Appendix 16.3
Groundwater Assessment Report

GROUNDWATER SPECIALISTS

Junction 5, M74, Raith

Groundwater Assessment for

Construction of Underpass

March 2007

Mouchel Fairhurst JV

Revision 15

J05/267/140R

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Executive Summary

This groundwater assessment is based on a comprehensive programme of ground and groundwater investigation, which included rigorous field testing to establish the groundwater behaviour in the vicinity of the proposed underpass at Junction 5 of the M74.

This report documents the analysis and interpretation of the groundwater investigation, which included a series of aquifer pumping tests within the various hudrogeological layers encountered during the ground investigation.

The main purpose of the analysis and interpretation in this report has been to:

- Determine if it is technically feasible and practical to lower the groundwater to a sufficient level to allow a safe and stable excavation for the construction of the underpass walls.
- Determine the range of expected groundwater abstraction rates required to lower the groundwater levels and artesian pressures to achieve a safe and stable excavation.
- Predict the consequential drawdown of the water table and artesian pressure outside the excavation, in order to assess the potential impact on surface waters and to enable the impact of the groundwater lowering on adjacent property to be calculated by geotechnical engineers.

The findings of the groundwater assessment concluded the following:

(i) The ground conditions encountered during the ground investigation can be simplified as upper and lower sand/gravel horizons separated in places by horizons of clay or silt, overlaying rock of the upper coal measures. The bedrock consists of horizons of sandstone, mudstone and siltstone.

The rock head is at a higher level in the northeast of the site than in the southwest. The dominant aquifer in the northeast is in the rock and the dominant aquifer in the southeast is in the lower sand/gravel horizons.

(ii) Because of the distinctly different ground conditions across the site, differing dewatering techniques are suggested for the northeast and southwest sections of the underpass.

(iii) The groundwater lowering requirement in the southwest can be achieved by pumping from the lower sand/gravel horizon and may be assisted by installing sacrificial, slurry cross-walls and/or by extending the underpass walls to rock head.

(iv) Artesian conditions in the rock and lower sand/gravel horizons, as observed in the northeast of the site, may be dealt with most effectively by limited pumping from the rock.

(v) To ensure stability of the silt/clay horizons above the rock, pumping from the rock can be supplemented by slender vertical pressure relief wells installed through the soft clay layer to dissipate the uplift pressures on the underside of the clay and so reduce the risk of heave of the excavation base.

(vi) In the permanent condition the slender pressure relief drainage wells could be linked to a drainage blanket beneath the slab and thus cater for any uncertainties in long-term groundwater behaviour.

(vii) In all the above cases, an observational approach to the construction and installation of the dewatering/depressurisation wells is advocated. This would allow the groundwater information gathered during the construction and installation of the dewatering/depressurisation wells to be used to optimise the final system installed.

(viii) This report demonstrates that the resulting cone of depression from the dewatering operation means that the drawdown is greatest near to the pumping wells (i.e. to the road infrastructure around Junction 5), but somewhat less at the distance of the closest property.

(ix) Based on the predicted drawdown contours as modelled in this report, the resulting settlements are estimated in the Geotechnical Interpretative Report complied by Mouchel Fairhurst Joint Venture (MFJV).

(x) Conventional measures of mitigating the effects of drawdown on ground settlement have been suggested. These measures have the ability to ameliorate the ambient water table in the upper gravel horizons beneath buildings where there may be an unacceptable groundwater drawdown.

(xi) The hydrogeological conditions of the site can be described as a dynamic system fed by a substantial catchment area. As a consequence the effects of dewatering are expected to be short-term and completely recoverable, with the risk of any detrimental permanent impact on the groundwater regime itself being considered to be insignificant.

(xii) This assessment has concluded that risks to the surface water environment are extremely low given the manner in which the surface water bodies are fed, and because proposals are being made for returning abstracted water to the ponds to ensure a balance is maintained. Clearly these proposals are dependent on an acceptable quality of returning water and this aspect is addressed in the groundwater chemistry section of the Interpretative Contamination Report compiled by (MFJV).

Based on the above findings, the main conclusions can be summarised as;

1. It is considered both technically feasible and practical to lower the groundwater levels to a sufficient level to allow the construction of a safe and stable excavation base within the underpass walls.

