.04	-	-	0.04	-	-
Groundwater Flow (m^3/d)						
Inflow to underpass cutting	9.2	-	-	1.36	-	-

[1] Layer 7 (uppermost Chalk) represents the chalk intersected by the diaphragm wall

4.1.2 The extent of the area impacted by dewatering (assume drawdown <+/- 0.01m) within Layer 2 to the south of the Scheme is similar to that modelled previously, although now a zone of negative drawdown (groundwater mounding) extends to the north of the Scheme. This arises because the diaphragm walls are assumed to be impermeable, whereas the secant piles incorporated into the 2014 model were assumed to be leaky.

4.1.3 For both the Construction and Operation Phase scenarios, the zone of influence to the south of the cutting is greater in Layer 3 (granular alluvium) than in Layer 2 (cohesive alluvium), although maximum drawdown values are not as great.

4.1.4 Figure 11.7.6 and Figure 11.7.10 show that there is negligible drawdown or groundwater mounding (<0.01m) in Layer 8, the main Chalk (more productive Chalk below the base of the diaphragm wall).

Figure 11.7.3: Drawdown in Layer 2 (cohesive deposits), Construction Phase scenario

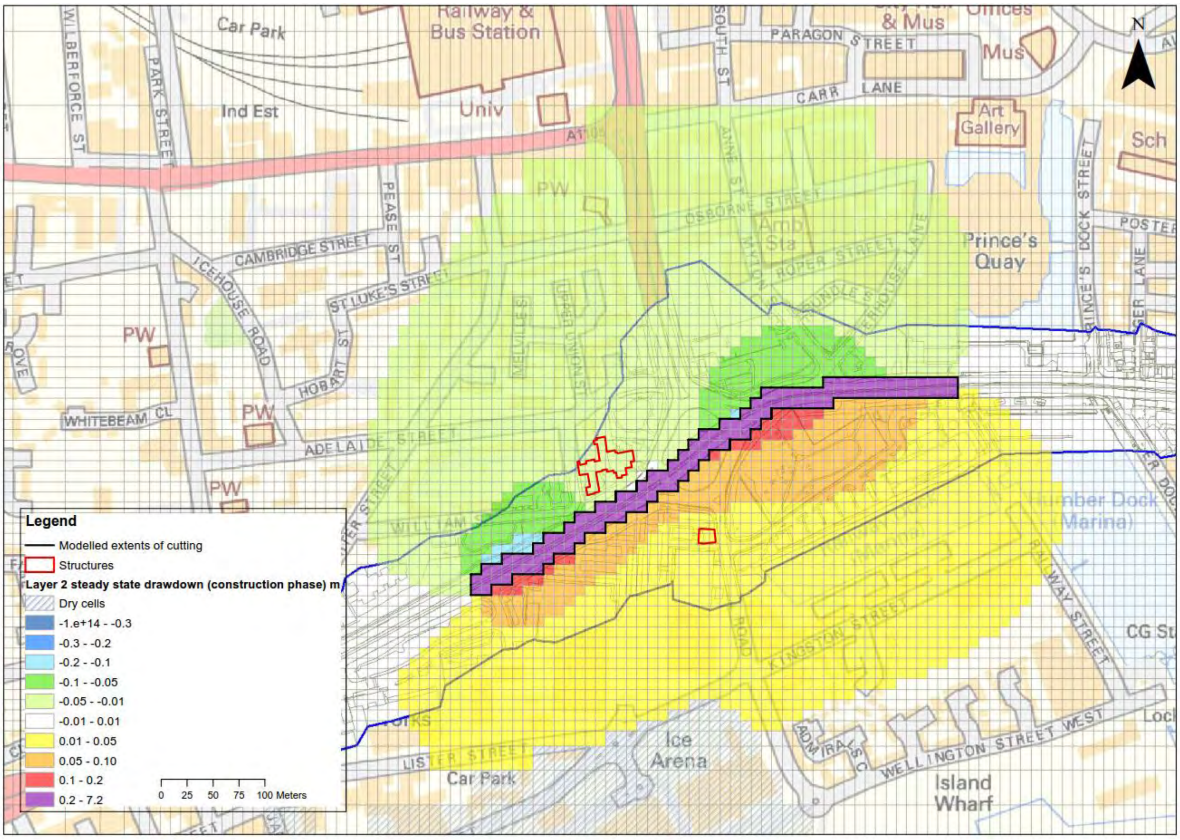


Figure 11.7.4: Drawdown in Layer 3 (granular alluvium), Construction Phase scenario

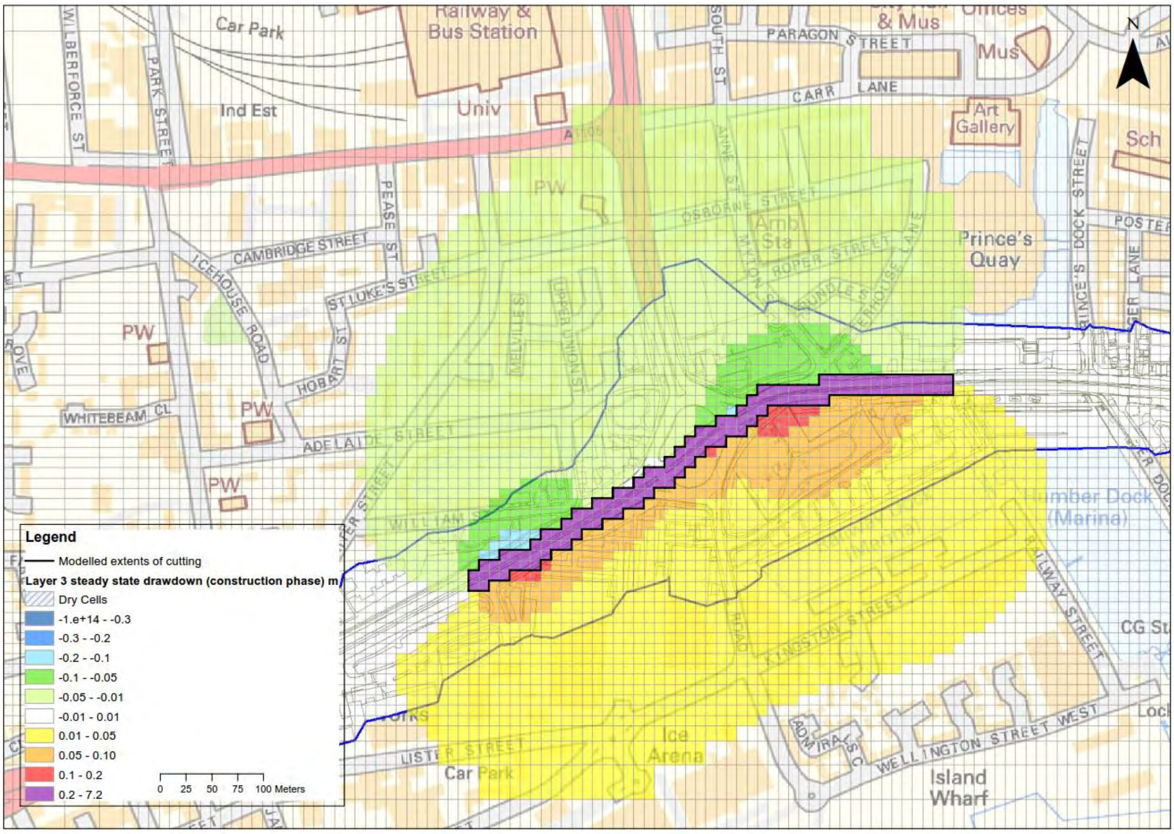


Figure 11.7.5: Drawdown in Layer 7 (uppermost Chalk), Construction Phase scenario

