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QUEENSFERRY CUTTING – HYDROGEOLOGICAL ASSESSMENT

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Table of attachments

Att. no.	Title/Subject	File name	Issuer's doc. ID
01	Constant Rate Pump Test Logger Data	Accompanying electronic data _constant rate test logger data.xlsx	
02			
03			
04			

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1. INTRODUCTION

1.1 Background

The Forth Replacement Crossing (FRC) is to be approached from the south by a new road alignment to the west of South Queensferry passing under the A904 in a cutting. The South Queensferry cutting will be constructed between chainages 3000 to 4300 at a maximum depth of 10m below ground level (mbgl) and a total length of approximately 1300m.

In addition, a temporary launch platform for the new bridge will be constructed within the northern part of the South Queensferry cutting. This is referred to as the South Launch. Between approximate chainages 4110 and 4350, the South Queensferry cutting will be deepened by up to 7m to a maximum depth of 11.5m bGL to accommodate the South Launch excavation, which will have a total length of approximately 200m.

Previous ground investigations have shown that the cutting will intercept groundwater and therefore dewatering will be required.

Dewatering of the proposed cutting and temporary South Launch excavation will be subject to the Water Environment (Scotland) Controlled Activities Regulations 2011 (CAR). Under these regulations, an authorisation is required from the Scottish Environmental Protection Agency (SEPA) to undertake dewatering activities, either by means of a Registration (dewatering abstraction rate between 10 and 50m^3/day) or CAR licence (dewatering abstraction rate greater than 50m^3/day). The CAR licence or registration is required to be in place prior to the works.

An application for a simple CAR licence was submitted by FCBC on 11/11/2011. SEPA issued the licence on 21/02/2012 (reference CSR/S/1098673), which permits abstraction from the excavation between National Grid Reference (NGR) NT 1143 7873 and NT 1172 7738, up to a maximum daily volume of 1,700 m^3/d (+/- 10%).

An earlier application for a complex CAR licence (abstraction volume greater than 2,000m^3/day) was submitted on 15th October 2010 by Jacobs Arup Joint Venture (JAJV), although this was not progressed and the licence was never granted. In support of the application, JAJV undertook a hydrogeological assessment of the local area and developed a SEEP/W model to estimate likely discharge volumes resulting from the dewatering of the cutting. The potential impact of dewatering the cutting on the water table at key receptors within a 1.2km radius (JAJV, October 2010) was also assessed using the Sichardt formula (see **Section 3.2.1**). This search radius is defined in the Regulatory Method (WAT-RM-11) Licensing Groundwater Abstractions including Dewatering V3.0 (SEPA, 2010) for abstractions potentially exceeding 500m^3/day.

JAJV subsequently developed a Modflow model to investigate the potential impacts of dewatering the cutting on groundwater levels (December 2010) and published a report on this analysis entitled Forth Replacement Crossing – Network Connections South – South Queensferry Cutting.

Following award of the FRC Contract, the Forth Crossing Design Joint Venture (DJV) and Forth Crossing Bridge Constructors (FCBC) have carried out a review of the Queensferry Cutting hydrogeological study previously completed by JAJV.

Subsequent to this review, the DJV carried out its own analysis of the dewatering of the Queensferry Cutting. This analysis included a review and assessment of the risks associated with the drawdown of the local water table such as consolidation settlement at key locations in the vicinity of the cutting. These key locations are identified as Echline Corner, Springfield, Linn Mill and the area north of Society Road. A draft report of the findings of this assessment was submitted to SEPA in support of the simple CAR licence application, which was submitted by FCBC on 11/11/2011 (see above).

The draft report did not take into account potential additional impact of the temporary South Launch excavation. Furthermore, FCBC has conducted additional ground investigations since the draft report was issued, and is continuing to undertake a groundwater monitoring programme.

1.2 Scope of This Report

This report considers information gained from the latest ground investigation (FCBC, 2012), which included permeability testing and a pumping test, as well as the groundwater level monitoring undertaken to date. The report also takes into account the profile of the South Launch excavation.

The aims of this report are as follows:
- assess the potential impacts of dewatering on properties at Echline Corner, Springfield, Linn Mill and north of Society Road;
- assess the potential impact of dewatering on a nearby watercourse, Linn Mill Burn; and
- determine the likely maximum dewatering abstraction rate.

Figure 1.1 on the following page shows an aerial photograph of the cutting area and the location of the potential receptors listed above.

Figure 1.1 – Aerial photograph of the cutting area

Note: Approximate location of labelled features shown. For further detail refer to **Figure 1 – Appendix A, Southern Network Connections Key Plan**

2. CONCEPTUAL HYDROGEOLOGICAL MODEL

2.1 Geological Overview

The superficial geology of the area comprises glacial till (Boulder Clay) ranging from approximately 0.5 to 17m thick in the area of the cutting. The glacial till largely comprises clayey deposits, although sand and gravel layers are also present, which may be associated with glacial meltwater channels. However, these are typically neither vertically nor laterally extensive.

The bedrock underlying the till largely consists of mudstones, siltstones and sandstones of the Hopetoun and Calders members of the West Lothian Oil-Shale Formation, although sandstone units are more dominant at the southern end of the Queensferry cutting footprint, close to Echline Corner, and at the northern end of the South launch excavation footprint. These do not appear to be laterally extensive and are also of limited thickness. The southern sandstone thins out to the north at around chainage 3625. The northern sandstone unit thickens to the north from approximately chainage 4110.

Igneous strata, primarily dolerite, are present beneath the sandstone and mudstone, but are more prominent at shallow depths towards the northern end of the cutting. The dolerite is mostly present in the form of sills.

Ground investigations (GI) have been carried out by Ritchies in 2008, 2009 and 2010, and by FCBC in 2011-2012 (FCBC, 2012). Full details of the GI borehole logs can be found in these reports.

Geological cross-sections along and approximately perpendicular to the centre line of the cutting are presented in **Appendix A**. The geological interpretation is based on the GI borehole logs and is discussed in more detail in **Sections 2.5 to 2.8 inclusive**.

2.2 Hydrogeological Overview

The regional groundwater flow direction is towards the Firth of Forth, to the north of the cutting. The recharge area in the vicinity of the cutting appears to be Dundas Hill, which is located to the south west of the cutting.

Superficial deposits are predominantly clayey and therefore are expected to be of low hydraulic conductivity (K), except where gravel and sand horizons are present in significant thicknesses. The principal aquifer units in the area are the sandstones, although the mudstones appear to be significantly fractured in some areas and may be less impermeable than is normally assumed. Groundwater is also present within the upper, more weathered dolerite horizon. Hydrogeological relationships are described in more detail in **Sections 2.5 to 2.8 inclusive**, while hydrographs are presented in **Appendix E**.

2.3 Aquifer Properties

During the previous ground investigations at the South Queensferry Cutting, a number of falling head and packer tests were carried out to determine the hydraulic conductivity (permeability) of the different geological units that will be intersected by the cutting. Data from the 2009 and 2010 tests were reported on by JAJV and used to define hydraulic conductivity values to inform the SEEP/W and Modflow models. Subsequent ground investigations undertaken by FCBC have allowed new boreholes in the South Queensferry Cutting area to be tested. These latest falling head tests, undertaken in February 2012, focussed on the sandstone and mudstone units. Results have been analysed by the DJV.

A number of the hydraulic conductivity values derived from the original falling head tests are considered to be very high for the geological unit under test. For example, a typical, published, mid-range value for glacial till is in the order of 10^{-8} or 10^{-9} m/s (Freeze and Cherry, 1979) whereas the most conservative estimates from the data presented in **Table 2.1** (from the falling head tests carried out in S86 and

DPS35) are two orders of magnitude higher than this (i.e. K values of 10^{-6} to 10^{-7} m/s). The driller's records for these particular tests indicate that water had been leaking around the side of the casing, inferring that the annulus of these boreholes was not appropriately sealed. Therefore, these results have been disregarded with respect to the calculations undertaken by DJV.

Table 2.1 records all the hydraulic conductivity results quoted in JAJV (October 2010) and those from more recent tests undertaken by FCBC (analysed by DJV). The table also identifies which of the results are considered to be questionable when compared with well-established published values and which are considered to be more representative of the relevant geological unit under test. The falling head test analysis undertaken by DJV is included in **Appendix D**.