2. The computer modelling undertaken as part of this assessment has determined that whilst the required groundwater abstraction is significant, it is within the abstraction rate considered practical for a project of this kind.

3. Whilst it is considered that a practical dewatering scheme can be designed and implemented to effect safe and reliable construction, it is emphasised that success is fundamentally dependent upon the experience and capability of the construction team. Careful selection of the contractor is paramount.

   As such, it is considered that the scope of investigations, and the quality of the findings are sufficient to enable competent D&B construction contractors to design a practical dewatering scheme which will allow the envisaged construction methods to be employed.

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1. Introduction

1.0.0. Transport Scotland requires the upgrade of Junction 5 of the M74 at Bothwell (which is known as the 'Raith Interchange') at the location shown in Figure 1.1. The upgrade will require the A725 to be re-routed through an underpass underneath the M74.

1.0.1. The construction of the underpass will be to a lowest finished road level of 15.390m OD (metres above Ordnance Datum), with deeper open excavation required during the construction period to an approximate depth of 14m OD.

1.0.2. High groundwater levels were observed during the start of the current ground investigation conducted by Mouchel Fairhurst Joint Venture (MFJV). In several of the Ground Investigation boreholes high artesian groundwater pressure was observed.

1.0.3. OGI was commissioned by MFJV to provide advice and direction relating to the impact of the existing high artesian groundwater pressure on the construction and post construction conditions. In consultation with MFJV, OGI identified the critical parameters for which the ground investigation and pumping testing were required.

1.0.4. The drilling of the boreholes, installation of the wells and the pump tests was undertaken by Raeburn Drilling and Geotechnical Ltd (Raeburn). OGI's report is based on the ground information compiled during the drilling of the boreholes and the data from the pump tests. The borehole logs which represent the ground conditions encountered during the drilling of the boreholes were provide to OGI in the draft version of Raeburn Factual Report issued on 26 May 2006.

1.0.5. OGI's report presents the results and interpretation of the Groundwater Investigation which was conducted between November 2005 and July 2006.

1.0.6. The pump test data on which this report was compiled was provided by MFJV as each pump test was completed. Tables No. A14.1 presented in Appendix No. 14 is a summary of the pump test data that OGI received. The tables show the data that was used to compile this report together with the data that were not used because the drawdown response was too small for an accurate analysis.

1.0.7. The pump test data that was used to assess the hydrogeological conditions of the ground, around the proposed location of the underpass, were from the pump tests that were conducted between 2 February and 5 June 2006.

2. Site Description

2.0.0. The site is located at the junction of the M74 and A725 (Whistleberry Road/Bellshill Road) approximately 1km west of Bothwell. Junction 5 is known as Raith Junction. Figure 2.1 shows the proposed location of the underpass and the surrounding area.

2.0.1. The site covers an area of approximately 170 hectares on the flood plain of the River Clyde and lies at a general level elevation of 22m OD.

2.0.2. The area to the south of the Raith Junction is designated as a Site of Special Scientific Interest (SSSI), in which is located a wildlife pond as illustrated in Figure 2.1.

2.0.3. The proposed underpass is approximately 625m long and 35m wide. The deepest finished road level in the underpass is at a level of 15.39m OD as shown in MFJV drawing, RAITHMFJV/ST3/S/1015.

2.0.4. MFJV has specified that a further 1.5m excavation below road level is required to construct the base slab and blinding layers. This results in a total excavation depth to a level of 13.89m OD.

2.0.5. The walls of the underpass road box are to be constructed using a Secant pile or Diaphragm wall extending to a greater depth, but at the present time the exact depth is uncertain.

2.0.6. A plan view of the site and the proposed route of the underpass are illustrated in Figure 2.2.

3. Geology and Ground Conditions

3.0.0. The geology of the area around Junction 5 of the M74 is dominated by Carboniferous Coal Measures. These consist of cyclic sequences of sandstone, mudstone, seat earths and coals, laid down in a fluvio-deltaic environment. The Coal Measures Group contain three formations, namely, the Lower Coal Measures, the Middle Coal Measures and the Upper Coal Measures.