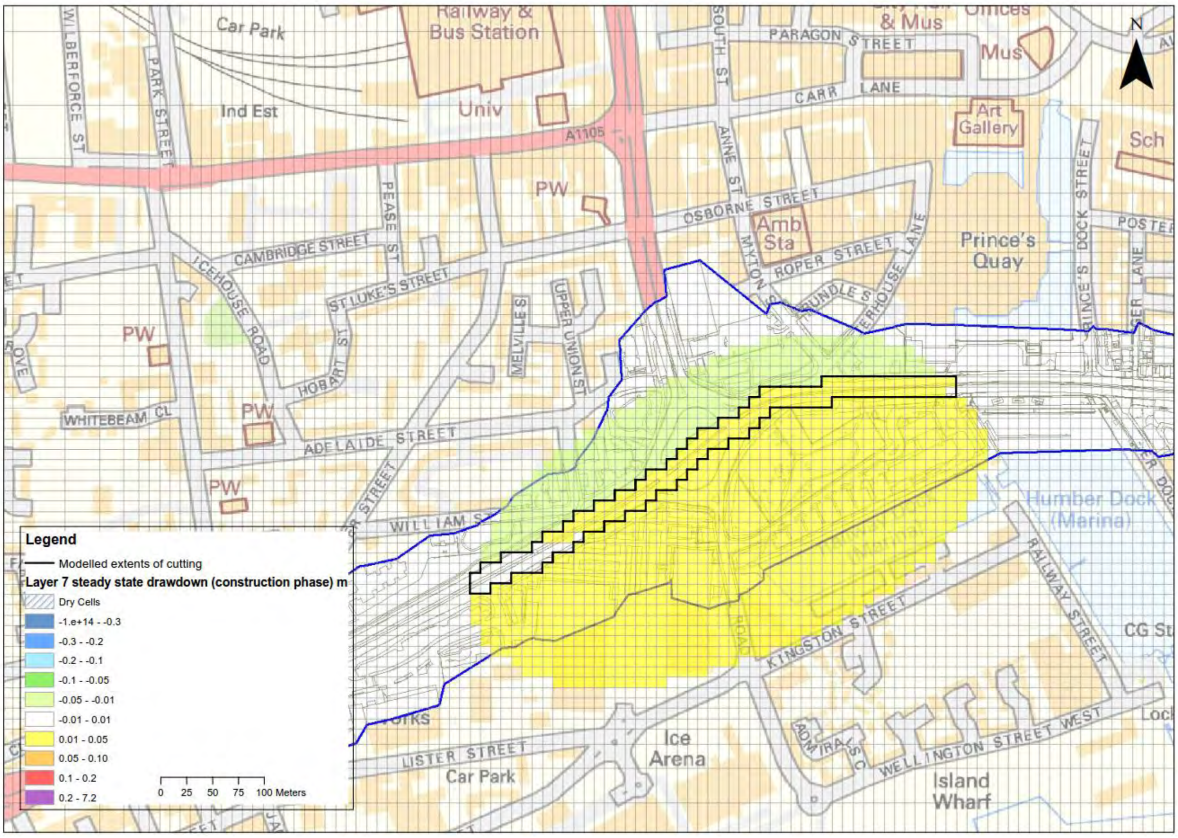


Figure 11.7.6: Drawdown in Layer 8 (main Chalk), Construction Phase scenario

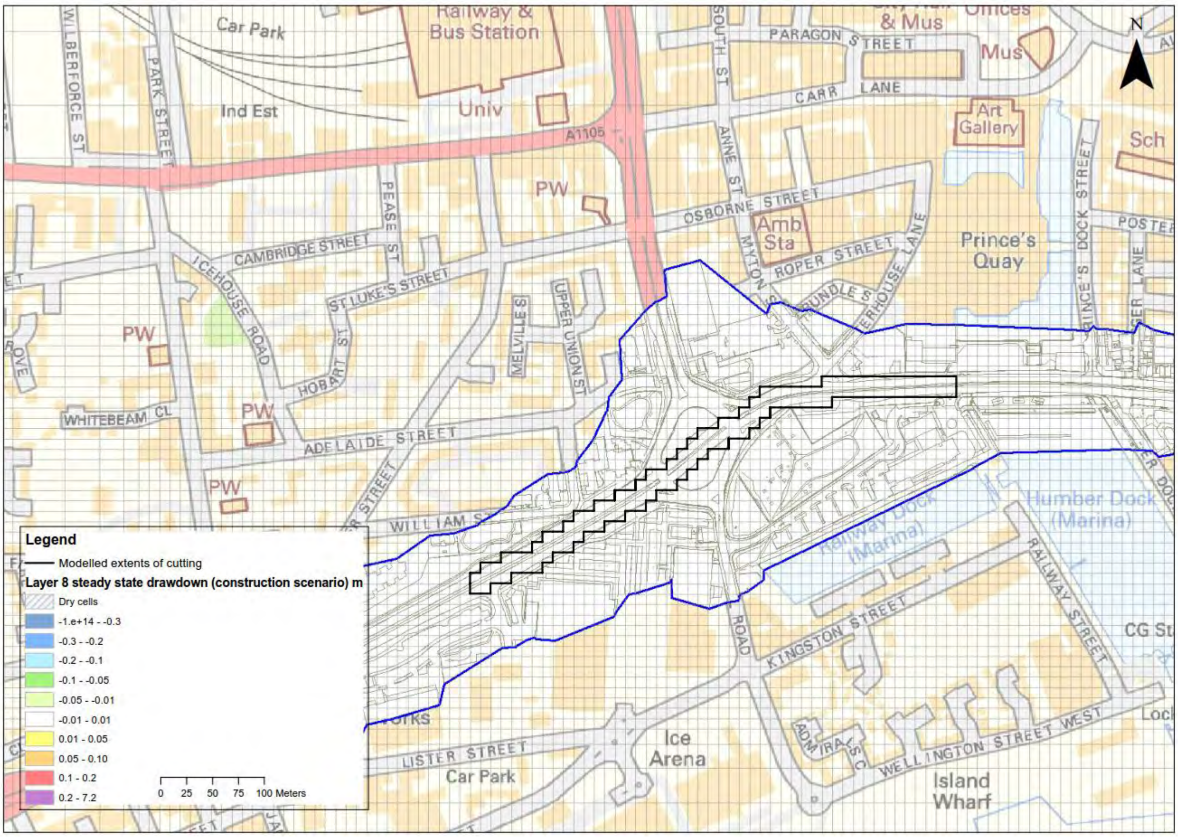


Figure 11.7.7: Drawdown in Layer2 (cohesive deposits), Operation Phase scenario

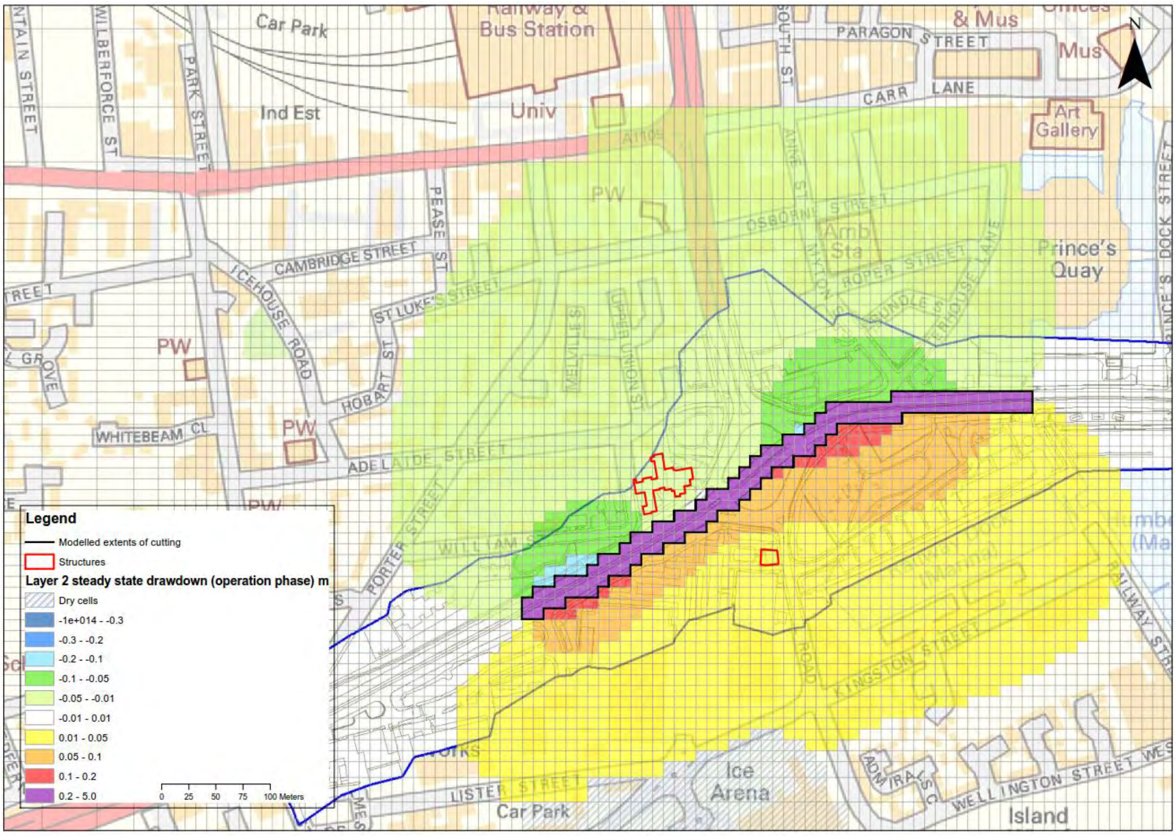


Figure 11.7.8: Drawdown in Layer 3 (granular alluvium), Operation Phase scenario

Figure 11.7.9: Drawdown in Layer 7 (uppermost Chalk), Operation Phase scenario

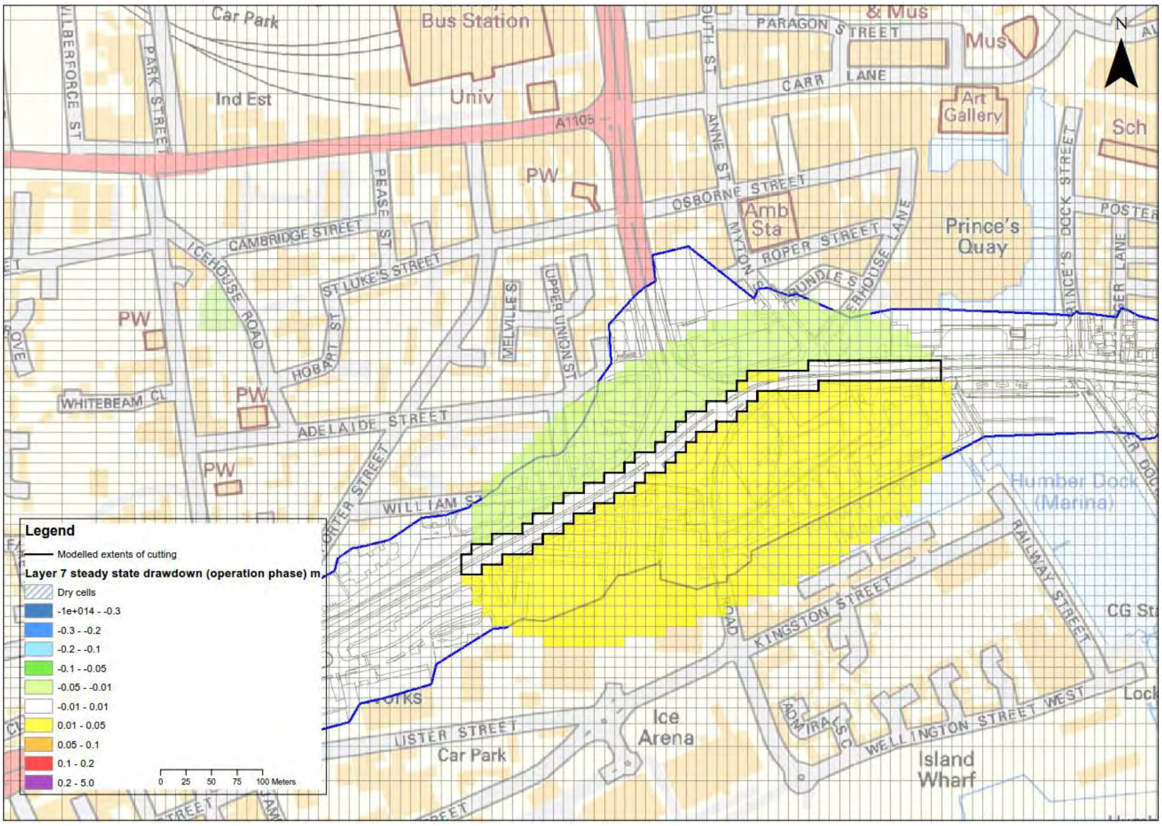


Figure 11.7.10: Drawdown in Layer 8 (main Chalk), Operation Phase scenario

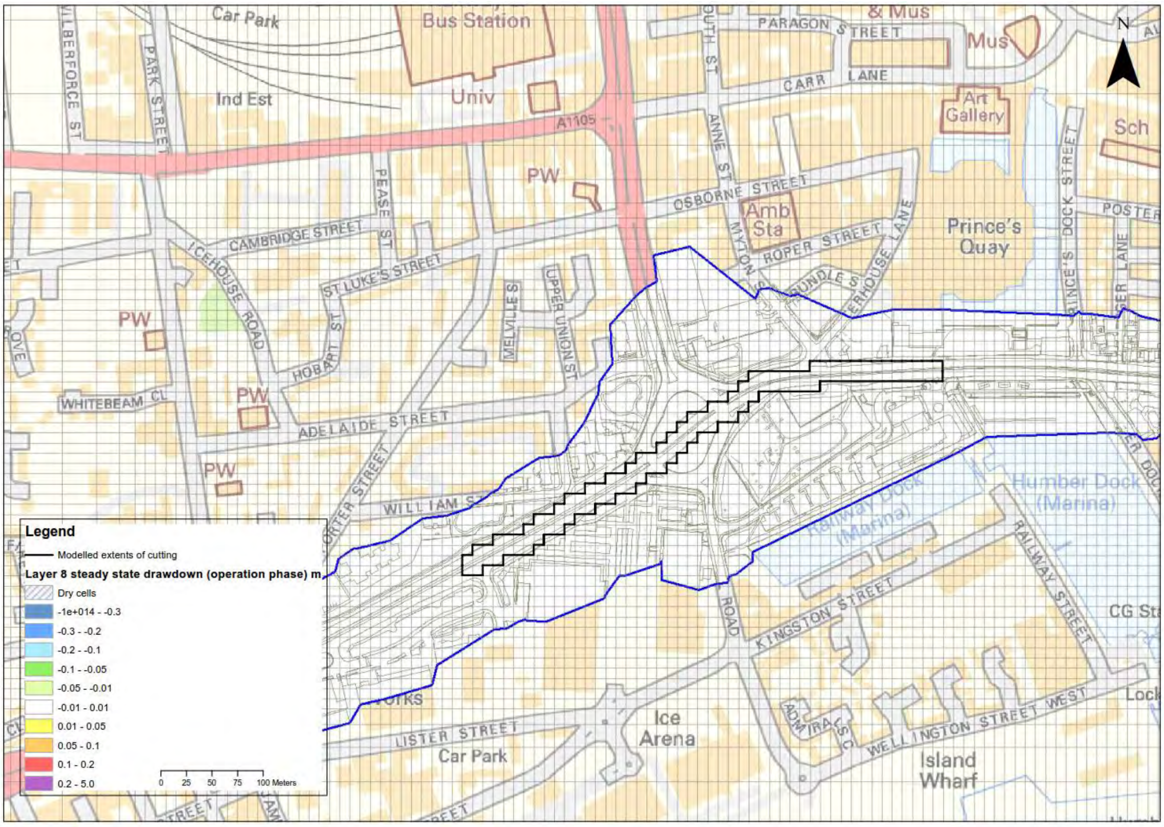
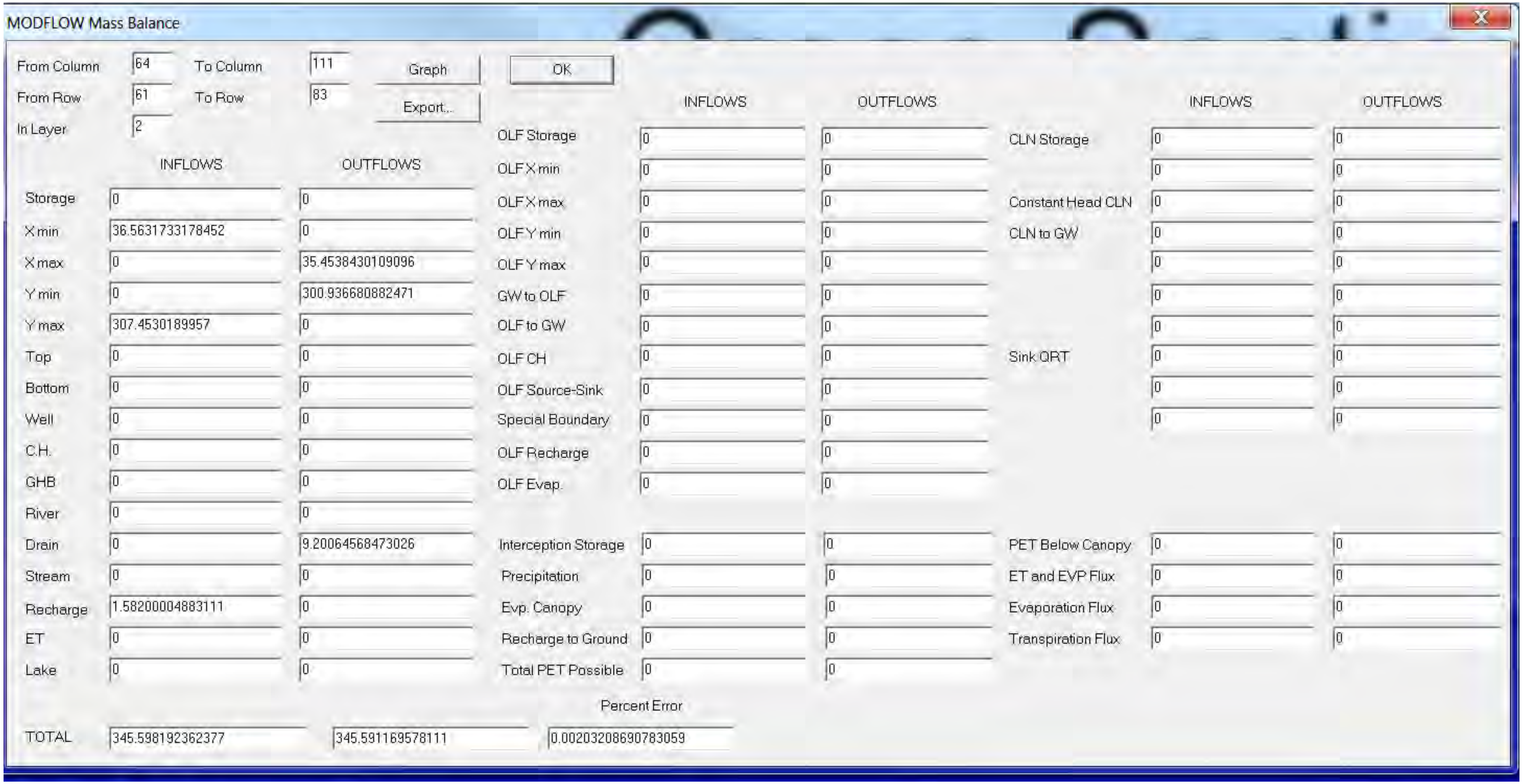


Figure 11.7.11: Mass balance results for all layers within the underpass cutting area, Construction Phase scenario



MODFLOW Mass Balance

From Column 64 To Column 111 Graph OK
From Row 61 To Row 83 Export...
In Layer 2

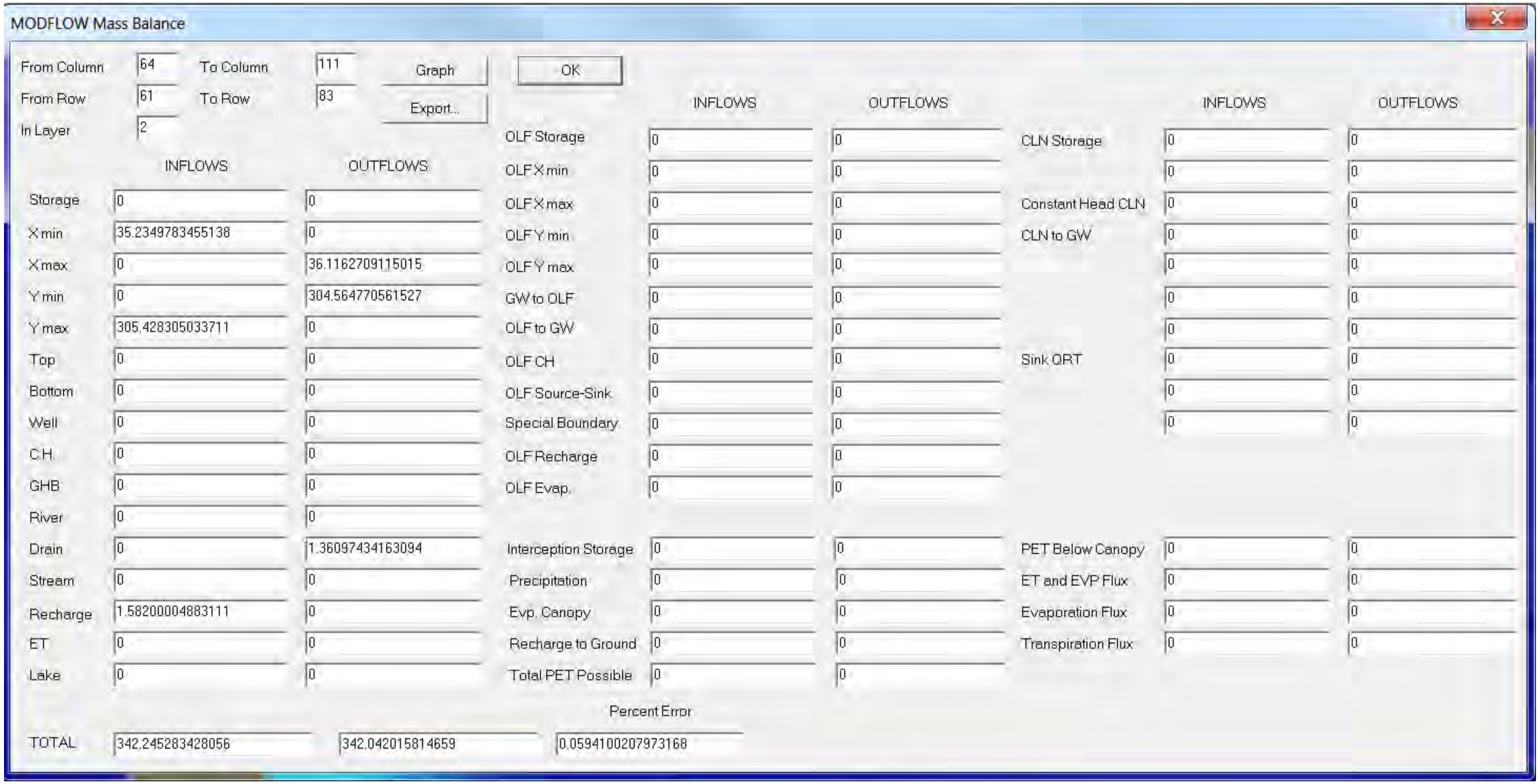
	INFLOWS	OUTFLOWS
Storage	0	0
X min	36.5631733178452	0
X max	0	35.4538430109096
Y min	0	300.936680882471
Y max	307.4530189957	0
Top	0	0
Bottom	0	0
Well	0	0
C.H.	0	0
GHB	0	0
River	0	0
Drain	0	9.20064568473026
Stream	0	0
Recharge	1.58200004883111	0
ET	0	0
Lake	0	0

	INFLOWS	OUTFLOWS
OLF Storage	0	0
OLF X min	0	0
OLF X max	0	0
OLF Y min	0	0
OLF Y max	0	0
GW to OLF	0	0
OLF to GW	0	0
OLF CH	0	0
OLF Source-Sink	0	0
Special Boundary	0	0
OLF Recharge	0	0
OLF Evap.	0	0
Interception Storage	0	0
Precipitation	0	0
Evp. Canopy	0	0
Recharge to Ground	0	0
Total PET Possible	0	0

	INFLOWS	OUTFLOWS
CLN Storage	0	0
	0	0
Constant Head CLN	0	0
CLN to GW	0	0
	0	0
	0	0
	0	0
Sink QRT	0	0
	0	0
	0	0
PET Below Canopy	0	0
ET and EVP Flux	0	0
Evaporation Flux	0	0
Transpiration Flux	0	0

	INFLOWS	OUTFLOWS	Percent Error
TOTAL	345.598192362377	345.591169578111	0.00203208690783059

Figure 11.7.12: Mass balance results for all layers within the underpass area, Operation Phase scenario



4.2 Tension pile model

4.2.1 Groundwater mounding effects at each pile are slight. Maximum change in groundwater heads adjacent to piles (when compared to model runs with no pilings) are summarised in Table 11.7.4 for each model scenario.