JAJV's calculated hydraulic conductivity values for some sandstone boreholes are high compared to book values; Freeze and Cherry (1979) suggested a range of 1×10^{-10} – 1×10^{-6} m/s for a typical sandstone. However, hydraulic conductivities derived from the recent tests analysed by DJV appear to be similar, varying by three orders of magnitude (5×10^{-7} - 4×10^{-5} m/s).

Hydraulic conductivities derived from permeability tests are also much higher for mudstone units (up to 10^{-5} m/s) than would be expected from book values (10^{-13} – 10^{-9} m/s; Freeze and Cherry, 1979). This may be due to the extensive fracturing observed within the mudstone during the most recent GI (FCBC, 2012) and indeed similarly high hydraulic conductivity values were derived from some of the recent tests.

The dolerite hydraulic conductivity value of 2×10^{-5} m/s is likely to be a reflection of the uppermost, weathered horizon, which is extensively fractured.

Table 2.1 – Hydraulic Conductivity (K) Values Derived from Permeability Tests (JAJV, October 2010; FCBC, March 2012)

Falling Head Test Borehole	Test Type & Depth (mbgl)	Comments	Derived K value (m/s)	Geological Unit under Test
SUPERFICIAL DEPOSITS				
S86 [1]	1.70	The test was terminated after 20 minutes with water seeping up outside the casing. **Suggests questionable test result when compared with published parameters.**	5.95×10^{-6}	Firm slightly sandy slightly gravelly clay. Weathered glacial till
S87 [1]	2.00	The test results appear in line with expected values	2.78×10^{-8}	Firm grey mottled orange brown slightly gravelly sandy clay
S89A [1]	2.00	Water level dropped to 0.25 after 1 minute and remained there for the entire test of 180mins. **K value may not be representative when compared with published parameters.**	1.18×10^{-7}	Firm locally stiff dark brown and grey mottled slightly gravelly sandy clay
S91A [1]	2.50	The test results appear in line with expected values	1.45×10^{-7}	Loose dark brown grey silty gravelly fine to coarse sand.
DPS31 [1]	4.50	The test results appear in line with expected values	7.27×10^{-8}	Stiff grey Boulder Clay

DPS33 [1]	4.50	The test results appear in line with expected values	1.65×10^{-9}	Stiff grey Boulder Clay
DPS34 [1]	4.50	The test results appear in line with expected values	6.17×10^{-8}	Boulder Clay
DPS35 [1]	4.50	Seems a high K value for stiff clay. **Similar K value to S86 where water was escaping. Suggests questionable test result when compared with published parameters.**	4.55×10^{-6}	Stiff grey Boulder Clay
DPS36 [1]	4.50	Test stopped after 1 hour due to end of shift. However, test results appear in line with expected values.	3.64×10^{-9}	Boulder Clay
BEDROCK				
S77 [1]	2.50	Between 25-30 mins water level rebounded by 0.4m. It then continued to drop thereafter? **DJV would question this result.**	1.65×10^{-6}	Moderately strong medium bedded brown medium grained **sandstone**, slightly weathered.
S78 [1]	4.00	Reached 1.44mbgl after 30 minutes **but remained there for the next 60 mins. As such, K value could be unrepresentative**	6.03×10^{-6}	Moderately strong grey fine to medium grained **sandstone**, slightly weathered.
S79 [1]	2.00	2 tests. TEST 1 - only 20 mins long. TEST 2 - Water seen outside casing, hole not sealed. **K value may be unreliable.**	4.52×10^{-5}	Fine grained **limestone** 2 - 2.25m going into **mudstone** at 2.45m.
S84 [1]	3.10	The test results appear in line with expected values	5.77×10^{-7}	Strong grey **siltstone**, boundary with mudstone at 3.1mgl
DPS27 [1]	10.00	The test results appear in line with expected values	6.64×10^{-8}	**Sandstone** (open holed)
DPS28 [1]	10.00	The test results appear in line with expected values	2.23×10^{-10}	Dark **shale**
DPS29 [1]	10.00	The test results appear in line with expected values	9.83×10^{-8}	Dark **shale**
DPS37 [1]	4.50	First attempt at test failed, unable to maintain head of water. Possibly indicative of an **unreliable K value.**	7.80×10^{-6}	Strong **dolerite**.
CSRO03A[2]	5.50	The test results appear in line with expected values	6.58×10^{-7}	Light brown fine to medium grained **sandstone**. Boundary with interbedded sandstone and black mudstone at 5.50mbgl
CSRO03B[2]	6.00	The test results appear in line with expected values	3.97×10^{-6}	Light brown **sandstone**

CSRO04A[2]	10.00	This value appears high for a mudstone but could indicate preferential flow through sandstone horizons and/or fractures within the mudstone[3].	1.71×10^{-6}	Interbedded **sandstone and mudstone**
CSRO05A[2]	13.30	The test results appear in line with expected values	4.95×10^{-7}	Light brown fine grained **sandstone**
CSRO06B[2]	4.00	The test results appear in line with expected values	6.00×10^{-6}	Orange **sandstone**
CSRO07A[2]	8.00	This value appears high for a mudstone but could indicate preferential flow through sandstone horizons and/or fractures within the mudstone[3]. Two tests were carried out and water returned to rest level within 2 minutes.	4.10×10^{-5}	Interbedded fine grained grey **sandstone and black mudstone**
CSRO08A[2]	7.00	This value appears **very high** for a mudstone but could indicate the presence of fractures within the mudstone[3]. First attempt at test failed as unable to maintain head of water. Possibly indicative of an **unreliable K value**.	4.85×10^{-5}	Black **mudstone**
CSRO09[2]	11.00	This value appears high for a mudstone but could indicate the presence of fractures within the mudstone[3]	4.13 a 10^{-6}	Dark blue grey **mudstone**
CSRO09A[2]	4.50	This is a high value of K for a typical igneous rock, but in line with a fractured igneous rock	2.51×10^{-5}	Light grey **dolerite**

Packer Test Borehole	**Centre of Packered Interval (mbgl)**	**Packer Test Interval and Comments (m)**	**Derived K Value (m/s)**	**Geology**
S77 [1]	4.95	4.4 - 5.5	1.05×10^{-6}	Moderately strong dark grey fine grained **sandstone**, slightly weathered.
S78 [1]	5.50	4.4 - 5.5 - Period 3 unable to achieve max pressure. Full water flow applied and pressure monitored.	9.31×10^{-6}	Moderately strong grey fine to medium grained **sandstone**, slightly weathered.
S80 [1]	5.50	5.0 - 6.0	6.89×10^{-7}	Moderately strong thinly bedded medium grained brown **sandstone** slightly

				weathered with **siltstone** inclusions.
S19 [1]	8.75	8.3 - 9.2	1.68×10^{-6}	N/A
S79 [1]	5.25	4.5 - 6.0	2.72×10^{-7}	Carbonaceous **mudstone**. Partially
S81 [1]	5.50	5.0 - 6.0 - Stage 3 data not used in calculation of K. Full flow not maintained at 0.75bar.	1.17×10^{-5}	Moderately weak dark grey **mudstone** with some thin laminations of carbonate.
S84 [1]	8.75	8.3 - 9.2	1.63×10^{-6}	Moderately strong locally strong thinly laminated dark grey **mudstone**.

Notes:
[1] JAJV (October, 2010).
[2] Results from falling head tests conducted by FCBC on new boreholes, analyses by DJV (March, 2012).
[3] Mudstone reported to be fractured/broken (visual observations)

A short constant rate pumping test was undertaken at CSRO10, close to the proposed gyratory near Echline Corner on 30th May 2012. Full details of the pumping test are presented in **Appendix C**. Despite a very low pumping rate (0.12 l/s), it was not possible to achieve a quasi-steady state condition during the test because of the extremely limited inflow to the borehole. Analysis of the test results gave an estimated sandstone K value of 7.34×10^{-7} m/s, at the low end of the range derived from the falling head and packer tests. This value is likely to most closely represent the bulk permeability of the sandstone unit within the Queensferry cutting footprint at Echline Corner, particularly as the sandstone is not massive but contains interbedded mudstones.