3.0.1. Junction 5 of the M74 is underlain by the Upper Coal Measures. The Upper Coal Measures are present in a broad irregular syncline, with the predominant axis aligned approximately west northwest/east southeast with Junction 5 being positioned on the northeast limb. The Upper Coal Measures are also referred to as the upper red barren measures, and are recorded on the BGS one inch series sheet 31 as “red sandstones with purple and mottled clays and blaes (mudstones) and rare thin limestones, with occasional thin coals in the lower half”.

3.0.2. The Upper Coal Measures were not mined for economic reasons, and mining operations in the region focused on the Middle Coal Measures. The county series geological map of the region (Ref. Geological Survey of Scotland) records several collieries in the area. The Hamilton Palace colliery was located approximately half a mile to the east, and the Bothwell Castle colliery pits 3 and 4 approximately a mile and a half to the west, of Junction 5 of the M74.

3.0.3. Sandstones from the Upper Coal Measures formation outcrop to the east of Junction 5 in a tributary of the River Clyde, and in old quarries about a mile to the northeast.

3.0.4. Overlying Quaternary deposits in the region of Junction 5 comprise glacial deposits of sands and gravels, laminated clay and glacial till, overlain by alluvial river terrace deposits of the River Clyde. At the Hamilton Palace No. 1 Pit an 18.3 metres sequence, from surface, is recorded as soil, clay, sand and gravel, mud, sand and gravel and clay and stones. The Coal Measures on the southwestern limb of the syncline are predominantly covered by subsequent marine deposition and later by glacial till.

3.0.5. Two faults are shown on the 1:50,000 series map Scotland 31 W (Ref: British Geological Survey 1992), striking northwest to southeast. The north easterly fault appears to run directly through Junction 5. These faults are not shown at surface on the county series map (Ref: Geological Survey of Scotland). However, a fault is shown with the same alignment in the Main and Pyotshaw coals with a down throw of 47 fathoms (85.95 metres) to the northwest. Two further faults are recorded in the Ell coal to the south of the site and have been projected to surface. They diverge from a single point to the southeast of the site and strike approximately east/west. They downthrow to the north; the more southerly of the two having a downthrow of 50 fathoms (91.44 metres).

3.0.6. The local site geology and ground conditions, as required to assess dewatering feasibility, were determined from boreholes drilled as part of the ground investigation conducted by Raeburn under the supervision of MFJV. The drilling and installation work was conducted between August 2005 and July 2006.

3.0.7. Over 90 boreholes were drilled as part of the current ground investigation, plus several boreholes that were abandoned due to ground problems that were encountered. Of the boreholes that were drilled the majority had monitoring or pumping well casing installed within the borehole. The Raeburn Factual Report, shows the detail of the ground encountered and the installations for each borehole.

3.0.8. The boreholes were drilled by cable percussion, rotary percussion or coring drilling method. Selected boreholes were installed with varying size stand pipes or well casing. The nature and size of the casing installed into each borehole was dependant on the proposed use of the borehole and also the groundwater conditions encountered. In particular, the boreholes and well casings needed to be of sufficient diameter to allow suitable pumps to be installed, and that the annulus of the boreholes can be adequately sealed with bentonite.

3.0.9. On completion of the borehole installation the borehole was either used as a pumping well for conducting a pumping test or as a monitoring well to record groundwater changes during a pumping test.

3.0.10. Across the site, made ground was encountered which ranged in thickness from approximately 1m to approximately 13m. The thicker deposits are adjacent to and within the embankments of the M74 and A725. The made ground comprises mainly clay, sandy gravelly clay, sand and gravel.

3.1 Local Geology

3.1.0. Due to the numerous differing lithological descriptions encountered during the ground investigation, they have been renamed into geotechnical groupings by MFJV for design purposes. MFJV report and drawing (Fairhurst's drawing reference 53213/002) show in detail each lithology. This section summarises each geotechnical grouping taken from the MFJV report. An idealised conceptual model of the geology and ground conditions encountered can be seen in Figure 3.1.

A detailed record of the geology and ground conditions encountered during the ground investigation can be seen in the MFJV, M74 Junction 5, Raith Interchange Geotechnical Interpretative Report on Construction of Underpass, Project No. M8MFJV/ Revision 2.

3.1.1. <u>Upper Sand and Gravel</u>. A relatively thin horizon which is discontinuous across the site and comprises sand, gravel and silty sand which varies in thickness from 1.0m to 5.8m. Within the sand and gravel layers many discrete cohesive layers were observed.