Table 11.7.4: Tension pile model results

Geology	Hydraulic conductivity (m/d)	Hydraulic gradient (m)	Maximum change in groundwater head (m)
Cohesive Alluvium / Till	0.05	0.005	0.005
Granular Alluvium	10	0.005	0.004
Glaciolacustrine deposits	0.01	0.005	0.004
Chalk	75	0.005	0.005
Sensitivity Analysis			
Chalk	75	0.01	0.009
Chalk	75	0.04	0.04

4.2.2 Groundwater head contour and drawdown plots for the model runs are presented in Figure 11.7.13 to Figure 11.7.16 inclusive, and sensitivity analysis results in Figure 11.7.17 and Figure 11.7.18.

4.2.3 For all scenarios, groundwater head contour plots show very little impact from the pilings, with groundwater readily flowing around the piles even when the hydraulic gradient was increased.

Figure 11.7.13: Drawdown and head contours for tension pile model – cohesive alluvium/till scenario

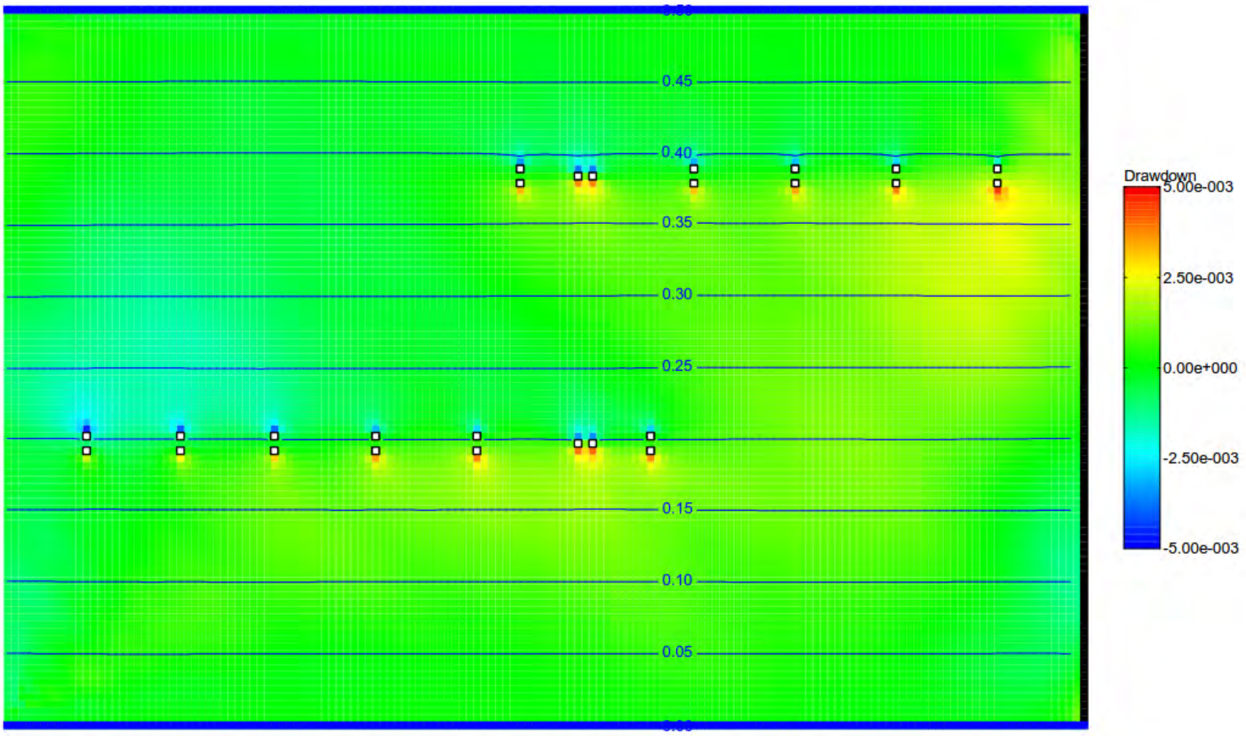


Figure 11.7.14: Drawdown and head contours for tension pile model – granular alluvium scenario

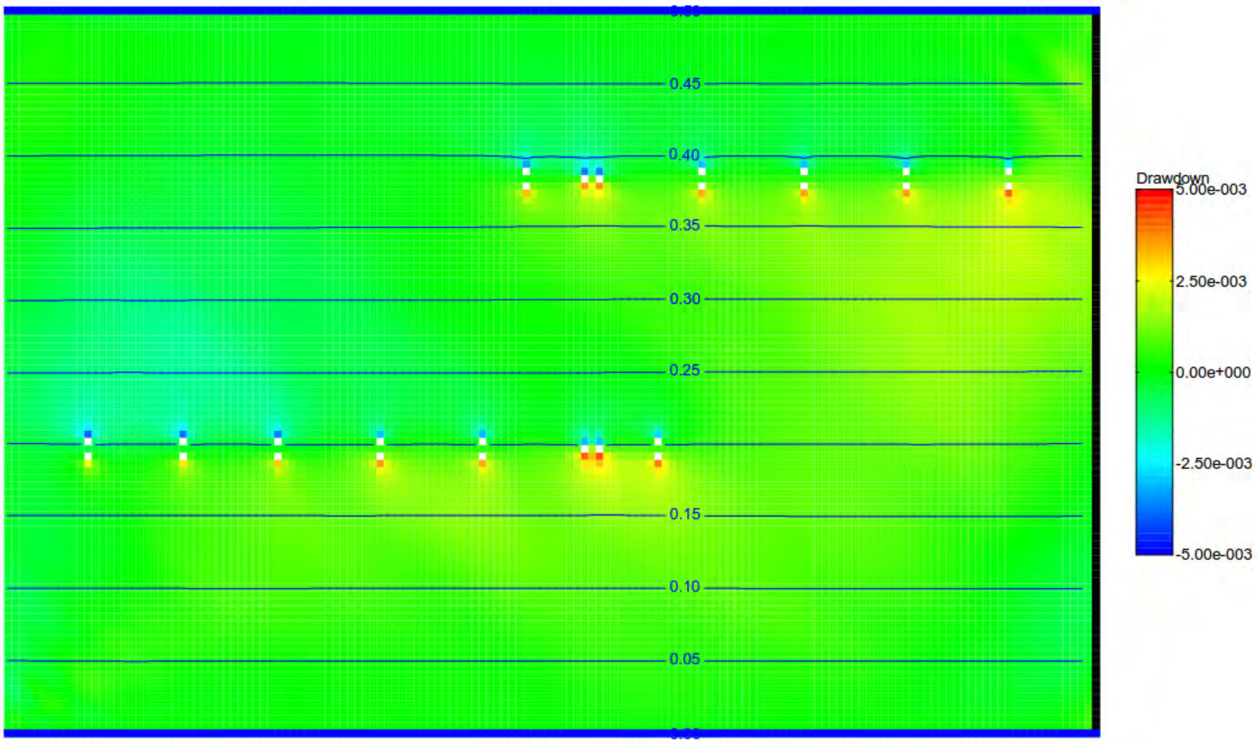


Figure 11.7.15: Drawdown and head contours for tension pile model – glaciolacustrine deposits scenario

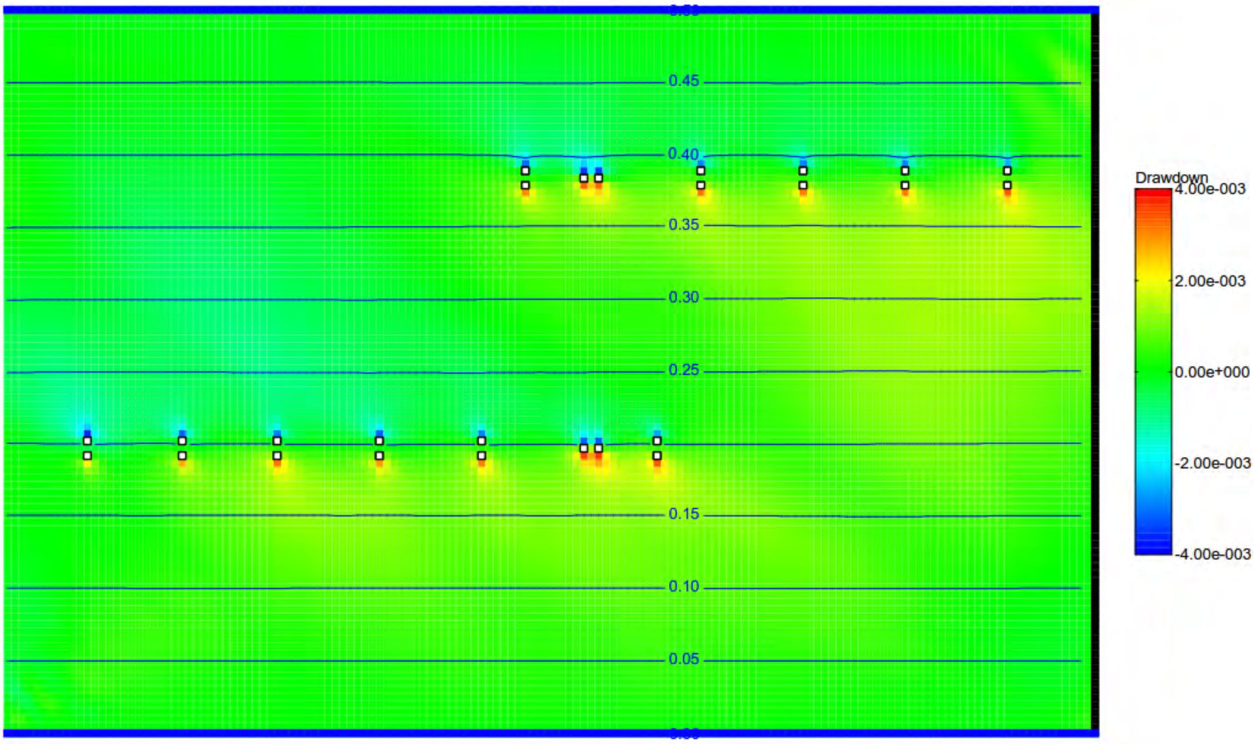


Figure 11.7.16: Drawdown and head contours for tension pile model – Chalk scenario

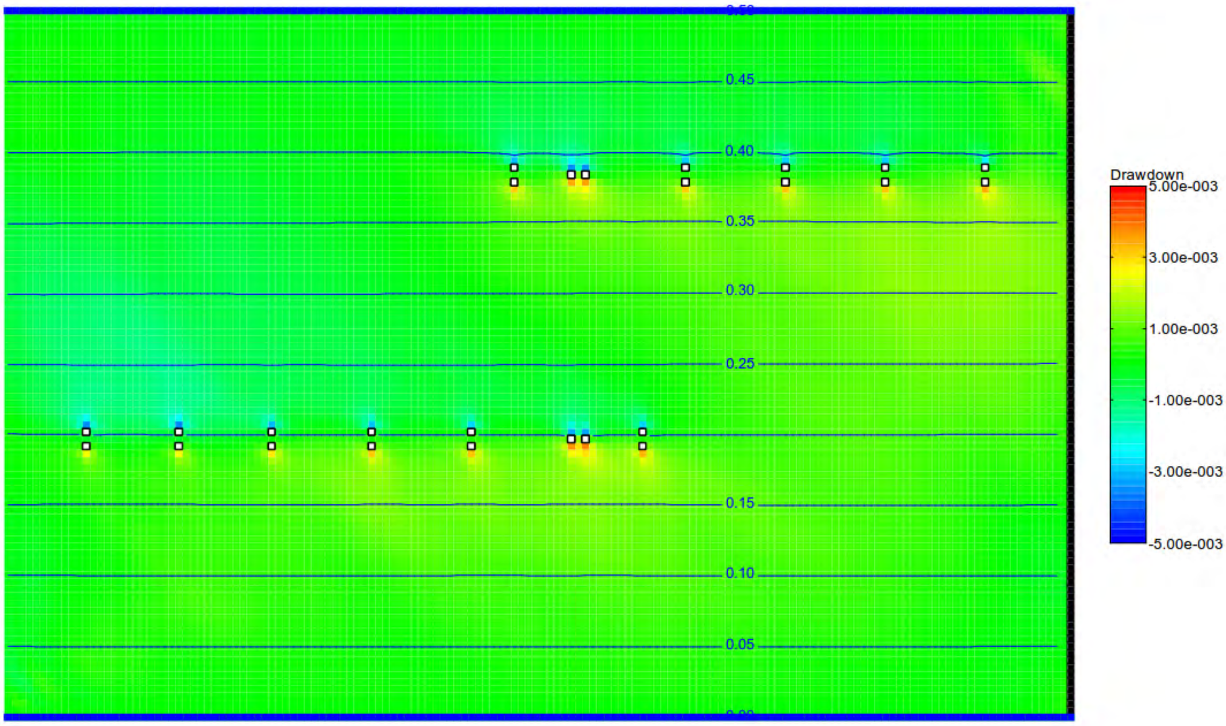


Figure 11.7.17: Sensitivity analysis results: drawdown and head contours for tension pile model – Chalk scenario, hydraulic gradient: 0.01m