2.4 Receptors

The receptors considered are the residential properties situated on Echline Corner, Linn Mill, north of Society Road, and the Springfields: - including Springfield Terrace, Springfield Place and Springfield Lea, and Linn Mill Burn. These residential areas and Linn Mill Burn are shown on **Figure 1 - Sections Key Plan**, presented in **Appendix A**, and are represented on the cross-sections described in the following **sections 2.5 to 2.8 inclusive**.

2.5 Detailed Geology and Hydrogeology

The detailed hydrogeological conceptual model is represented by the conceptualised cross-sections A to F inclusive and H to J inclusive, shown on **Figures 2 – 9** in **Appendix A**. Section lines are shown on **Figure 1** in **Appendix A**. Cross-sections orientated roughly perpendicular to the centre line of the cutting (sections A to F inclusive) were selected to target the properties at Echline Corner, Springfield, Society Road and Linn Mill, as well as the watercourse, Linn Mill Burn. The lines of section also take into account key boreholes for which geological and hydrogeological information is available. Long sections G, K and L are orientated along the centre line of the cutting.

All cross-sections show the topographic profiles prior to and following construction. Where relevant, the sections also show the profile of the South Launch excavation.

For the purposes of describing the conceptual hydrogeological model and calculating inflows, the excavation has been split into three areas: Ecline; Springfield (north); and Springfield (south), represented by long-sections G, K and L. The geology, hydrogeology and topography represented by these long-sections are described below. For the impact analysis described in **Section 3** it was considered appropriate to split the Springfield area into two segments: Springfield (north), which includes the South Launch; and Springfield (south), which covers the area of the cutting south of the South Launch. For the assessment of potential impacts on the properties at Linn Mill and north of Society Rd, as well as on Linn Mill Burn, the hydrogeological conceptual model for the northern segment, Springfield (north) has been used.

Water level data (described in **Section 2.6**) from the monitoring boreholes intersected by the cross-sections in Appendix A, have been plotted in a series of hydrographs (Appendix E).

2.6 Echline Corner

2.6.1 Geology

Cross-sections A, B, C and H presented in **Figures 2, 3 and 4** in **Appendix A** are orientated south west to north east and show the base of the cutting at approximately 7 - 10m below current ground level. The geological logs for boreholes in proximity to the section lines indicate that the geology in the area consists of a thin layer of clay, silt and sandy superficial deposits, up to 2m thick, underlain by sandstone. A 2m thick band of gravel is present at approximately 2mbgl at the location of the proposed cutting, which yielded water strikes at around 2mbgl during the drilling of DPS22 and DPS23. Sandstone thickness within the cutting decreases to the north (see long-section G).

The greatest thickness of sandstone (13m) was intersected by CSRO10 (cross-section B) and close to this location, approximately 6m of the sandstone unit would be intersected by the cutting e.g. at DPS22 (cross-section A). The sandstone is underlain by mudstone extending to the base of all the boreholes in the vicinity of the proposed cutting. Geological logs for the recently constructed boreholes suggest that the sandstone unit may actually comprise sandstone with interbedded mudstones in some areas (e.g. in CSRO03A). Dolerite bedrock underlies the mudstone. This was encountered in DPS20 and BHS1021 (cross-sections A and C respectively) located to the south west and centre of the cutting. BHS1021 shows the top of the dolerite at least 10m below the base of the cutting.

Although no borehole logs are available within the area of housing, the logs for DPS24 (cross-section A) and CSRO04A (cross-section C) suggests that the sandstone unit extends east of the cutting to the edge of the housing area. The sandstone is overlain by around 4m of superficial deposits, mudstone and siltstone here.

2.6.2 Hydrogeology

A number of boreholes in and around the area of the cutting have been installed as monitoring wells and are routinely monitored for groundwater levels. The installations, monitoring horizons and water levels are summarised in **Table 2.2** below. The full list of monitored boreholes and results are presented in the Groundwater Monitoring Report (DJV, 2012), while hydrographs for the monitoring boreholes shown in cross-sections A to L are presented in Appendix E. Data collection has been intermittent with around 10-12 water level dip measurements taken over the period May 2009 to May 2012 for boreholes denoted by DPS and S, and around 15-18 dip measurements taken over the period October 2009 to May 2012 for boreholes denoted by BHS. In addition, 5 dip measurements were taken between February and May 2012, in boreholes drilled in the most recent GI (denoted by CSRO).

Table 2.2 – Groundwater Monitoring Echline Corner

Borehole	Installation	Depth (mbgl)	Water Level Range (mbgl)	Monitored Geology
DPS20	Piezometer	8.0	0.53 – 0.98	Siltstone
DPS21	Piezometer	7.0	0.62 – 1.84	Mudstone
DPS22	Screened standpipe	1.0 – 15.5	0.56 – 1.92	The screened section intersects gravel, sandstone and mudstone.
DPS23	Piezometer	2.0	0.7 – 2.21	Gravel
DPS24	Piezometer	3.5	1.4 – 2.10	Clay, sand and gravel
BHS1021	Screened standpipe	1.5 - 20	1.79 – 2.37	Mudstone, sandstone, gravel and clay
BHS1019	Screened standpipe	2 – 14.5	3.77 – 5.13	Sandstone
S77	Screened standpipe	1.5 - 3	0.48 – 1.69	Sandstone
S78	Screened standpipe	2 - 13	0.85 – 2.31	Sandstone
S80	Screened standpipe	1 – 3.5	0.95 – 3.89	Clay
CSRO03A	Screened standpipe	3.5 – 5.5	3.43 - 3.93	Sandstone
CSRO03B	Screened standpipe	2.0 – 6.0	0.83 – 1.71	Sandstone
CSRO04A	Screened standpipe	5.0 – 11.0	2.05 – 2.37	Mudstone
CSRO05A	Screened standpipe	2.5 – 13.3	1.62 – 3.74	Sandstone
CSRO06B	Screened standpipe	3.0 – 4.0	0.70 – 1.56	Sandstone
CSRO07A	Screened standpipe	6.5 – 8.0	0.80 – 1.44	Sandstone (interbedded with mudstone)
CSRO08A	Screened standpipe	5.5 – 7.0	0.55 – 1.01	Mudstone
CSRO10 (pumping test BH)	Screened standpipe	2.0 – 13.0	1.52 – 3.08 (natural water levels)	Sandstone

Cross-sections A – C presented on **Figures 2 – 3** in **Appendix A,** along with the data presented in **Table 2.2,** show that the screened sections of the boreholes sometimes intersect more than one geological unit, although the boreholes constructed as part of the most recent GI (FCBC, 2012) were designed to avoid this as much as possible. This means that in some cases, it is not possible to determine groundwater levels in the individual aquifer units that will be intersected by the cutting. However, it is apparent that groundwater is present in the shallow superficial deposits, particularly where they contain local sands and gravels.

Cross-section A indicates that the most significant aquifer unit that would be intersected by this section of the cutting is the sandstone, of which approximately 6m will have to be dewatered prior to or during the works, and the sand and gravel horizons within the superficial deposits.

Hydrographs presented in **Appendix E** show that groundwater levels in the sandstone unit vary seasonally by up to around 2m, and in the more permeable superficial deposits by up to 1m.

2.6.3 Topography

The ground level of the properties at Echline corner is approximately 4m above the proposed base of the cutting, so it is considered reasonable to assume that dewatering only this 4m section of the sandstone unit has the potential to impact the water table underlying these properties (refer to cross-section A on **Figure 2, Appendix A**). The South Launch does not affect this area.

2.7 Springfield (south)

2.7.1 Geology

Cross-sections D and E presented on **figures 4 and 5** in **Appendix A** show significantly less sandstone in the area of the cutting than further south adjacent to Echline Corner. Superficial deposits in the vicinity of the proposed cutting consist of silt, sand and clay with occasional boulders. The maximum thickness of the superficial deposits encountered during the ground investigations in this area of the cutting was 10m in BHS1024. In general, these deposits are underlain by a thin layer of either mudstone or sandstone before the dolerite bedrock is encountered. In the west, at DPS37, the dolerite is encountered close to ground level, while further to the east the dolerite is encountered at 8mbgl in BHS1027A and 10mbgl in BHS1027.