3.1.2. Marine and Glaciolacustrine Silt/Clay. The horizon extends across the site with the thickness varying considerably from approximately 1m to up to 10.8m thick, with the silt being described as sandy silt.

3.1.3. Lower Sand and Gravel. This horizon is the most extensive and thickest stratum at the southwest end of the site. This stratum appears to reduce at the northeast end of the site as the rock head appears to rise. Discrete isolated layers of clay and cohesive lenses occur within the horizon but these are not common.

3.1.4. Glacial Till. The Glacial Till horizons extend across the majority of the route of the proposed underpass with it being up to 6.5m thick at the northeast end but reducing to a very thin horizon at the southwest end. Thick isolated bands of sand and gravel are common throughout the glacial till. The glacial till can be distinguished by its reddish brown colour and is typically stiff to very stiff sandy gravelly Clay.

3.1.5. Rock. The underlying rock is the Carboniferous Coal Measures sequence. These consist of cyclic sequences of sandstone, siltstone, and mudstone. The siltstone and mudstone are typically very weak to weak, and the sandstone moderately strong to very strong. The core recovery of the rock during the drilling process indicates that locally the rock may be highly fractured and may be locally fractured to an unknown depth.

4. Hydrogeology

4.0.0. Groundwater flow within the Upper Coal Measures is likely to be restricted to the sandstone units. The seat earths and mudstones, as well as the overlying glacial till, act as aquitards or aquicludes. Robins (Ref: Robins 1990) reports the mean permeability of the sandstones within the Coal Measures as 10^{-2} m/d, indicating the total hydraulic conductivity of the sequence is not high, except where fractures and joints promote secondary permeability.

4.0.1. It is probable that the groundwater conditions beneath the site are controlled by a combination of the topography and the structural geology.

4.0.2. Sandstone beds crop out to the east of the site, and are at a higher topographic level than Junction 5. Local groundwater recharge is most likely to occur in this area.

4.0.3. As these beds dip to the west beneath the site, they become confined by the overlying mudstones and glacial till (clay), producing a confined aquifer so giving rise to a piezometric surface that lies above ground level in areas of lower topography.

4.0.4. Mixed intergranular and fracture flow within sandstone units will be in a south-westerly direction towards the River Clyde. However, locally groundwater flow in the rock is likely to be dominated by fracture flow.

4.0.5. Towards the south west of the area, the groundwater flow will be dominated by the flow within the upper and lower gravel strata. These deposits comprise sand and gravel layers with varying interbedded layers of silt and clay.

4.0.6. Because the proposed underpass excavation is within the alluvial/fluvial deposits, it is necessary to understand the local hydrogeological characteristics at the Junction 5 site.

4.0.7. As discussed, Junction 5 is located on the north easterly limb of a broad syncline with groundwater flowing towards the River Clyde. Under these conditions, artesian conditions can result when the fractured or weathered rock dips beneath a low permeable clay or other low permeable superficial deposit.

4.0.8. Of particular significance is the fact that the underpass requires excavation into the lower sands and gravels. If the deposits have a high silt content, then the inflow of groundwater flow to the excavation may be low. However if the sands and gravels are clean, the inflow of groundwater to the open excavation can be substantial and require a groundwater management system to maintain low groundwater levels during construction. As can be seen in the PSD's in the Raeburn report the silt content in the sands and gravel does vary but generally the silt content in the sand and gravel horizons is low or none was recorded.

4.0.9. If the threat resulting from the presence of high groundwater levels, pressures and flows is not addressed, this will undoubtedly result in hazards during construction. For this reason it was considered essential that a comprehensive ground and groundwater investigation was undertaken to assess the groundwater characteristics surrounding the Junction 5 underpass construction.

4.1 Investigation Methodology

4.1.0. Initial investigation boreholes indicated the presence of a high water table surrounding Junction 5. Furthermore when drilling at the northeast side of the site, high flowing artesian pressures were encountered when the drilling advanced close to the underlying rock. These artesian pressures need to be addressed to avoid potential ground instability during the excavation and construction period.

4.1.1. To investigate the groundwater regime in the area, the groundwater investigation strategy included the drilling of sampling boreholes in the various drift deposits, together with drilling into the rock (i) to prove depth and (ii) to measure the piezometric head at this depth.