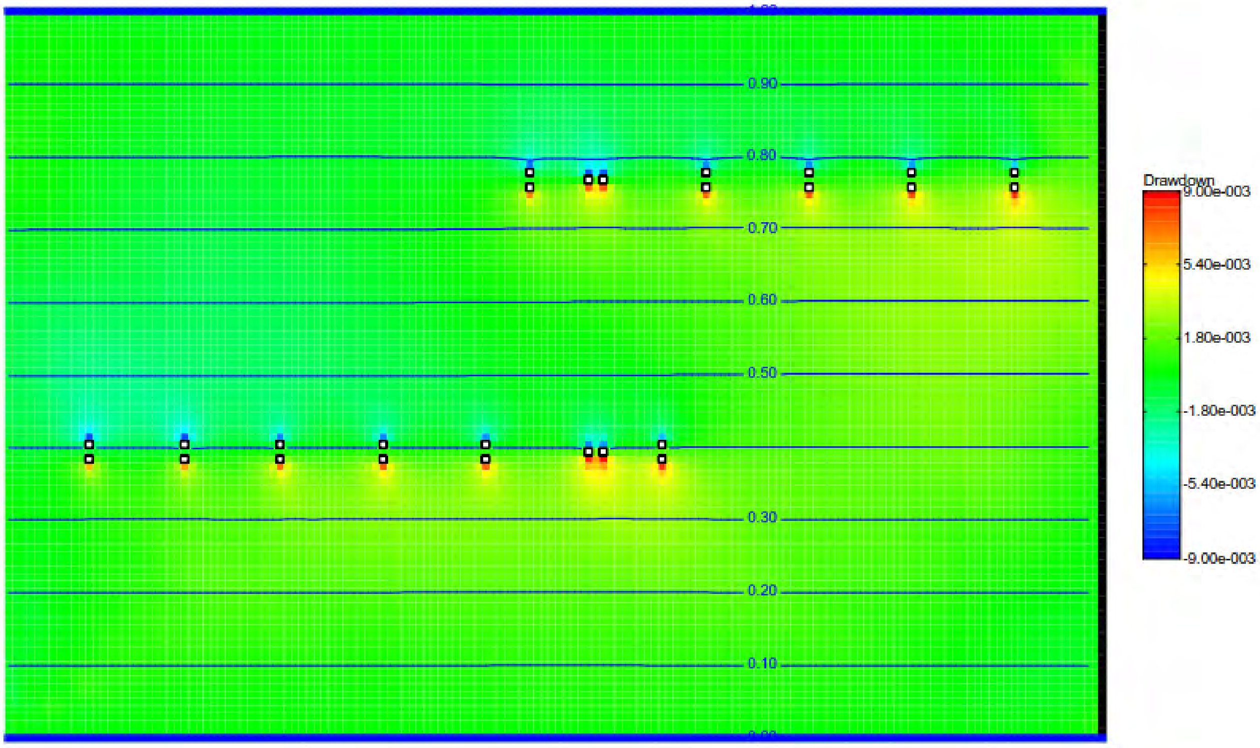
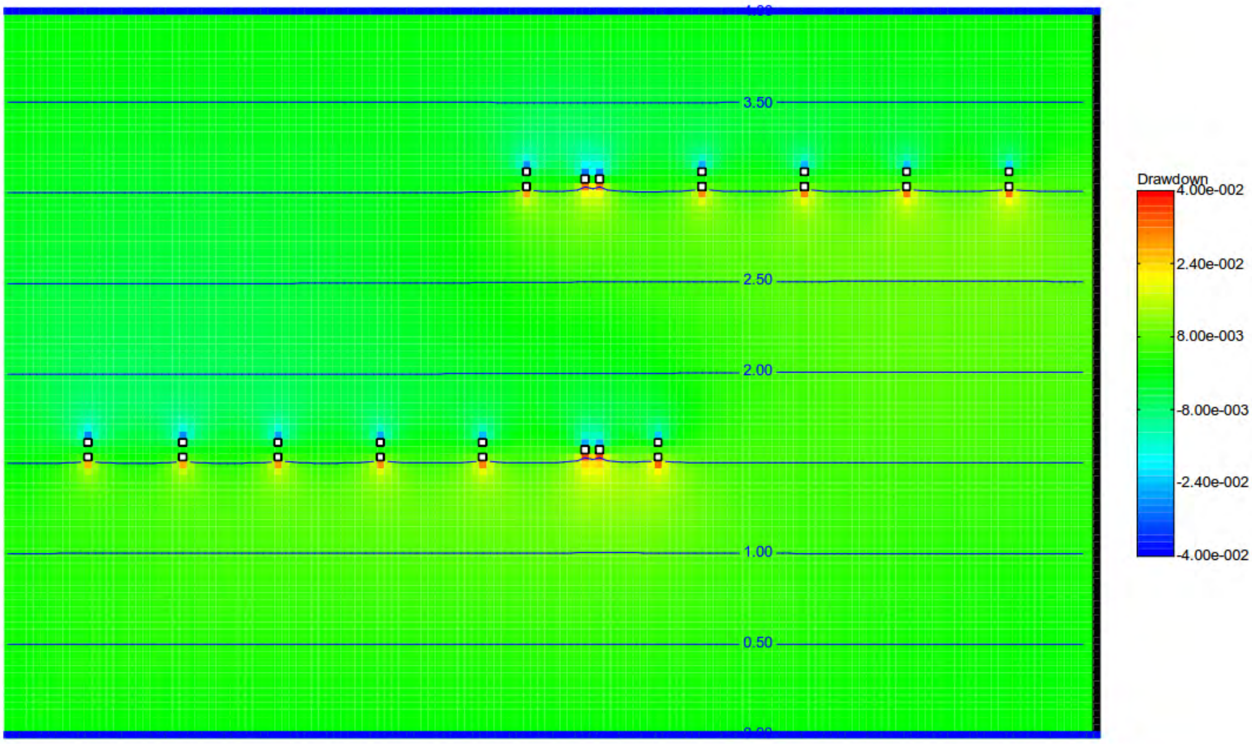


Figure 11.7.18: Sensitivity analysis results: drawdown and head contours for tension pile model – Chalk scenario, hydraulic gradient: 0.04m



5. Discussion of results

5.1 Construction and Operation Phase scenarios

5.1.1 Construction and Operation Phase scenario model results were similar. These are discussed together below and also in the context of the model's limitations. Results from Layer 2 are taken as being representative of the superficial deposits below the made ground. Results have not been investigated for the made ground (Layer 1) as this is too heterogeneous across the modelled area, and furthermore this was modelled as dry across much of the area.

5.1.2 The impermeable diaphragm walls result in a groundwater damming effect to the north of the underpass cutting of up to 0.14m in the superficial deposits during both Construction and Operation Phases. Corresponding drawdown effects to the south of, and downgradient of the underpass are 0.13m or less.

5.1.3 Groundwater mounding in the superficial deposits is greatest to the northeast of Mytongate Junction and at the western end of the underpass cutting. These two distinct locations are seen in both Construction and Operation Phase scenarios and are a function of changes in groundwater head from the modelled baseline and the impact of the impermeable diaphragm wall. Figure 11.7.19 presents the steady state heads for Layer 2 in the 2014 baseline model, which shows an area of higher groundwater heads located between the two areas of groundwater mounding.

5.1.4 Inflow into the open base of the underpass cutting is 9.20 m^3/d.

5.1.5 The model assumes that the jet grout layer has the same hydraulic conductivity as the surrounding ground during the Construction Phase, although this may actually reduce the permeability locally, and therefore reduce inflows into the cutting.

5.1.6 During operation, inflows into the road drainage system is 1.36 m^3/d. This is roughly equivalent to the rainfall recharge falling within the underpass area, however.

5.1.7 Changes in groundwater heads at the locations of the nearest buildings are less than 0.05m in the superficial deposits below the made ground. Although building foundations are only likely to extend into the made ground, it is not possible to provide model results for Layer 1 (made ground) as explained above.

5.1.8 The zone of influence extends into the Humber Docks in layers 2 and 3 (cohesive and granular alluvium respectively). However the dock structures have not been included in the model. Whilst there is the potential for the underpass cutting to impact very slightly on groundwater heads adjacent to Humber Dock, this would be far outweighed by the impact of Humber Dock being regularly topped up with water from the Humber Estuary.

5.1.9 Whilst the results for Layer 3 (granular alluvium) have been provided, these should be used with caution as the 2014 model failed to match target heads in this layer. This was due to the influence of tidal effects varying across this horizon.

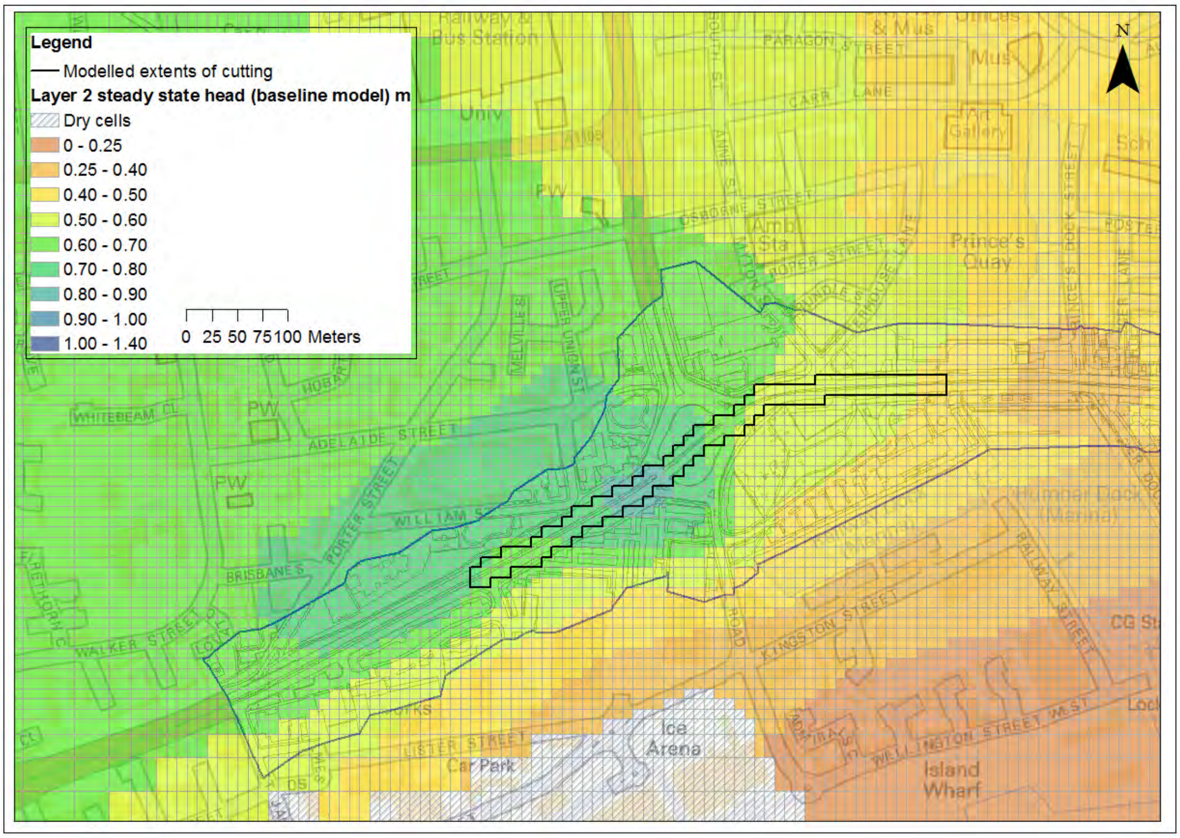
5.1.10 In the Chalk the impermeable diaphragm walls result in a groundwater damming effect to the north of the underpass cutting of up to 0.02m and a drawdown effect (as a result of the blocked groundwater flow) to the south of 0.04m during construction. There is no measurable impact on groundwater heads in the Chalk below the base of the diaphragm walls (i.e. Layer 8).

5.1.11 The steady state model represents average tidal levels. Under high tide conditions the pattern of groundwater damming and drawdown may be reversed.

5.1.12 During operation the impact on Chalk groundwater heads as a result of the diaphragm walls mounding groundwater is +/-0.03m.

5.1.13 The diaphragm walls have been represented in the model as a continuous, impermeable wall extending to 5.5m below the top of the Chalk along the full length of the underpass cutting. In reality, the wall becomes increasingly shallow towards both ends of the underpass and therefore will not present as great a barrier to groundwater flow in the Chalk.

Figure 11.7.19: Modelled steady state heads in Layer 2 for the baseline scenario (2014 Model)



5.2 Tension pile model

5.2.1 The model results indicate that there is very little impact on groundwater heads from the Porter Street Bridge pilings, with groundwater readily flowing around the piles, regardless of the hydraulic gradient modelled and hydraulic conductivity values.

5.2.2 The drift model scenarios suggest some very slight image well effects at the eastern and western extents of the model, resulting in drawdowns of +/-0.0025m extending south and north of the pilings respectively. This is due to the proximity of the piles to the model boundaries.

5.2.3 The results of the 2014 tension pile model are still valid, which shows that the impact of piling at a 4m x 5m grid spacing on heads is slight, with groundwater readily flowing around the piles.

A63 Castle Street Improvements, Hull
Environmental Statement

Volume 3 Appendix 11.8
ROAD DRAINAGE AND THE WATER ENVIRONMENT – DRAINAGE IMPACT ASSESSMENT

TR010016/APP/6.3
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31 July 2018

Highways England

A63 Castle Street Improvement

Drainage Strategy Report

HE514508-ARP-HDG-S0-RP-CD-000506

P02 | July 2018

Job number 237912-00

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		Signature			
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Issue Document Verification with Document ✓

Contents

1 Introduction

1.1 Purpose of this Report

This report sets out the proposed modifications to the road drainage network as part of the A63 Castle Street Improvement scheme. It defines the design criteria and assessment methodology, and summarises the existing and proposed network configurations.

This report focuses on the design following development by the Early Contractor Involvement (ECI) contractor Balfour Beatty and their designer, Arup ("the ECI team") following award of the ECI contract in 2014. The design has been developed from the Illustrative Design prepared by Mott MacDonald Sweco (MMS) and detailed in the following documents: -

- 1168-08-000-RE-001-A1 Existing Drainage Analysis
- 1168-08-005-RE-001-A2 At Grade Proposed Drainage Strategy
- 1168-08-005-RE-002-P2 Underpass Drainage System Strategy
- 1168-08-005-RE-003-P1 Outfall Location Report

1.2 Scheme Background

The A63 Castle Street Improvement scheme is being promoted by Highways England (HE), and involves the improvement by grade separation of the existing A63/A1079 Mytongate intersection in Hull, East Riding of Yorkshire. The scheme will replace the existing signalised roundabout with a grade-separated junction including an underpass for through traffic on the A63 mainline, and slip roads and an overbridge to provide a full-movement junction. The development, and in particular the construction of the mainline underpass, will result in: -

- Changes the pattern of surface water runoff locally;
- Creation of a catchment within the underpass that cannot be drained by gravity to any existing outfall or water body and therefore requires pumping; and
- A requirement for the diversion of public combined sewers.

The construction of the scheme will result in the surface of the A63 mainline being lowered by up to 7m from its existing level, and the road network above raised by up to 1m. The proposed levels of the underpass are lower than the receiving sewers, resulting in the requirement for a pumped system to discharge surface water collected in the underpass catchment.

The project requires removal of the permeable vegetated zones within the existing roundabout and replacement with impermeable paved road surfaces. Surface water will be drained predominantly using combined kerb and drainage (CKD) units installed along the realigned roads.

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2 Methodology

2.1 Introduction

The methodology for design and assessment of the drainage proposals was based upon key design criteria specified by stakeholders, the Design Manual for Roads and Bridges (DMRB), and National Planning Policy Framework (NPPF). Past consultation, summarised in the MMS *Existing Drainage Analysis*[1], *At Grade Proposed Drainage Strategy*[2], and *Underpass Drainage System Strategy*[3] reports, has been supplemented with further consultation with Yorkshire Water (YW) and Hull City Council (HCC). The details of the consultation process and key design criteria are detailed in the sections below.