The proposed cutting at this location has a maximum depth of approximately 7mbgl, as shown on cross-sections D and E, although within the South Launch excavation this increases to around 11m to accommodate the temporary access track (Section E). The geology that will be intersected by the cutting is limited to the superficial deposits, principally sandy gravelly clay with some boulders. However, the lowest 4m of the South Launch excavation will cut into sandstone as evidenced by DPS39 (Section E) and possibly also gabbro/dolerite (BHS1027/A).

There are no records of boreholes that have penetrated the bedrock close to the area of housing, so it is not possible to comment on how far the sandstone extends to the east of the cutting. However, the log for S94 (cross-section D) shows that sandstone is present close to the western edge of the housing development to the south of this.

2.7.2 Hydrogeology

Boreholes BHS1024 and BHS1027A were installed in the drift with screened sections intersecting more than one geology type. DPS39 was installed with a screened section intersecting both the drift deposits and the sandstone. This makes it difficult to assess water levels and hydrogeological properties in individual aquifer units. Groundwater levels in both BHS1024 and 1027A are within the sand and gravel horizons located between 0.5 and 1.0mbgl. Hydrographs for these boreholes (**Appendix E**) show that groundwater levels behave differently in these two boreholes, however, which suggests hydraulically separate groundwater bodies. A water strike was encountered at approximately 10.5mbgl, at the top of the dolerite, during the drilling of BHS1024. Water was also encountered in the trial pit TPS96 at 1.8mbgl in the Boulder Clay.

This suggests that groundwater in the uppermost, weathered section of the dolerite is hydraulically isolated from any drift aquifers and that the latter are likely to be perched. As DPS39 was completed across more than one geological unit, it is not possible to comment on groundwater levels in the sandstone except that the unit appears to be largely unsaturated at this location, with a groundwater level just above or just below the base of the South Launch excavation (see hydrograph in **Appendix** E).

The above evidence suggests that the majority of the cutting represented by cross-sections D and E is unlikely to yield significant amounts of water as no significant aquifer unit will be encountered other than the shallow, thin sand and gravel horizon and the sandstone unit, which appears to be largely unsaturated to the base of the excavation.

DPS24 to the east of the cutting and closer to the area of housing monitors groundwater levels in a sand and gravel horizon within the superficial deposits. During construction of this borehole, water strikes were recorded in the gravel horizon and in the sandstone, so it is likely that the gravel horizon represents a perched aquifer. The hydrograph for this borehole (**Appendix E**) shows that water levels vary by up to 1m.

2.7.3 Topography

Cross-sections D and E show that the Springfield residential area is a maximum of 5m above the base of the cutting and 8m above the base of the temporary South Launch excavation. The proposed ground level includes two raised bunds either side of the cutting (refer to cross-sections D and E on **Figures 4 and 5, Appendix A**).

2.8 Springfield (north)

2.8.1 Geology

Cross-sections M, F, I and J presented on **Figures 5, 6 and 7** in **Appendix A** show that up to 15m of sandstone is present beneath the overlying superficial deposits, which are similar to those encountered further to the south, i.e. silt, gravelly sand and clay with occasional boulders. The maximum thickness of the superficial deposits encountered during the ground investigations in this area is around 6m as shown at BHS1029, BHS1028 and DPS40A.

The permanent cutting will only intersect the superficial deposits, but the temporary South Launch will intersect the sandstone unit. The South Launch excavation will also intersect dolerite and possibly basalt towards the base in the area represented by cross-section M. Further to the west, at CSRO09, the dolerite is encountered closer to ground level (at 37.5 mAOD).

2.8.2 Hydrogeology

Fewer boreholes have been installed with monitoring equipment in this area. On Cross-section F, DPS40A (now removed) had a screened standpipe section covering several different geological units. Water levels measured between 10.5 and 11.3 mbgl, close to the top of the sandstone unit at this location. BHS1028 has a screened standpipe section in the upper drift layers, which comprise clays and gravels, with water levels measured between 0.6 and 1.4 mbgl. 75m to the west of the cutting, the mudstone and dolerite in CSRO09 and CSRO09A are monitored, showing water levels between 1.2 and 2.0mbgl, and 0.8 and 1.5mbgl respectively (see Cross-section M). When these boreholes were drilled water strikes at the top of the dolerite were recorded.

The main aquifer unit that will be intersected by the South Launch excavation is the sandstone, although BH1029A (Section M) shows that the uppermost, weathered horizon of igneous rock will also be intersected at this location. If, in the worst case scenario, water is found to the top of the sandstone aquifer unit, a maximum of 4m to 15m of saturated sandstone will be dewatered on excavation (depending on use of cross-section M or I as a model respectively). However, the groundwater levels in DPS40A, although close to the top of the sandstone, are equivalent to the base of the South Launch excavation. This suggests that, as is the case in the Springfield (south) area, the sandstone may be unsaturated to the base of the excavation.

As elsewhere, a limited amount of dewatering may be associated with more permeable horizons in the superficial deposits, but these do not appear to be laterally or vertically extensive.

2.8.3 Topography

Longitudinal sections G, K and L (**Figures 8 and 9** in **Appendix A**) cover the length of the cutting. Section L shows the topographical relationship between the proposed ground level and the current ground level to the properties north of Society Road. The section shows that the base of the South Launch excavation will be a minimum of 8.5m above the ground level at these properties. Cross-sections M and F demonstrate that the base level of the South Launch excavation will be between 7.5 and 13m (maximum) below the ground level at the Springfield (north) properties. Section F also shows that properties at Linn Mill and Linn Mill Burn are approximately 5.5m and 2.5m above the base of the South Launch excavation respectively.

The base of the permanent road cutting will remain above the elevations of these properties and Linn Mill Burn, so dewatering of the cutting will not affect ground water levels at these receptors. Whether the temporary South Launch excavation, with a maximum depth of approximately 11.5mbgl, will significantly impact on local groundwater levels depends on whether the sandstone unit and underlying weathered igneous rock proves to be largely unsaturated to this depth (i.e. base of excavation just above or just below the water table).

3. POTENTIAL IMPACT ON GROUNDWATER LEVELS

3.1 Review and Analysis of Work Carried out by JAJV

3.1.1 SEEP/W Model

JAJV (October 2010) estimated potential inflows into the permanent cutting using a 2-d finite element model, SEEP/W, based on four hydrogeological cross-sections roughly perpendicular to the cutting. For each cross-section, SEEP/W calculated the inflow per metre length of cutting. This was then scaled up to provide total inflow for each section of the cutting represented by the four cross-sections.

Modelling was undertaken for two scenarios.

1. Inflow to the full depth of the cutting, under naturalised conditions (static groundwater level prior to construction of the cutting).
2. Inflow to the full depth of the cutting under operational conditions (groundwater level following completion of the cutting).

Both scenarios assumed steady-state conditions (i.e. no change in groundwater level, inflow etc. with time).

In reality, DJV consider that these assumptions are likely to overestimate inflows for a number of reasons as detailed below:

1. The SEEP/W model assumed a single groundwater level, inferring that the individual aquifer units (permeable drift (till), sedimentary bedrock and weathered igneous bedrock) are in full hydraulic continuity such that they act as a single aquifer unit albeit with different zones of hydraulic conductivity. In fact, the conceptual hydrogeological model discussed in Chapter 2 shows that multiple aquifers are present, and that only the sandstone and upper, weathered igneous rock are likely to be in hydraulic continuity and that even then, the sandstone contains interbedded mudstone units. Perched aquifers are present within the superficial deposits in the form of sand and gravel horizons within the more impermeable, clayey glacial till, but these are of very limited thickness and extent, and will become rapidly dewatered.

2. The aquifer units intersected by the excavation are relatively thin, which means that as the local water table lowers in response to pumping, the saturated aquifer thickness will decrease and therefore the transmissivity (a measure of the aquifer's ability to transmit water) will reduce significantly.

   Transmissivity (T)

   $T = Kb$ where:

   K = hydraulic conductivity
   b = saturated aquifer thickness.