4.1.2. To assess the magnitude of flow through the ground to the open excavation during construction, the investigation boreholes were completed with the installation of piezometer casings or pumping well casings. This enabled aquifer pumping tests to be conducted from the wells, with the observed groundwater drawdown measured in the surrounding piezometers.

4.1.3. A well casing or a stand pipe was installed into selected boreholes to either allow a pump to be installed down the borehole to conduct a pumping test or to allow the borehole to be used as an observation well. The purpose of this installation was to record groundwater levels prior to, during and after a pumping test was conducted.

4.1.4. The well casing or stand pipe was installed so that the response zone (the slotted section of the well casing or stand pipe) was located adjacent to principle strata in which the testing was to be conducted. Tables No. 1 to No. 11 in section 5.3 indicate the depth at which the slotted section of the well casing was installed along with the use of the borehole. Figure 4.1 shows the location of the boreholes used for the pumping tests and to observe groundwater levels.

4.1.5. Groundwater levels were measured by transducers installed in observation wells. A number of these transducers were installed to measure background ambient groundwater levels, with higher frequency measurements monitored by transducers that were installed into specified observation wells to record groundwater level changes during a specific pumping test. These transducers were removed on completion of the pumping test and reinstalled into the next set of observation wells for the next pumping test. Hand dip measurements were also taken for the purpose of transducer calibration together with data backup in case of unforeseen transducer failure.

4.2 Groundwater Observations

4.2.0. Generally the groundwater data recorded in the boreholes to the northeast of the site demonstrate that the rock aquifer is under artesian conditions with the static state level of the piezometric surface above the existing ground level in that area. The groundwater levels recorded in the boreholes to the southwest of the site were consistently lower than at the northeast end of the site with the observed piezometric head in the southwest being high, but lower than the ground surface level.

4.2.1. The piezometers installed also indicate differences in piezometric levels at various depths into the ground strata. The recorded piezometric levels at the southeast end of the site indicated that the water level in rock, lower gravel and upper gravel are all close to the same average level of approximately 19.0m OD to 19.6m OD.

4.2.2. At the northeast end of the site the recorded piezometric level indicated that the rock and lower gravel have an average piezometric level of approximately 24.8m OD. This results in the average piezometric level in the rock and lower gravel at the northeast end of the site being approximately 1.9m higher than the average recorded level in the upper gravel.

4.2.3. From the observed data it is clear that there is a gradient in the piezometric surface falling from the northeast towards the River Clyde in the south west. This is illustrated in Figure 4.2 which presents an estimated contour plot of the piezometric surface. This was based on piezometric level observations taken during the drilling of the boreholes together with groundwater levels recorded prior to the pumping test being conducted.

4.2.4. The fault that runs in a northwest-southeast direction (as identified in the MFJV report) in the vicinity of the underpass may explain why there is a steeper hydraulic gradient which coincides with the M74 Carriageway. The impact of the fault causing a barrier of lower transmissivity has not been considered in this report as it is proposed to dewater both sides of the fault.

4.2.5. An alternative explanation for the increase in hydraulic gradient could be because the aquifer system changes in transmissivity over this same area. If the transmissivity of the ground to the northeast has a lower transmissivity than the sand & gravel aquifer to the southwest, then a change in the hydraulic gradient would be expected.

4.2.6. In either case, i.e. a change in transmissivity of the aquifers, or the presence of a fault which acts as a barrier to flow, a carefully designed dewatering system can overcome groundwater problems present.

4.2.7. The most likely explanation for high artesian head in the northeast of the site is likely to be caused by the presence of lower permeability ground overlying the fractured rock.

4.2.8. The artesian head in the rock (circa 3 – 4m above ground level) to the ground surface (this being at atmospheric pressure), will result in vertical upward gradient. The head distribution in the vertical direction is unlikely to be exactly linear because of the ground stratification. However, the head distribution will be from an elevated head in the rock (approximately 25m OD), to the head at the ground surface (approximately 22m OD).

5. Analysis & Interpretation of Field Pumping Tests

5.0.0. The main objective from the testing programme is to assess the range in transmissivity. It is this parameter that predominantly governs the pumping rate required to achieve a particular drawdown.