2.2 Consultation

2.2.1 Yorkshire Water and Hull City Council

2.2.1.1 2013 Consultation

YW were consulted by MMS in 2013 to agree criteria for the proposed drainage works. The design parameters set out by Yorkshire Water as part of these consultations were as follows: -

- The existing highway drainage connections can be maintained on the basis that the proposed flows are similar to or less than the existing flow rates;
- Where possible the proposed highway drainage system should be kept separate from the existing combined system; and
- The proposed surface water collection system should not be lower than the existing system to which it discharges, unless mitigation measures are put in place to prevent the possibility of backflow and flooding onto the carriageway.

The MMS *Existing Drainage Analysis*[1] report identified three outfalls from the highway drainage system to the public sewer. MicroDrainage software was used to calculate the existing peak discharges for each of the networks during a critical duration 1 in 5-year event. In the report, a commitment was made to match or reduce the existing discharge rates, which are detailed in the table overleaf.

[1] 1168-08-000-RE-001 Existing Drainage Analysis (August 2013)
[2] 1168-08-005-RE-001 At Grade Proposed Drainage Strategy (February 2014)
[3] 1168-08-005-RE-002 Underpass Drainage System Strategy (November 2013)

Table 1 – 2013 Existing Peak Discharge Rates

Catchment	Existing Catchment Impermeable Area (ha)	Receiving Sewer	Agreed Existing Peak Discharge (l/s)
101	1.947	2030mm diameter combined sewer	144.4
102	0.958	1066x711mm combined sewer	120.0
104	1.792	1520x1219mm combined sewer	193.8

2.2.1.2 2018 Consultation

Ongoing consultation with YW following appointment of the ECI team culminated in a meeting in May 2018 with YW and HCC to update and agree the methodology and requirements following development of the scheme proposals. The key matter of this consultation was whether the scheme would be considered as brownfield development, and therefore require an overall reduction in flow from the site, or an improvement of the existing system, and require no detriment in line with the 2013 agreement.

The outcome of this consultation was that, subject to production and formal acceptance of this updated strategy, YW and HCC agreed that: -

- There is no requirement for additional attenuation below existing flow rates leaving the site. The requirement for no overall increase in peak flowrate from the scheme area remains;
- The proposal to discharge the pumped underpass drainage network into the YW combined system was reasonable (see Section 4.2.5 for details); and
- The ECI team would assess the amendments required to the design in order to achieve a 30% reduction in peak flow rates leaving the site for comparison.

As part of the production of this revised drainage strategy, and to reflect development of the scheme design following appointment of the ECI team, the assessment of existing baseline peak discharge rates has been updated, as detailed in Section 0.

2.2.2 Environment Agency

Several meetings with the Environment Agency (EA) have been held to discuss the design criteria of the proposed A63 mainline underpass drainage system. The EA were only involved in the underpass drainage aspect of the scheme as all other networks discharge into YW sewers. The EA requirements can be summarised as follows: -

- The underpass should not flood for a 1-in-100-year return period with a 30% allowance for climate change (in line with guidance from the National Planning Policy Framework);

- Traffic diversion routes around the underpass should be drivable (taken to mean no flooding deeper than kerb level) during a 1-in-100-year return period rainfall event with a 30% allowance for climate change;
- Consideration must be given to the overland flows (external to the site) entering the underpass during extreme events;
- Flows may be pumped into the River Humber at an unrestricted rate;
- Alternative power supply sources (generator, uninterruptable power supply etc.) should be considered to manage the risk of power failure; and
- Emergency procedures should be developed to minimise the risk to road users should power failure occur over an extended period of time.

2.3 Design Criteria

In addition to the consultee requirements set out above, the following specific criteria were drawn from the DMRB and NPPF: -

- There must be no surcharge of pipes during any 1-in-1-year return frequency storm event; and
- There must be no flooding arising from road drainage in any 1-in-5-year return frequency storm event including a 30% allowance for increase in rainfall intensities due to climate change.

2.3.1 Hydraulic Modelling Parameters

To allow for the modelling of the flow rates of the various networks, hydraulic parameters were determined using the rainfall mapping tool and guidance in MicroDrainage to ensure that flow rates generated were comparable across the multiple scenarios. Table 2 below details the parameters that were used.

Table 2 – Simulation Parameters for Hydraulic Modelling

Parameter		Value
Region	M5-60 (mm)	18.600
	Ratio R	0.391
Volumetric Run-off Coefficient	Summer	0.750
	Winter	0.840

2.4 Assessment Methodology

The existing road drainage system discharges to the YW combined sewer network, which is present throughout the scheme. The proposed scheme design includes extensive modifications to the alignments of both the A63 mainline and connected side roads. In addition to extensive changes to localised topography and flow patterns, the creation of the mainline underpass will sever many of the existing drainage and sewer networks, resulting in significant changes to road drainage catchments and outfall locations. In order to assess the impact of the proposals on the existing network, the following key approaches have been adopted: -

- Four "outfall" locations have been identified on the YW network, which are the locations at which road drainage leaves the site to join the wider combined sewer network. Whilst the distribution of flows between these locations will change as a result of the scheme, all flows arising from both the existing and proposed systems pass through them, thus permitting an overall comparison.
- Several of the existing and proposed catchments connect to YW main sewers which pass through the scheme area, connecting road drainage sub-catchments and carrying combined flows from beyond the scheme area. As modelling of the wider Hull combined sewer network has been considered beyond the scope of this report, the modelling of these catchments has included the main sewers based upon YW and survey records, but with flows generated only from road drainage catchments within the scheme area.
- Due to the reconfiguration of catchments arising from the alignment changes comprising the scheme, a direct like-for-like comparison of the existing and proposed systems is not straightforward. To facilitate illustration of the impact of the scheme, this report presents three scenarios: -
    - The existing configuration, comprising of the existing network and contributing catchments within the scheme area,
    - The proposed configuration, comprising of proposed new networks and catchments based upon the proposed design topography and impermeable areas, and
    - A hybrid scenario, comprising of the proposed new pipe networks but catchment areas based upon the proposed topography but <u>existing</u> impermeable areas. This scenario permits a like-for-like comparison of the impact of changes to impermeable areas arising from the scheme, isolated from the impacts of changes to the network configurations.

3 Existing Drainage Network Analysis

This section should be read in conjunction with the 'Existing Network Catchment Areas' and 'Existing Network Layout' drawings included in Appendix A.

3.1 Methodology

In order to develop models of the existing road drainage networks, YW sewer records within and immediately surrounding the site boundary were reviewed along with outputs from ground-penetrating radar (GPR) utilities surveys. Corresponding network models were developed within MicroDrainage. The existing topography was assessed by means of a topographic survey and catchments for each network determined and prescribed in MicroDrainage.

The assessment of existing networks resulting in the identification of four existing outfall location at which road drainage leaves the scheme area. This is an increase from the 2013 illustrative design and results from a better understanding of the topography and drainage networks as a result of subsequent survey work. As a result, unless noted otherwise, all figures quoted hereafter within this report are not directly comparable to the 2013 illustrative design detailed in the MMS *Existing Drainage Analysis*[4] report.

It should be noted that not all drainage infrastructure could be confirmed from the existing sewer records or GPR surveys. However, given the otherwise extensive coverage of the records, it is envisaged this will have little impact on the overall discharge rates. Also, as noted in Section 2.4, only flows arising from the contributing catchment area within the site boundary were analysed as the drainage arrangements of external contributing areas will be unaffected by the proposed works and are beyond the scope of this assessment.

3.2 Existing Drainage Description

The A63/A1079 Mytongate junction is presently an at-grade signalised roundabout, with existing ground levels of between 2.8m and 4.1m A.O.D. Due to the site location falling within an urban area of Hull, the site predominantly comprises impermeable surfaces. Surface water is collected by positive drainage systems, predominantly kerb and gully, and conveyed by traditional below ground pipe networks. All surface water is ultimately discharged to the public combined sewer system and ultimately the Humber via Saltend Wastewater Treatment Works. Local topography is predominantly flat, with gentle falls towards the north and west, away from the River Humber, River Hull and associated docks. There is no evidence of an existing connection from the site to a surface water drain/sewer or other watercourse/waterbody.

As noted above, four drainage outfall/discharge points from the site into the combined public sewer network have been identified; these are identified as

[4] 1168-08-000-RE-001 Existing Drainage Analysis (August 2013)

outfalls 101, 102, 103 and 104. The four outfalls and upstream network arrangements are summarised in Table 3 below.

Table 3 – Existing Drainage Networks Summary

Network No.	Network Description	Discharge Location
101	Combined gravity sewers ranging from 300 – 1219mm in diameter. Flows are conveyed from east to west and around the existing ‘hamburger’ style roundabout. The network has one overflow outlet within the site boundary.	Flows outfall into an existing 2030mm combined sewer at the junction of A63 Hessle Road and Porter Street, labelled Outfall 101.
102	Surface water gravity sewers ranging from 150 – 300mm in diameter to the north of the existing Mytongate junction. This network predominately drains highway runoff from Ferensway and the A63. Flows are conveyed from south to north.	Flows outfall into an existing 1066x711mm combined sewer at the junction of Ferensway and Osborne Street, labelled Outfall 102.
103	Combined and surface water gravity sewers ranging from 300 – 1670mm in diameter to the east of the existing Mytongate junction. Flows are conveyed from west to east. The network has one overflow outlet and one overflow inlet within the site boundary.	Flows outfall into an existing 1520x1219mm combined sewer within Queen Street to the south of its junction with The A63, labelled Outfall 103.
104	Combined gravity sewers ranging from 1560 – 1828mm within the southern part of the existing Mytongate junction and Commercial Road. Flows are conveyed from north to south.	Flows outfall into an existing 1250mm combined sewer within Commercial Road, labelled Outfall 104

Sewer records and the Illustrative Design indicate a connection between Existing Networks 101 and 104 within the existing Mytongate Junction, which is identified as an assumed overflow. To permit modelling of the networks this connection has been excluded from the models, as was the case in the illustrative design. It will be necessary to confirm the nature of this connection by means of a comprehensive drainage survey as part of detailed design, which may affect the relative distribution of flows within the existing networks.

3.3 Hydraulic Modelling

The existing networks were modelled using MicroDrainage and the parameters and for the rainfall scenarios set out in Section 2.3. The results of the simulation are shown in Table 4 opposite.

Table 4 – Existing Network Flow Rates

Existing Network	Catchment Area (ha)	Return Period Flow Rates (l/s)	
		1-in-1-year	1-in-5-year +30%
101	1.97	117.2	251.9
102	0.98	66.2	118.6
103	1.78	111.9	246.3
104	0.32	26.8	57.1
Total	5.05	322.1	673.9

3.4 Summary

Four existing catchments have been defined and simulated. The number and configuration of catchments differs from previous assessments in the illustrative design reports due to the availability of more accurate information relating to topography and drainage networks. The flow rates from the existing catchments are generally higher than those modelled in previous work, although this may be due to the allowance for increasing rainfall intensity due to climate change, which has been included in this assessment for consistency when assessing the proposed networks.

[Page Not Used]

4 Proposed Drainage Network Analysis

This section should be read in conjunction with the 'Proposed Network Catchment Areas' and 'Proposed Network Layout' drawings included in Appendix B.

4.1 Methodology

The proposed drainage networks were based upon those prepared for the illustrative design, with development to incorporate the new existing asset information outlined in Section 3.1, topography of the current scheme geometric design, and latest YW diversion proposals.

The networks were modelled using the criteria set out in Section 2.3, and attenuation provided to match the peak 1-in-5-year flows to the existing peak outflows detailed in Section 0. Due to the complexity of the scheme and the variation between the nature of the networks, the approach to attenuation has been to achieve no overall increase across the scheme, rather than on an outfall-by-outfall case.

4.2 Proposed Drainage Description

The proposed road drainage system will comprise five networks, based upon the proposed topography of the scheme and discharging to the four outfalls identified in Section 3.2. Table 5, overleaf, summarises the proposed networks.

Table 5 – Proposed Drainage Networks

Network No.	Network Description	Discharge Location
101	Modification of Existing Network 101, comprising the A63 Mainline west of the underpass, eastbound diverge and westbound merge slip roads, western half of Commercial Road and Mytongate overbridge, and part of eastbound merge slip road from Ferensway to Myton Street.	Outfall 101
102	Modification of Existing Network 102, comprising Ferensway north of Mytongate overbridge.	Outfall 102
103	Modification of Existing Network 103, comprising mainline east of Princes Dock Street, and junctions with Market Place and Queen Street.	Outfall 103
104	Part of Existing Network 104 supplemented with eastbound merge slip road east of Myton Street, and A63 mainline east of the underpass to Princes Dock Street.	Outfall 104
105	New network comprising A63 mainline underpass, westbound diverge slip road, and eastern half of Mytongate overbridge.	Outfall 104

4.2.1 Proposed Network 101

The proposed drainage network discharges into Outfall 101 and utilises the existing main sewer supplemented with four new road drainage branches. The existing drainage system comprises combined gravity sewers ranging from 300 – 1219mm in diameter. Flows are conveyed from east to west and around the existing Mytongate roundabout. The network outfalls into an existing 2030mm combined sewer (Outfall 101) towards the western extent of the site and incorporates an overflow pipe to provide relief in times of extreme rainfall.

The proposed drainage system will use positive drainage systems throughout, anticipated to be combined kerb drains (CKD's). The existing main sewer within this network will require diversion to avoid severance by the A63 mainline underpass. The proposed diversions will be designed and constructed by YW; an indicative design has been included in the Appendix D.

Each of the four new branches incorporates attenuation by way of vortex flow control devices and oversized pipes. The details of the attenuation provision are set out in Table 6, opposite.

Table 6 – Proposed Network 101 Flow Restriction Summary

Flow Restriction in Manhole	Attenuation Pipe No.	Proposed Flow Restriction Rate (l/s)
SMH11	5.004 (525mm)	9.5
	5.005 (525mm)	
SMH04	8.002 (525mm)	10.0
SMH18	9.004 (525mm)	8.5
	9.005 (525mm)	
SMH23	10.003 (525mm)	10.5

4.2.2 Proposed Network 102

The proposed network broadly retains the existing network within Ferensway, with the addition of a new southern section to accommodate the proposed scheme. The proposed drainage system will use positive drainage systems throughout, and it is anticipated that the retained existing systems will be supplemented with combined kerb drains (CKD's).

Due to the network comprising predominantly existing retained pipework, attenuation would be impractical and is therefore not proposed within this network.

4.2.3 Proposed Network 103

This proposed network retained significant amounts of existing drainage pipework, with minor addition of new branches to accommodate reconfiguration of the Market Place / Queen Street junction. The existing drainage network comprises of combined and surface water gravity sewers ranging from 300 – 1670mm in diameter. Flows are conveyed from west to east before out-falling into an existing combined sewer (Outfall 103) towards the eastern extent of the site. The existing network has one overflow outlet and one overflow inlet within the site boundary.

The proposed drainage system will use positive drainage systems throughout, anticipated to be combined kerb drains (CKD's).

The largest of the new branches will be attenuated by means of a vortex flow control device and oversized pipes, as detailed in Table 7 overleaf.

Table 7 – Proposed Network 103 Flow Restriction Summary

Flow Restriction in Manhole	Attenuation Pipe Number	Proposed Flow Restriction Rate (l/s)
SMH04	4.000 (600mm)	5.0
	4.001 (600mm)	
	4.002 (600mm)	

4.2.4 Proposed Network 104

This proposed network retains some existing pipework within the A63 mainline but otherwise comprises of sub-catchments connected into a YW main sewer diversion. The existing main sewer crossing Mytongate junction will require diversion to avoid severance by the A63 mainline underpass. This diversion will pass to the east and south of Trinity Burial Ground before connecting into the existing sewer in Commercial Road at Outfall 104. The proposed diversions will be designed and constructed by YW; an indicative design has been included in Appendix D.

The proposed drainage system will use positive drainage systems throughout, anticipated to be combined kerb drains (CKD's).