3. These thin aquifer units will be dewatered gradually and in succession as the cutting is deepened. For the purposes of their SEEP/W model, JAJV assumed that the cutting would be instantaneously excavated to its full depth. Therefore, as stated in its report, the calculated initial inflows (model scenario 1) were conservative; the assessment of CAR requirements and calculation of the radius of influence were based on these.

4. The SEEP/W model assumed three aquifer units:

   a. glacial till ($K = 1.2 \times 10^{-6}$ or 6.0×10^{-6} m/s);
   b. sedimentary bedrock ($K = 4.5 \times 10^{-5}$ or 8.1×10^{-6} m/s); and
   c. weathered igneous bedrock ($K = 8.1 \times 10^{-6}$ m/s).

As discussed in **Section 2.3**, the hydraulic conductivities of the superficial deposits (primarily glacial till) are considered to be over-estimated. The till largely comprises clay or sandy clay with gravel and boulders, whereas the hydraulic conductivity assumed above is more representative of a much more permeable sand and gravel unit. Sand and gravel units, where present, are not particularly extensive and are only a metre or so thick at most. Therefore, storage will be limited and they will become rapidly dewatered, significantly reducing the transmissivity.

The hydraulic conductivity of a clayey till is typically orders of magnitude less than that for a sand and gravel, which means that where the cutting intersects clay or sandy clay, inflows are likely to be small.

The range of hydraulic conductivity values for the sandstone also appears to be overestimated by roughly one order of magnitude in comparison with both book values and those derived from the tests conducted by FCBC (see **Section 2.3**). However, whilst the mudstone values calculated by JAJV also appear to be very high (range is more typically 1 x 10^{-13} – 1 x 10^{-9} m/s; Freeze and Cherry, 1979), similarly high values were derived from some of the tests conducted by FCBC, possibly because the mudstone is fractured.

5. JAJVs SEEP/W model assumed a flat topographical surface. In reality, the ground elevations of the properties north of Society Road are below the base of the cutting, which means that the permanent cutting can be dewatered without potentially affecting groundwater levels below these properties.

The SEEP/W model predicted initial inflows into the permanent cutting of between 951 and 4,845 m^3/day, with steady state inflows between 54 and 264 m^3/day. The initial inflows are at least an order of magnitude greater than those predicted by DJV even under a worst-case scenario (**Section 3.2**).

3.1.2 MODFLOW Model

JAJV also developed a 2-d steady state Modflow model of the superficial deposits (essentially Boulder Clay) to assess potential settlement arising from dewatering of the cutting (JAJV, December 2010). The model was not used to assess inflows to the excavation but did predict a groundwater level impact, which could result in a measured total settlement in the order of 10mm at Echline Corner.

Whilst DJV has been able to review the output from this Modflow model, a technical note detailing the conceptual model on which the Modflow model was based is not available. However, telephone discussions with JAJV's hydrogeologist have indicated the following:

1. Modflow was run in steady state mode to simulate the "naturalised" (pre-cutting) hydrogeological condition. A time-variant model, incorporating the temporal variations in rainfall/recharge and groundwater levels could not be undertaken due to data limitations. The permanent cutting was then inserted into the model, which again was only run in steady state mode in order to predict the long-term equilibrium groundwater levels.

2. The delineation of the model's geographical extent was based on the surface water natural catchment as this was understood to be a good representation of groundwater recharge and water balance in the area of interest.

3. As with the SEEP/W model, the Modflow model had two aquifer layers representing the superficial deposits and bedrock, with two zones within the bedrock layer to represent the sandstone and the uppermost weathered igneous strata. Hydraulic conductivity values for each layer were as follows (JAJV pers. comm.):

- horizontal hydraulic conductivity in Layer 1 (till deposits): 0.10368 m/d (1.2 x 10^{-6} m/s), average value from aquifer test results;
- horizontal hydraulic conductivity in Layer 2, Zone 1: 0.69984 m/d (8.1 x 10^{-6} m/s), average value from aquifer test results. Layer 2, Zone 2: 0.3 m/d (3.47 x 10^{-6} m/s), based on model calibration and the CSM: this area is known to comprise predominantly of the intrusive sill and hence is expected to have a slightly lower permeability than the sedimentary rock;
- vertical hydraulic conductivity in Layer 1: 0.051 m/d (5.9 x 10^{-7} m/s);
- vertical hydraulic conductivity in Layer 2, Zone 1: 0.35 m/d (4.05 x 10^{-6} m/s); and
- vertical hydraulic conductivity in Layer 2, Zone 2: 0.15 m/d (1.74 x 10^{-6} m/s

The assumptions made with respect to hydraulic conductivity values were much the same as those in the SEEP/W model, i.e. that JAJV appear to have overestimated the hydraulic conductivity of the superficial deposits and also possibly the sandstone. In particular, like the SEEP/W model, the JAJV Modflow model assumed that all of the superficial deposits, including the more clayey till, have a minimum hydraulic conductivity of 1.2 x 10^{-6} m/s, which is more indicative of a sand and gravel. The vertical hydraulic conductivities are also high and infer good hydraulic connectivity between the different units, whereas this is known not to be the case.

The hydraulic conductivity values adopted in the Modflow model were derived during model calibration, whereby modelled water levels were compared with measured ("naturalised") water levels in specific boreholes along the proposed scheme. Water levels were averaged for each borehole. Calibration of this steady state model could only be undertaken on averaged groundwater levels. To make the model "fit", hydraulic conductivity values had to be set high, particularly for the drift and sedimentary bedrock layers, even where borehole logs show these to be poorly permeable. It should be noted that the completions of the monitoring boreholes constructed prior to the most recent GI undertaken by FCBC are such that in many cases they intersect more than one aquifer unit. Therefore it is questionable whether monitored groundwater levels truly reflected actual hydrogeological conditions.

The model was extremely sensitive to variations in hydraulic conductivity. It is not known whether sensitivity analysis was also carried out on other aspects of the model, such as grid spacing.

Unlike the SEEP/W model, the Modflow model did take into account topography (based on a 1:25,000 map).

The above discussion details reasons why we believe that the JAJV models may have overestimated both inflows to the permanent cutting and the potential impact on groundwater levels beneath the Echline Corner properties.

3.2 DJV Analysis Methodology

3.2.1 Review of Methods for Estimating the Radius of Influence of a Dewatering Abstraction

The following industry standard documents have been referred to with respect to estimating the impact on groundwater levels of dewatering an excavation:

- Groundwater control – design and practice. CIRIA 515, 2000;
- Hydrogeological impact appraisal for dewatering abstractions. Environment Agency (EA) science report SC040020/SR1, 2007; and
- Regulatory method (WAT-RM-11) Licensing groundwater abstractions including dewatering. SEPA, 2010.

In addition, internet research has been undertaken and an on-line discussion with dewatering experts in the UK, Europe and US conducted through the Hydrogeology Forum discussion page on Linkedin:

(http://www.linkedin.com/groupAnswers?viewQuestionAndAnswers=&discussionID=80897396&gid=703977&commentID=58576497&trk=view_disc&ut=0BHJmQpe6r1Rk1).

Some of the correspondents also provided additional information outside of this forum. A telephone discussion was also held with the principal author of the EA (2007) report.

There are a number of analytical equations for calculating the extent of the impact of dewatering, presented in the first two of the documents listed above.

The empirical Sichardt formula presented in both CIRIA (2000) and EA (2007) is a very commonly used method for estimating the radius of influence (R_0) under steady state conditions and assuming radial flow:

$R_0 = C (H_0 – h_w) \sqrt{K}$

where:

> H_0 = water level above the base of the aquifer prior to dewatering (i.e. at R_0)
> h_w = water level at the equivalent radius (r_e) of the excavation (i.e. the water level required to dewater the excavation)
> Therefore $H_0 – h_w$ = target drawdown
> K = hydraulic conductivity of the aquifer
> C = an empirical calibration factor.

CIRIA 515 recommends that C = 3000 and this is the value adopted in JAJV (2010). However, EA (2007) notes that a value of 3000 is more appropriate for radial flow, while a value of 1500 – 2000 should be used for linear flow into a trench (as the cutting best approximates to a trench).