5.0.1. Transmissivity and not permeability has been used to determine the groundwater flows to the construction dewatering pumping system. Transmissivity is also used to simulate the wider lowering of the water table, or the reduction of artesian pressures resulting from the dewatering process.

5.0.2. The implications of using a transmissivity value derived from a limited test length have also been considered when applied to the whole "system", with the resulting analysis considered to be robust.

5.0.3. Two techniques were used to determine the transmissivity from the field pumping test data. These techniques used were (i) a standard analytical method, and (ii) curve matching with a numerical model.

5.0.4. The first technique to derive transmissivity is the Jacob method for single well pumping in a confined aquifer. This method is described in detail in Kruseman and de Ridder, 1990. This is a curve fitting technique to establish the properties of the gravel and bedrock aquifers.

5.0.5. This method is simple and very practical because whilst it does calculate transmissivity accurately, the straight line produced on a semi-log scale can be used to identify where there are discrepancies with the field results.

5.0.6. To derive the Storage Coefficient, S, from the Raith test results a second technique is use. Here, OGI has used a combination of using the Jacob method to calculate the value of transmissivity, followed by a numerical method to calculate S. This technique results in an accurate analysis of both transmissivity and storage coefficient.

5.1 General methodology

5.1.0. Prior to the commencement of each pumping test, OGI advised on the specific observation wells in which changing piezometric level is to be recorded. In some wells such water level recording was conducted with automatic electronic transducer data.

5.1.1. The observation wells (sometimes called piezometers) were selected on the basis of two main criteria:

(i) their location/distance relative to the pumping well; and,
(ii) the depth of the observation well screen relative to that in the pumping well.

Observation wells were selected to monitor the effect of the test pumping in the upper and lower parts of the sand & gravel aquifer and in the underlying rock aquifer.

5.1.2. Prior to a full pumping test being conducted, a trial pumping test was conducted in the well to calibrate the pumps and to assess the flow that could be expected from the well. Once the trial test was conducted, pumping was undertaken based on the flows that were achieved in the trial. The data that were recorded in the trial pumping test were also analysed when a sufficient magnitude of response was achieved.

5.1.3. For each pumping test, the water levels in all relevant observation boreholes were recorded at the start of the test. Water levels were recorded during the test in the observation wells using a combination of pressure transducer readings.

5.1.4. Prior to the analysis of each set of drawdown or recovery readings, the readings were visually assessed to determine if analysis was feasible. Appendix 14 presents an overview of the pumping and observation wells for which analysis was conducted, and those observation wells where monitoring was conducted but the drawdown response was not of sufficient magnitude for accurate analysis.

5.1.5. Each relevant pumping test has been analysed separately using the Jacob method and reported individually in the relevant Appendices. An overall report of the pumping test results is given in Section 5.3.

5.1.6. To evaluate the storage coefficient, S, a numerical model was used to simulate the pumping test. The transmissivity used was that derived from the Jacob analysis, with the storage coefficient, S, derived from adjusting S to establish a best match of the field data.

5.2 Pumping test analysis

5.2.0. The pumping test data were analysed using The Jacob Drawdown Method as described in Kruseman and de Ridder (1990). Jacob Method requires the observed drawdown (*s*) from initial water level to be plotted against time since the commencement of pumping, with time plotted on a log scale ($\log_{10}t$).

5.2.1. Pumping test data comprise the measurement of drawdown versus time as illustrated in Figure 5.1. Jacob method of analysis requires first plotting the same data on a log scale as shown in Figure 5.2.

5.2.2. The next stage is the fitting of a straight line over the linear section of the Δs vs. $\log_{10}t$ plot as shown in Figure 5.3.

5.2.3. The transmissivity can then be derived from the Jacob equation:-

$$T = kb = \frac{2.3Q}{4\pi\Delta s}$$

where the transmissivity, T, is the product of the effective aquifer thickness, b, and the average hydraulic conductivity, k.

5.2.4. Q is the pumping rate (m^3/sec); and Δs is the calculated drawdown over one $\log_{10}$ cycle in metres. The hydraulic conductivity can be estimated from transmissivity if the saturated aquifer thickness is known. Note that ‘hydraulic conductivity’ is also known by the term ‘permeability’ in the United Kingdom. Both have dimensions [L/T] normally defined in units of meters/second (m/s) and metres/day(m/d).