The branch of this network within the A63 mainline is proposed to be attenuated as detailed in Table 8 below.

Table 8 – Proposed Network 104 Flow Restriction Summary

Flow Restriction in Manhole	Attenuation Pipe Number	Proposed Flow Restriction Rate (l/s)
EXMH5404	9.001 (750mm)	30.0
	10.000 (750mm)	

4.2.5 Proposed Network 105

This proposed network is entirely new, and drains the A63 mainline underpass and eastbound diverge slip road to Outfall 104. The proposed drainage system will use positive drainage systems throughout, anticipated to be combined kerb drains (CKD's).

Due to the proposed levels of the scheme design within the underpass, this network will be drained by means of a pumping station, which will return all flows generated within the catchment to Outfall 104.

4.2.5.1 Rising Main and Outfall Location

The MMS *Outfall Location Report*[5] identified a number of potential locations for discharge of the pumping station rising main. The locations identified as most favourable were direct discharge to the Humber via a route along Commercial Road. A reserve option of discharge to YW sewers was included as a last resort, should alternatives prove impracticable.

Following further investigation, the proposed solution of outfalling to the Humber is no longer considered feasible, for the following reasons: -

- The route along Commercial Road is highly congested with utilities. The rising main would be of a reasonably large diameter and would require construction of several buried structures for valve chambers and thrust blocks. Construction of the rising main on this route would result in significant disruption to the local area, as well as significant programme and cost risk.
- The ground conditions at the site are very poor. This would exacerbate the construction challenges detailed above.
- The route from public highway to the final discharge into the Humber would require acquisition of private land and subsequent sterilisation for future development. This would likely result in challenge to the Development Consent Order and significant compensation costs.
- The outfall to the Humber would be technically challenging, requiring installation of the rising main through the existing seawall structure, including buried supporting structures.
- Discharge to the Humber would require provision of an oil interceptor. As the route to the Humber would need to be pressurised throughout (as the outfall point would be below the highest spring tide level), the only solution would be to provide the interceptor upstream of the pumps, within the wet well shaft. This would introduce complexity to the structure and create maintenance liabilities with implications for health and safety.

As a result of the issues outlined above, it is now proposed that the rising main outfall to the YW combined sewer at Outfall 104.

4.2.5.2 Pumping Rate

The design of the pumping station and approach to balancing pumped discharge rate with storage volume has sought to balance the following factors: -

- The high flow rates generated by the 1-in-100-year design storm mandated by the EA, which will generate significant volumes of runoff;
- The extremely challenging ground conditions at the site, which will significantly increase the cost and risk of constructing the pumping station civil works and disproportionately increase the cost of storage-based solutions versus pump-rate-based ones;

[5] 1168-08-005-RE-003 Outfall Location Report (March 2014)

- The need to mitigate the impact of flow rates on the YW sewer network; and
- The need to maintain operational flexibility and resilience.

Mindful of the constraints above, it is proposed that the pumping rate be based on the peak flow rate from a 1-in-5-year storm event. The modelled flow from the catchment for such an event is 202.9 l/s, and therefore a design pumping rate of 200.0 l/s has been selected. It is anticipated that this will be provided by three pumps operating in a duty/assist/standby configuration. The proposed approach is considered optimal for the following reasons: -

- The proposed flow rate is accommodated within the overall site peak flow rate of not more than the existing;
- Higher pumping rates would only result in marginal reductions in the volume of the pumping station wet well, whilst increasing the impact on the YW network;
- Lower pumping rates would result in disproportionate cost of construction due to the volume of the civil works; and
- Lower pumping rates would reduce the resilience of the drainage system to storm events beyond the design criteria, and would increase the time taken to drain the underpass in the event of flooding.

4.3 Hydraulic Modelling

The proposed drainage networks were modelled using MicroDrainage using the criteria set out in Section 2.3. The simulation results are summarised in Table 9 below.

Table 9 – Proposed Network Flow Rates

Proposed Network	Catchment Area (ha)	Return Period Flow Rates (l/s)		
		1-in-1-year	1-in-5-year +30%	1-in-100-year +30%
101	1.72	60.8	113.4	-
102	0.61	65.2	118.3	-
103	1.72	93.8	198.9	-
104	0.60	23.4	42.4	-
105	1.32	137.7	200.0	200.0
Total	5.96	380.9	673.0	-

4.4 Summary

Table 10 below provides a comparison between the modelled existing flow rates and those from the proposed catchments.

Table 10 – Comparison of Existing and Proposed Flow Rates

<table>
<tr><th rowspan="2">Network</th><th rowspan="2">Existing Area (ha)</th><th rowspan="2">Proposed Area (ha)</th><th colspan="2">Existing Flow Rates (l/s)</th><th colspan="2">Proposed Flow Rates (l/s)</th></tr>
<tr><th>1-in-1-year</th><th>1-in-5-year +30%</th><th>1-in-1-year</th><th>1-in-5-year +30%</th></tr>
<tr><td>101</td><td>1.97</td><td>1.72</td><td>117.2</td><td>251.9</td><td>60.8</td><td>113.4</td></tr>
<tr><td>102</td><td>0.98</td><td>0.61</td><td>66.2</td><td>118.6</td><td>65.2</td><td>118.3</td></tr>
<tr><td>103</td><td>1.78</td><td>1.72</td><td>111.9</td><td>246.3</td><td>93.8</td><td>198.9</td></tr>
<tr><td>104</td><td rowspan="2">0.32</td><td>0.60</td><td rowspan="2">26.8</td><td rowspan="2">57.1</td><td>23.4</td><td>42.4</td></tr>
<tr><td>105</td><td>1.32</td><td>137.7</td><td>200.0</td></tr>
<tr><td>Total</td><td>5.05</td><td>5.96</td><td>322.1</td><td>673.9</td><td>380.9</td><td>673.0</td></tr>
</table>

As detailed above, the proposed scheme results in an increase in impermeable area of 0.81ha or 18%. Despite this increase, through use of attenuation there is a marginal decrease in the peak discharge rate in a 1-in-5-year storm event.

Due to the complex and varying nature of the drainage networks, the overall attenuation of across the scheme is achieved by reductions for some networks and increases for others. However, the proposals have been assessed by YW using their sewer network model and do not result in unacceptable impacts at any location.

[Page Not Used]

5 Hybrid Drainage Network Analysis

This section should be read in conjunction with the ‘Hybrid Network Catchment Areas’ drawing included in Appendix C.

5.1.1 Methodology

As detailed in Section 2.4, in order to facilitate more direct comparison between the existing and proposed situations, and take account of the necessary reconfigurations of the drainage networks, a hybrid drainage model has been developed. This model uses the proposed drainage network configurations, but with existing extents of impermeable areas attributed to them. As a result, is it possible to identify the impact arising from the increase in impermeable area resulting from the scheme.

It should be noted that the attenuation design in the hybrid scenario is unchanged from the proposed design, and therefore has not been optimised for the flows generated by the hybrid catchments.

5.1.2 Hydraulic Modelling

The proposed drainage networks were modelled using MicroDrainage using the criteria set out in Section 2.3. The simulation results are summarised in Table 11 below:

Table 11 – Hybrid Network Flow Rates

Proposed Network	Catchment Area (ha)	Return Period Flow Rates (l/s)	
		1-in-1-year	1-in-5-year +30%
101	1.21	62.8	135.8
102	0.61	63.5	116.4
103	1.78	91.3	211.3
104	0.45	20.5	46.1
105	1.01	94.6	201.8
Total	5.05	332.7	711.4

5.1.3 Summary

Table 12 below provides a comparison between the modelled existing flow rates and those from the hybrid and proposed catchments.

Table 12 – Comparison of Existing, Hybrid and Proposed Flow Rates

Network	Existing Flow Rates (l/s)		Hybrid Flow Rates (l/s)		Proposed Flow Rates (l/s)	
	1-in-1-year	1-in-5-year +30%	1-in-1-year	1-in-5-year +30%	1-in-1-year	1-in-5-year +30%
101	117.2	251.9	62.8	135.8	60.8	113.4
102	66.2	118.6	63.5	116.4	65.2	118.3
103	111.9	246.3	91.3	211.3	93.8	198.9
104	26.8	57.1	20.5	46.1	23.4	42.4
105			94.6	201.8	137.7	200.0
Total	322.1	673.9	332.7	711.4	380.9	673.0

As can be seen above, the reconfiguration of the networks would result in a transfer of flow between outfalls, primarily from Outfall 101 to Outfall 104. In addition, reconfiguration alone would result in an increase in peak flow rate, as demonstrated by the hybrid total peaks versus the existing.

When comparing the proposed to the hybrid, it can be seen that the proposed results in an increase in flow in the 1-in-1-year scenario, but lower than proportional for the increase in impermeable area. In the 1-in-5-year the proposed shows a reduction in the peak flow versus the hybrid.

6 Conclusion

6.1.1 Proposed Scheme and Methodology

The proposed A63 Castle Street Improvement will upgrade the existing at-grade Mytongate Roundabout to a grade-separated junction. This will require reconfiguration of drainage networks, necessitate diversion of existing sewers, and create new impermeable areas to be drained.

Through consultation with stakeholders, common design criteria have been developed with which to develop and assess the proposed drainage works to be implemented as part of the scheme.

The proposed A63 mainline underpass will be drained by means of a pumping station, which will discharge into the YW sewer network. The discharge rate will be 200l/s, which is considered optimal to balance the cost and risk of construction with the impact on the receiving network.

Models have been created to allow an assessment and comparison between the existing and proposed drainage systems. In addition, a hybrid model has been developed to permit assessment of the impacts of reconfiguration and increased impermeable area separately.

6.1.2 Assessment Results

Table 13 overleaf details the results of the assessment for the existing, hybrid, and proposed scenarios. In summary: -

- The scheme will result in an increase in impermeable area of 18.0%.
- The reconfiguration of networks would result in an increase in peak outflows of 3.3% in the 1-in-1-year scenario and 5.6% in the 1-in-5-year scenario.
- The proposed networks would result in an increase in peak outflows of 18.3% in the 1-in-1-year scenario, but would be attenuated such that the 1-in-5-year peak flows are reduced by 0.1%.

Table 13 – Existing, Hybrid and Proposed Network Assessment

<table>
<tr><th rowspan="2">Network</th><th colspan="3">Catchment Areas</th><th colspan="2">Existing Flow Rates</th><th colspan="4">Hybrid Flow Rates</th><th colspan="4">Proposed Flow Rates</th></tr>
<tr><th>Existing (ha)</th><th>Proposed (ha)</th><th>Variance</th><th>1-in-1-year (l/s)</th><th>1-in-5-year +30% (l/s)</th><th>1-in-1-year (l/s)</th><th>Variance</th><th>1-in-5-year +30% (l/s)</th><th>Variance</th><th>1-in-1-year (l/s)</th><th>Variance</th><th>1-in-5-year +30% (l/s)</th><th>Variance</th></tr>
<tr><td>101</td><td>1.97</td><td>1.72</td><td>-12.7%</td><td>117.2</td><td>251.9</td><td>62.8</td><td>-46.4%</td><td>135.8</td><td>-46.1%</td><td>60.8</td><td>-48.1%</td><td>113.4</td><td>-55.0%</td></tr>
<tr><td>102</td><td>0.98</td><td>0.61</td><td>-37.8%</td><td>66.2</td><td>118.6</td><td>63.5</td><td>-4.1%</td><td>116.4</td><td>-1.9%</td><td>65.2</td><td>-1.5%</td><td>118.3</td><td>-0.3%</td></tr>
<tr><td>103</td><td>1.78</td><td>1.72</td><td>-3.4%</td><td>111.9</td><td>246.3</td><td>91.3</td><td>-18.4%</td><td>211.3</td><td>-14.2%</td><td>93.8</td><td>-16.2%</td><td>198.9</td><td>-19.2%</td></tr>
<tr><td>104</td><td rowspan="2">0.32</td><td>0.60</td><td rowspan="2">+500.0%</td><td rowspan="2">26.8</td><td rowspan="2">57.1</td><td>20.5</td><td rowspan="2">+329.5%</td><td>46.1</td><td rowspan="2">+334.2%</td><td>23.4</td><td rowspan="2">+501.1%</td><td>42.4</td><td rowspan="2">+324.5%</td></tr>
<tr><td>105</td><td>1.32</td><td>94.6</td><td>201.8</td><td>137.7</td><td>200.0</td></tr>
<tr><td>Total</td><td>5.05</td><td>5.96</td><td>+18.0%</td><td>322.1</td><td>673.9</td><td>332.7</td><td>+3.3%</td><td>711.4</td><td>+5.6%</td><td>380.9</td><td>+18.3%</td><td>673.0</td><td>-0.1%</td></tr>
</table>

Appendix A

Existing Drainage Networks

Millimetres
0 10 100

DO NOT SCALE

Key:

- Existing Network 101 (Impermeable Area = 1.97ha)
- Existing Network 102 (Impermeable Area = 0.98ha)
- Existing Network 103 (Impermeable Area = 1.78ha)
- Existing Network 104 (Impereable Area = 0.32ha)

GENERAL NOTES:

1. ALL DIMENSIONS ARE IN METRES UNLESS NOTED OTHERWISE.
2. ONLY WRITTEN DIMENSIONS SHALL BE USED, DO NOT SCALE.

SAFETY, HEALTH AND ENVIRONMENTAL INFORMATION

In addition to the hazards/risks normally associated with the types of work detailed on this drawing, note the following significant residual risks (Reference shall also be made to the design hazard log).

Construction
None

Maintenance / Cleaning
None

Use
None

Decommissioning / Demolition
None

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Suitability
S2 SUITABLE FOR INFORMATION

ARUP Balfour Beatty

Client
highways england

Project Title
A63 CASTLE STREET

Drawing Title
DRAINAGE STRATEGY REPORT
EXISTING NETWORK CATCHMENT AREAS

Scale	By	Checked	Approved	Authorised
N/A	EH	TH	AD	---
Original Size	Date	Date	Date	Date
A1	06/07/18	06/07/18	06/07/18	---

Drawing Number
HE PIN | Originator | Volume
HE514508 - ARP - HDG
S0_ML - DR - CD - 000516
Location | Type | Role | Number

Project Ref. No.

Revision
P01

DO NOT SCALE

Millimetres 0 10 100

Key:

Proposed carrier pipe	Proposed chamber
Existing pipe to be retained	Existing chamber to be retained
Proposed Yorkshire Water diversion (subject to confirmation from YW)	Proposed Yorkshire Water diversion chamber (subject to confirmation from YW)
	Proposed Flow control chamber
	Outfall

GENERAL NOTES:

1. ALL DIMENSIONS ARE IN METERS UNLESS NOTED OTHERWISE
2. ONLY WRITTEN DIMENSIONS SHALL BE USED, DO NOT SCALE

SAFETY, HEALTH AND ENVIRONMENTAL INFORMATION

In addition to the hazards/risks normally associated with the types of work detailed on this drawing, note the following significant residual risks (Reference shall also be made to the design hazard log).

Construction
None

Maintenance / Cleaning
None

Use
None

Decommissioning / Demolition
None

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Suitability
S2 SUITABLE FOR INFORMATION

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Client
highways england

Project Title
A63 CASTLE STREET

Drawing Title
DRAINAGE STRATEGY REPORT
EXISTING NETWORK LAYOUT
SHEET 1 OF 5

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Original Size	**Date**	**Date**	**Date**	**Date**
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Location | Type | Role | Number

Project Ref. No.
237912-00

Revision
P01

DO NOT SCALE

Key:

- Proposed carrier pipe
- Existing pipe to be retained
- Proposed Yorkshire Water diversion (subject to confirmation from YW)
- Proposed chamber
- Existing chamber to be retained
- Proposed Yorkshire Water diversion chamber (subject to confirmation from YW)
- Proposed Flow control chamber
- Outfall

GENERAL NOTES:

1. ALL DIMENSIONS ARE IN METERS UNLESS NOTED OTHERWI
2. ONLY WRITTEN DIMENSIONS SHALL BE USED, DO NOT SCAL

SAFETY, HEALTH AND ENVIRONMENTAL INFORMATION

In addition to the hazards/risks normally associated with the types of work detailed on this drawing, note the following significant residual risks (Reference shall also be made to the design hazard log).