However, EA (2007) also notes that the origins of the Sichardt formula are obscure and the assumptions behind the value of C unclear. Furthermore, the Sichardt formula does not take into account whether an abstraction is influenced by a recharge boundary such as a river, or a low permeability boundary such as a fault.

It appears that the Sichardt formula was derived from a series of pumping tests carried out in an unconfined, granular aquifer in the 1930s (Sichardt and Kyrieleis, 1930). Although it is the most commonly used equation to estimate inflows to an excavation, it can apparently under-estimate the radius of influence and over-estimate the inflow rate (when combined with the Dupuit-Thiem or Thiem equations for unconfined or confined aquifers respectively) except in the case of a very permeable gravel aquifer. The reason that it is so commonly used is because of its simplicity and also because dewatering investigations often focus on the rate of inflow and the Sichardt equation generates a conservative value.

Other equations that can be used to calculate the radius of influence under steady state conditions include those of Lembke (1886, 1887; referenced by Gidahatari, 2011), Niccoli *et al*. (1998), Jacob (Bear, 1979; and http://www.aqtesolv.com/forum/roi1.asp) and Weber (Sichardt and Kyrieleis, 1930; and Gidahatari, 2011).

Some of these (Lembke, Weber) are similar to the Sichardt formula is that they are empirical equations (derived from observations) that allow an estimation of the radius of influence to be made, even when some key parameters required by theoretical equations are not known. However, the disadvantage of these is that, like the Sichardt formula, assumptions have to be made and their accuracy depends on how closely the hydrogeological conditions under investigation approach those from which the empirical equations were derived. Gidahatari (2011) compared a number of empirical equations and found that the calculated radius of influence varied significantly.

Equations derived from well hydraulics theory should be more robust, though again the degree that the calculated radius of influence approaches reality will depend on how closely the assumptions behind the equation approach the hydrogeological conditions under investigation. Common steady state approaches include Thiem, 1906 (Kruseman and de Ridder, 1994), who used two or more water level monitoring points to calculate the transmissivity of an aquifer. If the drawdown at two monitoring boreholes and the transmissivity (hydraulic conductivity x aquifer thickness) is known, then the radius of influence can be calculated. Niccoli *et al.* 1998 (EA, 2007) derived a method to estimate the drawdown at a given radius of dewatering a pit (effectively a large diameter well) if other parameters can be estimated with reasonable accuracy, including the height of the saturated seepage face, recharge and the naturalised water level at the radius of influence. However, this calculation assumes radial flow with the pit walls approximating to a cylinder. Moreover, the likely height of the seepage face is not known.

The alternative approach is to develop a numerical model, such as JAJV's Modflow model discussed in **Section 3.1.2** above. The main advantage of a numerical model is the ability to more closely simulate the conditions under investigation, for example variations in the lateral and vertical extent, and hydraulic properties of the different aquifer units, as well as recharge and discharges. However, numerical models are only as good as the data and level of detail behind them. Over-simplifying the conceptual hydrogeological model, which in this instance is complex (refer to **Chapter 2**), may result in model output that has the same degree of uncertainty as that derived from analytical equations. For example, as discussing in **Section 3.1** above, by calibrating its Modflow model against averaged groundwater levels in boreholes that are mostly open to more than one aquifer unit (because this data was all that was available at the time), JAJV (December 2010) appear to have adopted hydraulic conductivity values that are significantly higher than those derived from permeability testing, particularly for the superficial deposits.

EA (2007) recommends a slightly different approach that was also recommended by some of the Hydrogeology Forum on-line discussion correspondents. EA (2007) aimed to provide practical guidance to EA staff on how to assess the hydrogeological impact of groundwater abstractions in connection with dewatering operations. The approach is based on defining the radius of influence R_0 through an iterative process, whereby a radius of influence (which may be affected by boundary conditions) is assumed from the conceptual model, a steady state pumping rate, Q, calculated and steady-state drawdowns estimated based on Q. Depending on the significance of the drawdown, R_0 may be revised and so on.

Q for an unconfined aquifer and hence the drawdown at the feature of interest can be calculated using the Thiem-Dupuit equation:

$$Q = \frac{\pi K(h_2^2 - h_1^2)}{2.3 \log(r_2/r_1)}$$

Where:

h = elevation of water table at radius, r

r_2 can be taken as the radius of influence R_0 and $h_2 = H_0$

r_1 can be taken as the equivalent radius of the excavation and h_1 the target groundwater level inside the excavation.

H_2 and r_2 can also be taken as the water level at a feature of interest located at distance r_2 from the centre of the excavation.

The difficulty in this case is defining R_0, when no clear hydraulic boundaries can be defined (other than perhaps the Forth estuary). Sensitivity analysis demonstrates that this has a significant impact on Q; the greater the radius of influence, the smaller the hydraulic gradient adjacent to the abstraction and the smaller the flow rate.

DJV Analysis

Because of the difficulty in defining R_0 and because the Sichardt formula may underestimate the radius of influence, FCBC/DJV undertook a constant rate pumping test of CSRO10, close to Echline Corner in May 2012, with a view of applying the results to the Thiem-Depuit equation as well as deriving bulk aquifer characteristics (transmissivity, hydraulic conductivity and storage coefficient). Groundwater levels were monitored in two observation boreholes constructed close to the pumping borehole for the purposes of the test and other nearby monitoring boreholes. The test methodology is presented in **Appendix C**, together with the results and analyses.

The Thiem-Depuit equation assumes steady state conditions. Unfortunately inflows to the test borehole were extremely low and it was not possible to maintain a sustainable pumping rate and the test had to be terminated after 3 hours and 10 minutes to prevent damage to the pump, even though the pump intake was located as near to the bottom of the borehole (base of the sandstone unit) as possible. This meant that quasi-steady state conditions could not be achieved. However, the test did demonstrate that the limited thickness and lateral extent of the sandstone units means that dewatering abstraction rates are likely to be significantly less than those predicted from aquifer properties, particularly after the initial phase of pumping.

In view of this and the inherent assumptions behind other analytical equations, which are not satisfied by the conceptual hydrogeological model of this excavation, the Sichardt formula has been used to estimate the radius or zone of influence for the purposes of this assessment. Inflow, Q, has been calculated using the Thiem-Dupuit equation.

The above discussion assumes radial flow to an abstraction point. An equivalent radius, r_e can be calculated to reflect that the cutting is rectangular where:

$$R_e = \sqrt{(a\ b\ /\ \pi)}$$

where:

a and b are the length and width of the cutting respectively.

$R_0 + R_e$ = the total extent of influence from the centre line of the cutting.

3.2.2 Assumptions

The sequence of works will result in the cutting to the north of the existing A904 (approximate chainage 3620 – 4200) being excavated before the cutting south of this.

As discussed in **Chapter 2**, the main aquifer units intersected by the Queensferry cutting and South Launch excavation generally comprise sandstone in the southern section, clayey glacial till and mudstone with some sandstone in the middle section and sandstone with some underlying dolerite in the northern section. Most permeable horizons within the superficial deposits are generally thin and not laterally extensive so are considered to act as perched aquifers with limited storage.

The four areas where properties are considered to be potentially at risk are:

- Echline Corner;
- Springfield (split into 'south' and 'north' at approximate chainage 4060, as the properties span a large section of the excavation);
- Linn Mill; and
- north of Society Road.

In addition, Linn Mill Burn to the west of the excavation has also been considered as a potential receptor.

As discussed in **Section 2.8.3**, ground elevations north of Society Road are at or below the lowest part of the proposed excavation, which means that the excavation can be dewatered without affecting groundwater levels below these properties. Therefore this assessment focuses on Echline Corner, Springfield north and south, Linn Mill, and Linn Mill Burn. The distances from the centre of the cutting to the nearest of the residences are included in **Table 3.2**.

Ground levels in the vicinity of the Echline Corner properties are a few metres below the current ground level along the alignment of the proposed excavation (5.5m and 4m above the lowest part of the proposed excavation on cross-sections A and B). Excavation above these elevations can therefore take place without affecting groundwater levels at Echline Corner. However, to ensure that the worst case scenario is considered, calculations have been carried out assuming a naturalised groundwater level (H_0) both approaching ground level (to reflect groundwater levels seen in boreholes on cross-section A) and at a level approximating to the ground elevation at the nearest properties. Properties at Springfield north/south, Linn Mill and Linn Mill Burn are similarly above the base of the excavation and therefore H_0 has been taken as either the top of the aquifer unit or the maximum measured groundwater level (where available). Where no groundwater levels are available, H_0 has also been considered as the elevation of the receptor, to show worst case scenarios.

Total inflow rate (Q) has been calculated using the sum of Theim-Dupuit calculated inflows for the three main sections, which the total length of cutting has been split into: - Ecline, Springfield south and Springfield north (Section G, K and L).

3.3 Results of DJV Analysis

Calculation spreadsheets are provided in **Appendix B**, together with the key assumptions behind these. The results are summarised in **Tables 3.1 and 3.2** below.

<u>Zone of Influence</u>

Table 3.2 shows that using the Sichardt equation, the radius or zone of influence is less than the distance to the nearest properties at Echline Corner, Springfield and Linn Mill as well as Linn Mill Burn, even when worst case scenarios in terms of water levels, aquifer thicknesses and hydraulic conductivities are considered. This means that dewatering activities should not have any impact on groundwater levels beneath these properties and Linn Mill Burn.

Although the Sichardt equation is acknowledged to underestimate the zone of influence under some conditions, the fact that the zone of influence does not extend to the nearest receptors even under the worst case (and highly unlikely) hydrogeological scenarios suggests that the impact on these receptors is likely to be negligible.

The predicted zone of influence is extremely sensitive to hydraulic conductivity. For example, the findings of the pumping test (**Appendix C**) indicate that the sandstone is heterogeneous in terms of hydraulic properties as well as lithology, and that hydraulic conductivities are likely to be much lower than assumed for the worst case scenario, probably in the order of 10^{-6} – 10^{-7} m/s. Taking the largest predicted zone of influence from the centre line of the excavation under worst case conditions of 180m (Springfield (north), scenario 1: cross-section M), reducing the hydraulic conductivity from 2.5 x 10^{-5} m/s to 7.3 x 10-7 m/s has the effect of halving the predicted zone of influence to 87m.

Similarly, taking into account topographical differences between the existing ground elevation within the footprint of the excavation and at the nearest properties reduces the predicted zone of influence. For example, the zone of influence of dewatering near Echline Corner is predicted to be 158m from the centre line of the excavation under the worst case scenario, i.e. rest water level approaching ground level within the footprint of the excavation (Echline Corner, scenario 2: cross-section B). If the rest water level is reduced to the ground elevation of the Echline Corner properties; i.e. 8m above the base of the excavation, then the predicted zone of influence reduces to 107m. If the pumping test hydraulic conductivity value is used, then this reduces further to 86m.

In addition to this, with the exception of the Echline sandstone unit represented by cross-section A, the sandstone units do not appear to be fully saturated. Indeed, groundwater levels in the sandstone unit that will be intersected by the South Launch excavation may be close to or below the base of the excavation (cross-sections E, F and I). If this is the case, little or no dewatering will be required except for some associated with dewatering more permeable horizons within the superficial deposits.

Drawdown impacts also need to be considered in the context of natural groundwater level fluctuations, which are in the order of 1 – 2m in the sandstone units.

<u>Inflow</u>

Table 3.1 summarises predicted inflows to each section of the excavation as well as the total inflow for best and worst case scenarios.

Although the worst case scenario predicts a total inflow of around 3,000 m^3/day, inflows are likely to be significantly lower than this for the reasons given below.

- The worst case scenario assumes a sandstone/mudstone hydraulic conductivity of 4×10^{-5} m/s based on the CSRO03A falling head test (**Section 2.3**). This is considered to be very high, even if the strata are very fractured, particularly as the constant rate test gave an average bulk hydraulic conductivity value two orders of magnitude lower than this. If this hydraulic conductivity value is excluded, then the predicted worst case total inflow reduces to 1800 m^3/day.

- The Sichardt equation tends to over-estimate inflow (see **Section 3.2.1**).

- The aquifer units intersected by the excavation are relatively thin, which means that as the local water table lowers in response to pumping, the saturated aquifer thickness will decrease and therefore the transmissivity (a measure of the aquifer's ability to transmit water) will reduce significantly.

- The estimated inflow rates assume steady-state conditions occur instantaneously. In reality, these thin aquifer units will be dewatered gradually and in succession as the cutting is deepened. Even within the sandstone/mudstone units, aquifer properties vary with depth, which means that inflow rates will reduce as the more permeable horizons become dewatered. This was seen during the CSRO10 pumping test near Echline Corner (**Appendix C**).

On balance, it is considered likely that abstraction due to dewatering will stay within the CAR licence limit of 1,700 m^3/day +/- 10%. The CAR licence excludes additional abstraction due to rainfall within the excavation.

Observations during the initial phases of construction support this appraisal. These are discussed in **Chapter 4** below.

Table 3.1 – Estimated Inflow

Cutting Section	Chainage	Length of section (m)	Min. estimated inflow (m^3/day)	Max. estimated inflow (m^3/day)
G 'Ecline'	3250 - 3720m	470	23	577 or 1792 (the latter figure if mudstone K of 4 x 10^{-5} used)
K 'Springfield (south)'	3720 - 4060m	340	1	362
L 'Springfield (north)'	4060 - 4250m	190	10	869
		Total Inflow	**34**	**1808 or 3023**

Table 3.2 - Predicted radius of influence and predicted drawdown at receptors

Receptor	Minimum distance from centre of cutting to receptor (m), r_i	Maximum zone of influence ($R_o + r_e$)*	Drawdown at radius r_i (m)
Ecline	180	157.9	0
Springfield (south)	270	114.0	0
Springfield (north)	227	179.8	0
Linn Mill House	137	87.0	0
Linn Mill Burn	186	82.1	0
Society Road	The elevation of the properties is below the base of the cutting and the South Launch temporary excavation, so dewatering will not occur		

* Worst case conditions assumed

See **Appendix B** for the full set of calculations and scenarios tested.

4. SITE OBSERVATIONS

Scottish Power Diversion Duct

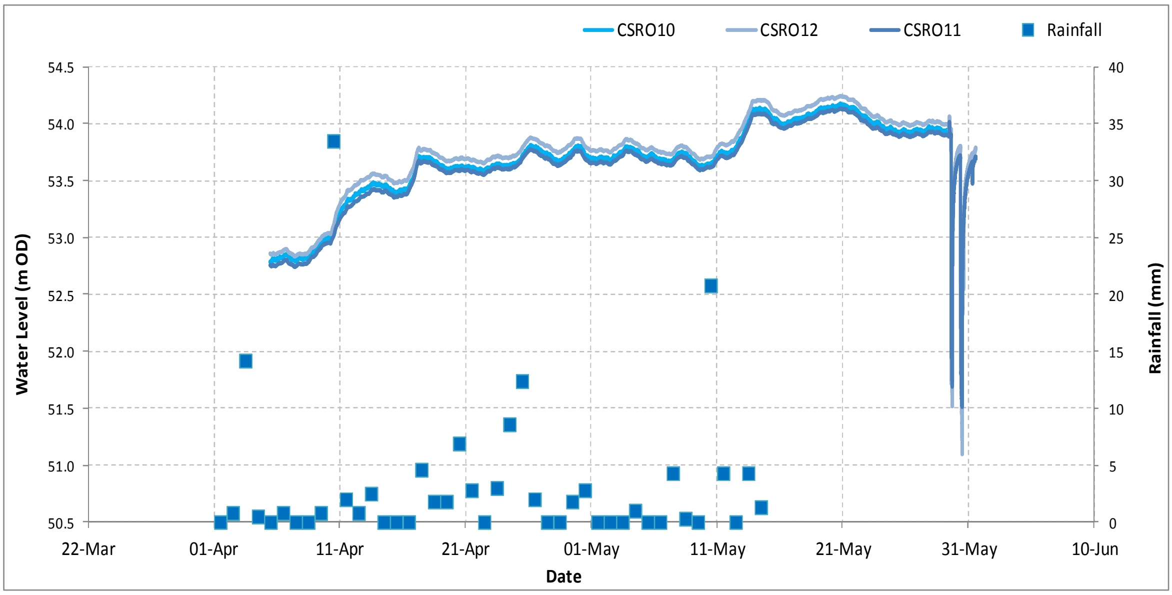
Preliminary excavation works are ongoing. These include those associated with the Scottish Power Diversion Duct, which is located in the central section of the proposed excavation (spanned by long section K, used to assess the impact on properties at Springfield (south)). Although the maximum depth of the diversion trench is 39.64 mAOD i.e. just below the maximum proposed permanent cutting depth (but slightly above the maximum final depth of the South Launch temporary excavation at the same location), the superficial deposits (primarily glacial till) yielded very little water and the small amount of water collecting in the bottom of the trench was thought to largely consist of rainwater. Observations also showed that there was no or very little inflow from bedrock groundwater. This supports the discussion in **Section 3.3** above, in that predicted inflows and associated zone of influence on groundwater levels are likely to be conservative.