Construction
None

Maintenance / Cleaning
None

Use
None

Decommissioning / Demolition
None

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P01	06/07/18	FIRST ISSUE	EH	TH	AD	---

Suitability
S2 SUITABLE FOR INFORMATION

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Client
highways england

Project Title
A63 CASTLE STREET

Drawing Title
DRAINAGE STRATEGY REPORT
EXISTING NETWORK LAYOUT
SHEET 2 OF 5

Scale	By	Checked	Approved	Authorised
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Drawing Number
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S0_ML - DR - CD - 000512
Location | Type | Role | Number

Project Ref. No.
237912-00

Revision
P01

Millimetres

0 10 100

DO NOT SCALE

Key:

- Proposed carrier pipe
- Existing pipe to be retained
- Proposed Yorkshire Water diversion (subject to confirmation from YW)
- Proposed chamber
- Existing chamber to be retained
- Proposed Yorkshire Water diversion chamber (subject to confirmation from YW)
- Proposed Flow control chamber
- Outfall

GENERAL NOTES:

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SAFETY, HEALTH AND ENVIRONMENTAL INFORMATION

In addition to the hazards/risks normally associated with the types of work detailed on this drawing, note the following significant residual risks (Reference shall also be made to the design hazard log).

Construction
None

Maintenance / Cleaning
None

Use
None

Decommissioning / Demolition
None

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P01	06/07/18	FIRST ISSUE	EH	TH	AD	---
Rev.	Date	Description	By	Chk'd	App'd	Auth'd

Suitability
S2 SUITABLE FOR INFORMATION

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www.arup.com

Client
highways england

Project Title
A63 CASTLE STREET

Drawing Title
DRAINAGE STRATEGY REPORT
EXISTING NETWORK LAYOUT
SHEET 3 OF 5

Scale	By	Checked	Approved	Authorised
N/A	EH	TH	AD	---
Original Size	**Date**	**Date**	**Date**	**Date**
A1	05/07/18	06/07/18	06/07/18	---

Drawing Number
HE PIN | Originator | Volume
HE514508 - ARP - HDG
S0_ML - DR - CD - 000513
Location | Type | Role | Number

Project Ref. No.
237912-00

Revision
P01

DO NOT SCALE

Millimetres 0 10 100

Key:

- Proposed carrier pipe
- Existing pipe to be retained
- Proposed Yorkshire Water diversion (subject to confirmation from YW)
- Proposed chamber
- Existing chamber to be retained
- Proposed Yorkshire Water diversion chamber (subject to confirmation from YW)
- Proposed Flow control chamber
- Outfall

GENERAL NOTES:

1. ALL DIMENSIONS ARE IN METERS UNLESS NOTED OTHERWISE
2. ONLY WRITTEN DIMENSIONS SHALL BE USED, DO NOT SCALE

SAFETY, HEALTH AND ENVIRONMENTAL INFORMATION

In addition to the hazards/risks normally associated with the types of work detailed on this drawing, note the following significant residual risks (Reference shall also be made to the design hazard log).

Construction
None

Maintenance / Cleaning
None

Use
None

Decommissioning / Demolition
None

Rev.	Date	Description	By	Chk'd	App'd	Auth'd
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P01	06/07/18	FIRST ISSUE	EH	TH	AD	---

Suitability
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Client
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Project Title
A63 CASTLE STREET

Drawing Title
DRAINAGE STRATEGY REPORT
EXISTING NETWORK LAYOUT
SHEET 4 OF 5

Scale	By	Checked	Approved	Authorised
N/A	EH	TH	AD	---
Original Size	**Date**	**Date**	**Date**	**Date**
A1	05/07/18	06/07/18	06/07/18	---

Drawing Number
HE PIN | Originator | Volume
HE514508 - ARP - HDG
S0_ML - DR - CD - 000514
Location | Type | Role | Number

Project Ref. No.
237912-00

Revision
P01

DO NOT SCALE

Key:

- Proposed carrier pipe
- Existing pipe to be retained
- Proposed Yorkshire Water diversion (subject to confirmation from YW)
- Proposed chamber
- Existing chamber to be retained
- Proposed Yorkshire Water diversion chamber (subject to confirmation from YW)
- Proposed Flow control chamber
- Outfall

GENERAL NOTES:

1. ALL DIMENSIONS ARE IN METERS UNLESS NOTED OTHERWISE
2. ONLY WRITTEN DIMENSIONS SHALL BE USED, DO NOT SCALE

SAFETY, HEALTH AND ENVIRONMENTAL INFORMATION

In addition to the hazards/risks normally associated with the types of work detailed on this drawing, note the following significant residual risks (Reference shall also be made to the design hazard log).

Construction
None

Maintenance / Cleaning
None

Use
None

Decommissioning / Demolition
None

Rev.	Date	Description	By	Chk'd	App'd	Auth'd
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P01	06/07/18	FIRST ISSUE	EH	TH	AD	---

Suitability
S2 SUITABLE FOR INFORMATION

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Client
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Project Title
A63 CASTLE STREET

Drawing Title
DRAINAGE STRATEGY REPORT
EXISTING NETWORK LAYOUT
SHEET 5 OF 5

Scale	By	Checked	Approved	Authorised
N/A	EH	TH	AD	---
Original Size	**Date**	**Date**	**Date**	**Date**
A1	05/07/18	06/07/18	06/07/18	---

Drawing Number
HE PIN | Originator | Volume
HE514508 - ARP - HDG
S0_ML - DR - CD - 000515
Location | Type | Role | Number

Project Ref. No.
237912-00

Revision
P01

Appendix B

Proposed Drainage Networks

DO NOT SCALE

Millimetres 0 10 100

Key:

- Proposed Network 101 (Impermeable Area =1.72ha)
- Proposed Network 102 (Impermeable Area =0.61ha)
- Proposed Network 103 (Impermeable Area =1.72ha)
- Proposed Network 104 (Impermeable Area =0.60ha)
- Proposed Network 105 (Impermeable Area =1.32ha)

GENERAL NOTES:

1. ALL DIMENSIONS ARE IN METRES UNLESS NOTED OTHERWISE.
2. ONLY WRITTEN DIMENSIONS SHALL BE USED, DO NOT SCALE.

SAFETY, HEALTH AND ENVIRONMENTAL INFORMATION

In addition to the hazards/risks normally associated with the types of work detailed on this drawing, note the following significant residual risks (Reference shall also be made to the design hazard log).

Construction
None

Maintenance / Cleaning
None

Use
None

Decommissioning / Demolition
None

Rev.	Date	Description	By	Chk'd	App'd	Auth'd
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P01	06/07/18	FIRST ISSUE	EH	TH	AD	---

Suitability
S2 SUITABLE FOR INFORMATION

ARUP Balfour Beatty

Client
highways england

Project Title
A63 CASTLE STREET

Drawing Title
DRAINAGE STRATEGY REPORT
PROPOSED NETWORK CATCHMENT AREAS

Scale	By	Checked	Approved	Authorised
N/A	EH	TH	AD	---
Original Size	**Date**	**Date**	**Date**	**Date**
A1	05/07/18	06/07/18	06/07/18	---

Drawing Number
HE PIN | Originator | Volume
HE514508 - ARP - HDG
S0_ML - DR - CD - 000522
Location | Type | Role | Number

Project Ref. No.
237912-00

Revision
P01

DO NOT SCALE

Millimetres

Key:

Proposed carrier pipe	Proposed chamber
Existing pipe to be retained	Existing chamber to be retained
Proposed Yorkshire Water diversion (subject to confirmation from YW)	Proposed Yorkshire Water diversion chamber (subject to confirmation from YW)
	Proposed Flow control chamber
	Outfall

GENERAL NOTES:

1. ALL DIMENSIONS ARE IN METERS UNLESS NOTED OTHERWISE
2. ONLY WRITTEN DIMENSIONS SHALL BE USED, DO NOT SCALE

SAFETY, HEALTH AND ENVIRONMENTAL INFORMATION

In addition to the hazards/risks normally associated with the types of work detailed on this drawing, note the following significant residual risks (Reference shall also be made to the design hazard log).

Construction
None

Maintenance / Cleaning
None

Use
None

Decommissioning / Demolition
None

Rev.	Date	Description	By	Chk'd	App'd	Auth'd
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P01	06/07/18	FIRST ISSUE	EH	TH	AD	---

Suitability
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Project Title: A63 CASTLE STREET

Drawing Title: DRAINAGE STRATEGY REPORT PROPOSED NETWORK LAYOUT SHEET 1 OF 5

Scale	By	Checked	Approved	Authorised
N/A	EH	TH	AD	---
Original Size	**Date**	**Date**	**Date**	**Date**
A1	05/07/18	06/07/18	06/07/18	---

Drawing Number: HE PIN | Originator | Volume — HE514508 - ARP - HDG - S0_ML - DR - CD - 000517 (Location | Type | Role | Number)

Project Ref. No.: 237912-00

Revision: P01

Key:

- Proposed carrier pipe
- Existing pipe to be retained
- Proposed Yorkshire Water diversion (subject to confirmation from YW)
- Proposed chamber
- Existing chamber to be retained
- Proposed Yorkshire Water diversion chamber (subject to confirmation from YW)
- Proposed Flow control chamber
- Outfall

GENERAL NOTES:

1. ALL DIMENSIONS ARE IN METERS UNLESS NOTED OTHERWI
2. ONLY WRITTEN DIMENSIONS SHALL BE USED, DO NOT SCAL

SAFETY, HEALTH AND ENVIRONMENTAL INFORMATION

In addition to the hazards/risks normally associated with the types of work detailed on this drawing, note the following significant residual risks (Reference shall also be made to the design hazard log).

Construction
None

Maintenance / Cleaning
None

Use
None

Decommissioning / Demolition
None

Rev.	Date	Description	By	Chk'd	App'd	Auth'd
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P01	06/07/18	FIRST ISSUE	EH	TH	AD	---

Suitability
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Client
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Project Title
A63 CASTLE STREET

Drawing Title
DRAINAGE STRATEGY REPORT
PROPOSED NETWORK LAYOUT
SHEET 2 OF 5

Scale	By	Checked	Approved	Authorised
N/A	EH	TH	AD	---
Original Size	Date	Date	Date	Date
A1	05/07/18	06/07/18	06/07/18	---

Drawing Number
HE PIN | Originator | Volume
HE514508 - ARP - HDG
S0_ML - DR - CD - 000518
Location | Type | Role | Number

Project Ref. No.
237912-00

Revision
P01

DO NOT SCALE

Millimetres 0 10 100

For Continuation See HE514508-ARP-HDG-S0_ML-DR-CD-000518

Key:

- Proposed carrier pipe
- Existing pipe to be retained
- Proposed Yorkshire Water diversion (subject to confirmation from YW)
- Proposed chamber
- Existing chamber to be retained
- Proposed Yorkshire Water diversion chamber (subject to confirmation from YW)
- Proposed Flow control chamber
- Outfall

GENERAL NOTES:

1. ALL DIMENSIONS ARE IN METERS UNLESS NOTED OTHERWISE
2. ONLY WRITTEN DIMENSIONS SHALL BE USED, DO NOT SCALE

SAFETY, HEALTH AND ENVIRONMENTAL INFORMATION

In addition to the hazards/risks normally associated with the types of work detailed on this drawing, note the following significant residual risks (Reference shall also be made to the design hazard log).

Construction
None

Maintenance / Cleaning
None

Use
None

Decommissioning / Demolition
None

Rev.	Date	Description	By	Chk'd	App'd	Auth'd
P01	06/07/18	FIRST ISSUE	EH	TH	AD	---

Suitability: S2 SUITABLE FOR INFORMATION

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Client: highways england

Project Title: A63 CASTLE STREET

Drawing Title: DRAINAGE STRATEGY REPORT PROPOSED NETWORK LAYOUT SHEET 3 OF 5

Scale	By	Checked	Approved	Authorised
N/A	EH	TH	AD	---
Original Size	Date	Date	Date	Date
A1	05/07/18	06/07/18	06/07/18	---

Drawing Number: HE514508 - ARP - HDG - S0_ML - DR - CD - 000519 (HE PIN | Originator | Volume | Location | Type | Role | Number)

Project Ref. No.: 237912-00

Revision: P01

DO NOT SCALE

Millimetres 0 10 100

Key:

Proposed carrier pipe	Proposed chamber
Existing pipe to be retained	Existing chamber to be retained
Proposed Yorkshire Water diversion (subject to confirmation from YW)	Proposed Yorkshire Water diversion chamber (subject to confirmation from YW)
	Proposed Flow control chamber
	Outfall

GENERAL NOTES:

1. ALL DIMENSIONS ARE IN METERS UNLESS NOTED OTHERWISE
2. ONLY WRITTEN DIMENSIONS SHALL BE USED, DO NOT SCALE

SAFETY, HEALTH AND ENVIRONMENTAL INFORMATION

In addition to the hazards/risks normally associated with the types of work detailed on this drawing, note the following significant residual risks (Reference shall also be made to the design hazard log).

Construction
None

Maintenance / Cleaning
None

Use
None

Decommissioning / Demolition
None

Rev.	Date	Description	By	Chk'd	App'd	Auth'd
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P01	06/07/18	FIRST ISSUE	EH	TH	AD	---

Suitability
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Client
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Project Title
A63 CASTLE STREET

Drawing Title
DRAINAGE STRATEGY REPORT
PROPOSED NETWORK LAYOUT
SHEET 4 OF 5

Scale	By	Checked	Approved	Authorised
N/A	EH	TH	AD	---
Original Size	**Date**	**Date**	**Date**	**Date**
A1	05/07/18	06/07/18	06/07/18	---

Drawing Number
HE PIN | Originator | Volume
HE514508 - ARP - HDG
S0_ML - DR - CD - 000520
Location | Type | Role | Number

Project Ref. No.
237912-00

Revision
P01

DO NOT SCALE

Millimetres

Key:

Proposed carrier pipe		Proposed chamber	
Existing pipe to be retained		Existing chamber to be retained	
Proposed Yorkshire Water diversion (subject to confirmation from YW)		Proposed Yorkshire Water diversion chamber (subject to confirmation from YW)	
		Proposed Flow control chamber	
		Outfall	

GENERAL NOTES:

1. ALL DIMENSIONS ARE IN METERS UNLESS NOTED OTHERWISE
2. ONLY WRITTEN DIMENSIONS SHALL BE USED, DO NOT SCALE

SAFETY, HEALTH AND ENVIRONMENTAL INFORMATION

In addition to the hazards/risks normally associated with the types of work detailed on this drawing, note the following significant residual risks (Reference shall also be made to the design hazard log).

Construction
None

Maintenance / Cleaning
None

Use
None

Decommissioning / Demolition
None

Rev.	Date	Description	By	Chk'd	App'd	Auth'd
P01	06/07/18	FIRST ISSUE	EH	TH	AD	—

Suitability
S2 SUITABLE FOR INFORMATION

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Client
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Project Title
A63 CASTLE STREET

Drawing Title
DRAINAGE STRATEGY REPORT
PROPOSED NETWORK LAYOUT
SHEET 5 OF 5

Scale	By	Checked	Approved	Authorised
N/A	EH	TH	AD	—

Original Size	Date	Date	Date	Date
A1	05/07/18	06/07/18	06/07/18	—

Drawing Number
HE PIN | Originator | Volume
HE514508 - ARP - HDG
S0_ML - DR - CD - 000521
Location | Type | Role | Number

Project Ref. No.
237912-00

Revision
P01

Appendix C

Hybrid Drainage Networks

DO NOT SCALE

Millimetres 0 10 100

N

Key:

- Hybrid Network 101 (Impermeable Area =1.21ha)
- Hybrid Network 102 (Impermeable Area =0.61ha)
- Hybrid Network 103 (Impermeable Area =1.78ha)
- Hybrid Network 104 (Impermeable Area =0.45ha)
- Hybrid Network 105 (Impermeable Area =1.01ha)

GENERAL NOTES:

1. ALL DIMENSIONS ARE IN METRES UNLESS NOTED OTHERWISE.
2. ONLY WRITTEN DIMENSIONS SHALL BE USED, DO NOT SCALE.

SAFETY, HEALTH AND ENVIRONMENTAL INFORMATION

In addition to the hazards/risks normally associated with the types of work detailed on this drawing, note the following significant residual risks (Reference shall also be made to the design hazard log).

Construction
None

Maintenance / Cleaning
None

Use
None

Decommissioning / Demolition
None

Rev.	Date	Description	By	Chk'd	App'd	Auth'd
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P01	06/07/18	FIRST ISSUE	EH	TH	AD	---

Suitability
S2 SUITABLE FOR INFORMATION

ARUP Balfour Beatty

Client
highways england

Project Title
A63 CASTLE STREET

Drawing Title
DRAINAGE STRATEGY REPORT
HYBRID NETWORK CATCHMENT AREAS

Scale	By	Checked	Approved	Authorised
N/A	EH	TH	AD	---
Original Size	Date	Date	Date	Date
A1	05/07/18	06/07/18	06/07/18	---

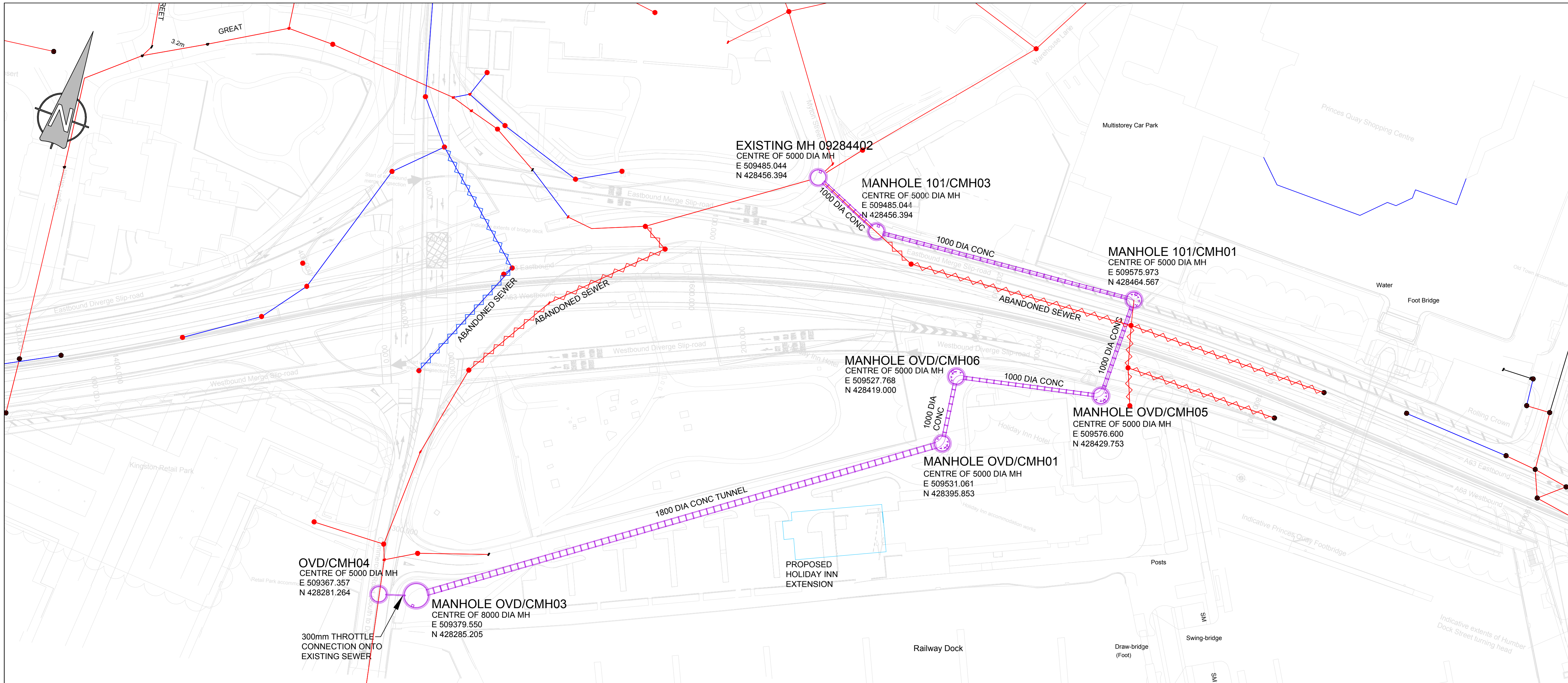
Drawing Number
HE PIN | Originator | Volume
HE514508 - ARP - HDG
S0_ML - DR - CD - 000523
Location | Type | Role | Number

Project Ref. No.

Revision
P01

Appendix D

Yorkshire Water Sewer Diversions



THIS DRAWING IS ISSUED FOR THE SOLE AND EXCLUSIVE USE OF THE NAMED RECIPIENT. DISTRIBUTION TO ANY THIRD PARTY IS ON THE STRICT UNDERSTANDING THAT NO LIABILITY IS ACCEPTED BY BARHALE FOR ANY DISCREPANCIES, ERRORS OR OMISSIONS THAT MAY BE PRESENT, AND NO GUARANTEE IS OFFERED AS TO THE ACCURACY OF THE INFORMATION SHOWN

NOTES:

GENERAL :

1. ALL WORK TO BE COMPLETED IN ACCORDANCE WITH YORKSHIRE WATER'S ENGINEERING SPECIFICATION (VERSION 15) AND ASSET STANDARDS INCLUDING SEWERS FOR ADOPTION 7th EDITION.
2. ALL DIMENSIONS ARE IN MILLIMETRES (mm) & ALL LEVELS ARE IN METRES (m) AOD, UNLESS STATED OTHERWISE.
3. PRECAST CONCRETE PRODUCTS ARE TO COMPLY WITH THE REVELANT PROVISIONS OF BS5911:2002, BS EN 1916:2002 AND BS EN 1917:2002 AND BE KITEMARKED.
4. MANHOLE COVERS AND FRAMES ARE TO COMPLY WITH THE RELEVANT PROVISIONS OF BS EN 124, ARE NON-ROCKING DESIGN WITHOUT CUSHION INSERTS AND ARE KITEMARKED. LOAD CLASS D400 UNLESS STATED OTHERWISE.
5. NO WORK TO BE CARRIED OUT ON THE EXISTING PUBLIC SEWER WITHOUT THE ISSUING OF AUTHORISATION FOR SEWER WORKS FROM YORKSHIRE WATER.
6. HOLIDAY INN DRAINAGE TO BE CONFIRMED AT A LATER STAGE AND RE-ROUTED INTO CMH05.

SAFETY, HEALTH AND ENVIRONMENTAL INFORMATION

In addition to the hazards/risks normally associated with the types of work detailed on this drawing, note the following significant residual risks:

Refer to designers residual risk register for full details

CONSTRUCTION
1.1 DEEP EXCAVATION
1.2 POSSIBLE SERVICES IN THE VICINITY
1.3 TRAFFIC AROUND THE SITE
1.4 GROUND CONDITIONS

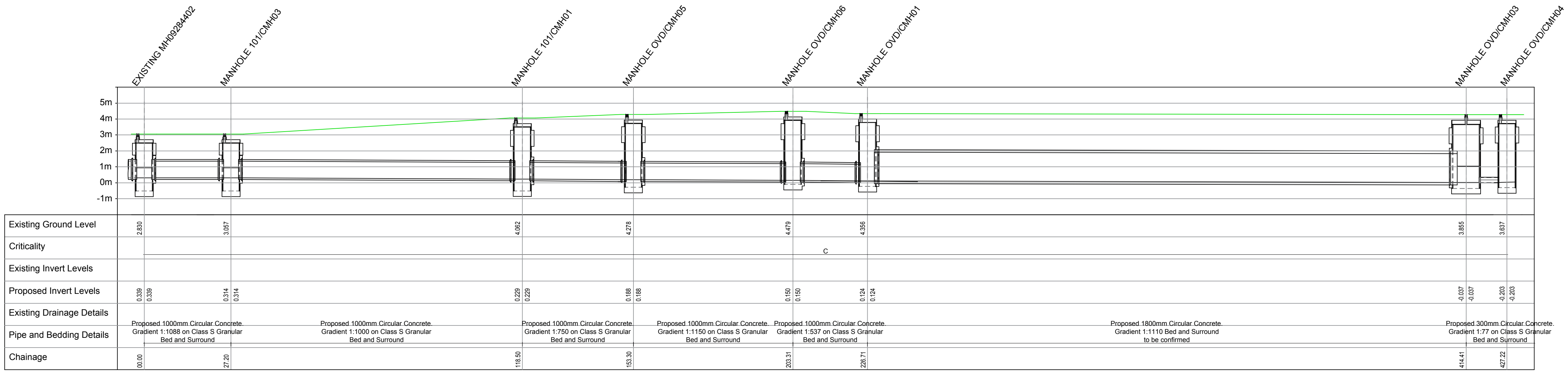
MAINTENANCE
1.1

DEMOLITION
1.1

It is assumed that all works will be carried out buy a competent contractor working, where applicable to an approved method statement

KEY TO HEALTH AND SAFETY SYMBOLS

INDICATES A RESIDUAL RISK REQUIRING A COMPULSORY ACTION.
INDICATES A RESIDUAL RISK FOR INFORMATION.
INDICATES A RESIDUAL RISK REQUIRING A PROHIBITIVE ACTION.
INDICATES A RESIDUAL RISK AS A WARNING.



LONG SECTION
Scale:1:750 H, 1:125 V

LONG SECTION ALONG PROPOSED FOUL SEWER

PRELIMINARY

REV	DATE	BY	COMMENTS	CHK'D	APP'D
B	06/12/16	KAA	FOR COMMENT	TS	RW
A	28/09/16	KAA	FOR COMMENT	TS	RW

MULTIDISCIPLINARY CHECK BOX

REV	DATE	CIVIL	PRO	MECH	ELEC	STRUC	
B	16/11/16	-	-	-	-	-	-

GHD
Troy Mills
Troy Road
Horsforth, Leeds
LS18 5GN
0113 259 1927
UKmail@ghd.com

CLIENT: YorkshireWater

PROJECT TITLE:
A63 CASTLE STREET HULL FEASABILITY

DRAWING TITLE:
PIPE ROUTE LONGITUDINAL SECTION

Barhale
LIVINGSTONE HOUSE, CHADWICK STREET, LEEDS, LS10 1LJ
Tel: 0844 848 1092

SCALE	STATUS	DRAWN BY INITIALS	DRAWN BY DATE	CHECKED BY INITIALS	CHECKED BY DATE	APPROVED BY INITIALS	APPROVED BY DATE
NTS	COMMENT	KAA	28/09/16	TS	28/09/16	RW	28/09/16

BARHALE PROJECT NUMBER	LT0009
GHD JOB NUMBER	CH7410
DATE IN GROUND	NA
DAZ No.	439
NGR	NA
YW DRAWING NUMBER	2016-R0925-LT0009-LS-013
REV	B
SHEET SIZE	A1
YW BATCH No.	NA
YW SOLUTION ID	NA

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PLAN

MH Name	MH1	MH2	MH3	EXMH1203	EXMH0203			EXMH9104	MH8	EXMH8120
PN	1.000	1.001	1.002	1.003	1.004	1.005	1.006	1.007	1.008	
Dia (mm)	1200	1200	1200	1219	1219	1219	1219	1200	1200	
Slope (1:X)	375.0	369.9	221.8	507.3	483.3	503.7	409.1	295.7	451.1	
Cover Level (m)	3.189 2.670 2.734	3.163 3.068 2.948	2.832 3.239 3.061	2.617 2.793 2.818	2.702 2.948 2.934	2.624 2.935 2.926	2.608 2.943 2.944	2.491	2.440	2.530
Invert Level (m)	-0.580	-0.787 -0.787	-1.037 -1.037	-1.280 -1.280	-1.420 -1.420	-1.479 -1.479	-1.543 -1.543	-1.680 -1.680	-1.960 -1.960	-2.000
Length (m)	77.696	92.469	53.886	71.016	28.515	32.239	56.043	82.790	18.043	

LONGITUDINAL SECTION

Key:

Existing Drainage (CAT Surveys)

Proposed Sewer Diversion

CPO Boundary

Rev.	Date	Description	By	Chk'd	App'd	Auth'd
REV10	DATE10	NOTE10	BY10	CHK10	APP10	AUTH10
REV9	DATE9	NOTE9	BY9	CHK9	APP9	AUTH9
REV8	DATE8	NOTE8	BY8	CHK8	APP8	AUTH8
REV7	DATE7	NOTE7	BY7	CHK7	APP7	AUTH7
REV6	DATE6	NOTE6	BY6	CHK6	APP6	AUTH6
REV5	DATE5	NOTE5	BY5	CHK5	APP5	AUTH5
REV4	DATE4	NOTE4	BY4	CHK4	APP4	AUTH4
REV3	DATE3	NOTE3	BY3	CHK3	APP3	AUTH3
REV2	DATE2	NOTE2	BY2	CHK2	APP2	AUTH2
REV1	DATE1	NOTE1	BY1	CHK1	APP1	AUTH1

Suitability
S3 SUITABLE FOR REVIEW & COMMENT

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Client
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Project Title
A63 CASTLE STREET

Drawing Title
YW SEWER DIVERSION - ARCO OPTION

Scale	By	Checked	Approved	Authorised
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P01

A63 Castle Street Improvements, Hull
Environmental Statement

Volume 3 Appendix 11.9

ROAD DRAINAGE AND THE WATER ENVIRONMENT - ADDITIONAL FLOOD RISK ASSESSMENT INFORMATION REQUIREMENTS

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A63 Castle Street Improvements, Hull

Environmental Statement

Appendix 11.9
Additional flood risk assessment information requirements

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1. Introduction

1.1 Background

1.1.1 A meeting was held with the Environment Agency in August 2018 to discuss key outcomes of the flood risk assessment (see Volume 3, Appendix 11.2 Flood risk assessment). During this meeting, it was agreed that additional detailed information relating to flood risk would be provided at a later date. The purpose of this document is to outline and record those additional information requirements for future discussion.

1.2 Additional flood risk information requirements

1.2.1 The following is a list of the additional information to be provided to the Environment Agency:

- Plans showing comparisons of flood extents to identify additional areas of flooding or areas no longer at risk of flooding as a result of the Scheme
- Plans or data tables showing changes in flood depth as a proportion of the existing (baseline) flood depth
- Plans or figures highlighting areas of change in Flood Hazard
- A review of road levels and flood depth information to identify the level at which the road must be constructed in order to manage flooding of the underpass from Humber wave overtopping or River Hull tidal flooding events
- A review of the potential impacts of proposed flood defence upgrades as part of the Humber Hull Frontages scheme based on information on these upgrades to be supplied by the Environment Agency