Additional Groundwater Level Monitoring

Prior to the pumping test at CSRO10 in May 2012, data loggers were installed in CSRO10, CSRO11 and CSRO12, which recorded groundwater levels in the sandstone unit near Echline Corner at 10 minute intervals. **Figure 4.1** shows that over a period of about six weeks prior to the test, groundwater levels gradually increased by nearly 2m as a result of the high rainfall over this period. Groundwater levels also exhibited a quick response to individual rainfall events. The large changes in water levels at the end of May are as a result of the pumping test.

These observations reflect the responses seen in other boreholes in the area (see hydrographs in Appendix E). It is intended that groundwater monitoring is continued in selected boreholes throughout the construction period to assess the impact of dewatering in comparison to natural variations in groundwater levels.

Figure 4.1 – Groundwater levels and rainfall (10 minute frequency data)



5. CONCLUSIONS

This report updates the findings of the draft Hydrogeological Assessment (FCBC, 2011). It takes into consideration information gained from the latest ground investigation (FCBC, 2012), which included permeability testing and a pumping test, as well as the groundwater level monitoring undertaken to date and observations made during preliminary construction works. The report also takes into account the profile of the South Launch excavation.

The aims of this report are as follows:

- assess the potential impacts of dewatering on properties at Echline Corner, Springfield, Linn Mill and north of Society Road;
- assess the potential impact of dewatering on a nearby watercourse, Linn Mill Burn; and
- determine the likely maximum dewatering abstraction rate.

Previous assessments (JAJV, October 2010 and December 2010) considered that there may be a limited impact on groundwater levels at these locations, particularly Echline Corner. However, a review of the JAJV model assumptions suggests that they may be overly conservative and that the groundwater level impacts may be overestimated.

Hydraulic conductivity data derived from previous ground investigations included some values which were substantially higher than expected for the aquifer unit under test. Values derived from hydraulic conductivity testing of new boreholes constructed by FCBC generally fell within the range of published values, although some were again higher than expected. However. a subsequent pumping test conducted in a borehole within the footprint of the proposed excavation near Echline Corner gave a hydraulic conductivity value at the low end of the range derived from falling head and packer testing, which is likely to be more representative of aquifer conditions.

A review of industry-standard and regulatory approaches to estimating inflows to an excavation and the impact of dewatering was conducted, which included consideration of the pros and cons of each approach.

In view of the level of applicability and/or uncertainty associated with the methods, particularly when considered in the context of the conceptual hydrogeological model and excavation proposals, a pumping test was undertaken with a view of applying the results to the Thiem-Depuit equation for steady state flow as well as deriving bulk aquifer parameters. However, a very low pumping rate could not be sustained due to excessive drawdown and the test had to be terminated before steady state conditions could be achieved.

In light of this and the review of dewatering assessment methods, the Sichardt formula was used to estimate the radius or zone of influence for the purposes of this assessment, acknowledging that this can underestimate the zone of influence but overestimate inflows under certain conditions. Inflows were calculated using the Thiem-Dupuit equation.

The zone of influence arising from the South Queensferry cutting and temporary South Launch excavation was estimated to be less than the distance to the nearest properties at Echline Corner, Springfield and Linn Mill as well as Linn Mill Burn, even under the worst case hydrogeological scenarios. This means that dewatering activities are likely to have a negligible impact on all receptors. Even if application of the Sichardt formula results in the zone of influence being underestimated, this is likely to be more than compensated for by the application of the most realistic hydrogeological scenario (particularly with respect to hydraulic conductivity and actual topographic elevations) rather than the worst case.

Ground elevations in the area of housing north of Society Road are below the lowest level of the proposed excavations, so dewatering activities should not impact on groundwater levels in these areas.

Drawdown impacts also need to be considered in the context of natural groundwater level fluctuations, which are in the order of 1 – 2m in the more permeable aquifer units.

Although the worst case scenario predicts a total inflow of 3,000 m^3/day to the excavation, inflows are likely to be significantly lower than this for a number of reasons and it is likely that abstraction for dewatering purposes will remain within the existing CAR licence limit of 1,700 m^3/day +/- 10%. The CAR licence excludes the abstraction of water that has accumulated in the excavation as a result of direct rainfall.

The likelihood of inflows being significantly less than those predicted is supported both by the results of the pumping test and by observations during excavations for preliminary construction works.

6. RECOMMENDATIONS

Groundwater level monitoring should continue in selected boreholes throughout the construction period to allow potential impacts on local groundwater levels, if any, to be assessed.

The volume of water abstracted from the excavation should be measured on a daily basis, both to comply with the CAR licence and to allow comparison with the findings of this assessment. Concurrent with this, rainfall should be measured on a daily basis so that the contribution of direct rainfall to the excavation can be excluded from the daily volume abstracted under the CAR licence.

7. REFERENCES

Bear, J. (1979) Hydraulics of Groundwater.

CIRIA (2000) Groundwater control – design and practice CIRIA 515

DJV (2012) FRC-O-____EL-000-R-NT-ENV-30601 Groundwater monitoring Report, February – May 2012

EA (2007) Hydrogeological impact appraisal for dewatering abstractions. Environment Agency (EA) science report SC040020/SR1

FCBC (2011) FRC-P-_____E-099-R-NT-EAR-06001-04 Draft Hydrogeological Assessment, October 2011

FCBC (2012) FRC-P-FCBC-REP-00000-NTRS-GENROD-CAN-00019 FRC Confirmatory GI (Land Phase): Southern Network Connections and Main Crossing.

Gidahatari (2011) Overview of the "Radius of Influence".
(http://www. gidahatari.com/IS_overview_Radius_of_influence)

JAJV (2009) Forth Replacement Crossing – Network Connections South Ground Investigation Report

JAJV (October 2010). Forth Replacement Crossing – Queensferry Cutting Groundwater Assessment CAR Licence Process Supporting Technical Document

JAJV (December 2010). Forth Replacement Crossing – Network Connections - South Queensferry Cutting.

JAJV (2010) Network Connections South Land Contaminated Information Report

Kruseman, G. P. and deRidder, N. A. (1994) Analysis and Evaluation of Pumping Test Data (2nd Edition). ILRI publication 47

Niccoli W, Marinelli F, Fairbanks T and Dancause R (1998). Latin hypercube sampling: application to pit lake hydrologic modeling study. Proceedings of the 1998 Conference on Hazardous Waste Research

Ritchies (2008) Forth Replacement Crossing – Ground Investigation South of Forth Estuary

Ritchies (2009) FRC Network Connections – South Additional Ground Investigation (Ritchies, 2009)

Ritchies (2010) Forth Replacement Crossing Network Connections South – Dialogue Period Ground Investigation ref. 3189C, July 2010)

SEPA (2007) Regulatory Method (WAT-RM-11) Abstraction from Groundwater. V1.1. June 2007

Sichardt, W. and Kyrieleis, W., (1930) Grundwasserabsenkungen bei Fundierungsarbeiten, Berlin.

APPENDIX A

HYDROGEOLOGICAL CROSS SECTIONS

APPENDIX B

ZONE OF INFLUENCE AND INFLOW CALCULATIONS

APPENDIX C

CSRO10 PUMPING TEST

APPENDIX D

FALLING HEAD TEST ANALYSIS

APPENDIX E

HYDROGRAPHS