5.3 Pumping tests results and interpretation

5.3.0. This section is a summary of the pumping tests conducted, the well construction and the results of the pumping tests. The OD level in metres of the ground level at the borehole location, and the location of the well screen was taken from the borehole logs in the Raeburn Factual Report. The distances between the pumping wells and the observation wells were calculated from borehole easting and northing coordinates provided by MFJV. The location of the screen in each well was taken from the borehole logs.

All data comes from the Phase 4 Ground Investigation, but note that the prefix "4" has been dropped from the borehole number system.

5.3.1. Pumping test BH 209 (1^{st} test)

Table 1 - Summary of well construction for BH 209 pumping test

Borehole	Type	Ground Level (m OD)	Distance from Pumping Well (m)	Screen (m OD)
BH 209	Pumping from lower gravel	21.22	-	4.72 to 9.72
BH 207	Observation in upper/lower gravel	19.68	36.92	3.68 to 9.68
BH 207A	Observation in upper gravel	19.69	36.92	15.69 to 18.19
BH 208	Observation in upper gravel	21.25	7.40	6.75 to 15.45
BH 217	Observation in lower gravel	21.48	88.72	-0.72 to 5.08

Table 1A - Summary of BH 209 pumping test analyses

Start Date	2nd February 2006	
Finish Date	2nd February 2006	
Duration of test	2.5 Hours	
Type of data analysed	Transducer	
Pumping Rate	1.05 litres/second	
Observation well	**$\Delta s/\log_{10}t$ (m)**	**Transmissivity (m^2/day)**
Drawdown data		
BH 207	0.0392	423.6 (UG/LG)
BH 207A	0.0435	381.7 (UG)
BH 208	0.0710	233.9 (UG)
BH 217	0.0775	214.2 (LG)
Recovery Data		
BH 207	0.052	319.3 (UG/LG)

The drawdown and recovery graphs used to calculate the transmissivity can be seen in Appendix 1. Pumping and observation well locations are depicted in Figure 5.4.

Note that borehole BH214 was also monitored with a pressure transducer during this pumping test. However, the drawdown response was small in this borehole and so was not able to be accurately analysed.

5.3.2. Pumping test BH 209 (2nd test)

Table 2 - Summary of well construction for BH 209 pumping tests

Borehole	Type	Ground Level (m OD)	Distance from Pumping Well (m)	Screen (m OD)
BH 209	Pumping from lower gravel	21.22	-	4.72 to 9.72
BH 207	Observation in upper/lower gravel	19.68	36.92	3.68 to 9.68
BH 207A	Observation in upper gravel	19.69	36.92	15.69 to 18.19
BH 208	Observation in upper gravel	21.25	7.40	6.75 to 15.45
BH 217	Observation in lower gravel	21.48	88.72	-0.72 to 5.08

Table 2A - Summary of 209 BH pumping test analyses

Start Date	6th February 2006	
Finish Date	6th February 2006	
Duration of test	8 Hours	
Type of data analysed	Transducer	
Pumping Rate	0.25 litres/second	
Observation well	**$\Delta s/\log_{10}t$ (m)**	**Transmissivity (m^2/day)**
BH 207	0.114	34.68 (UG/LG)
BH 207A	0.205	19.28 (UG)
BH 208	0.197	20.07 (UG)
BH 217	0.112	35.30 (LG)

The drawdown and recovery graphs used to calculate the transmissivity can be seen in Appendix 1. Pumping and observation well locations are depicted in Figure 5.4.

Note that borehole BH214 was also monitored with a pressure transducer during this pumping test. However, the drawdown response was small in this borehole and so was not able to be accurately analysed.

5.3.3. Pumping Test BH 217 (1st test)

Table 3 - Summary of well construction for BH 217 pumping tests

Borehole number	Use of Borehole	Ground Level (m OD)	Distance from Pumping Well (m)	Screen (m OD)
BH 217	Pumping from lower gravel	21.48	-	-0.72 to 5.08
BH 208	Observation in upper gravel	21.25	96.12	6.75 to 15.45
BH 209	Observation in lower gravel	21.22	88.72	4.72 to 9.72
BH 216	Observation in upper gravel	21.33	8.68	12.33 to 15.58
BH 218B	Observation in lower gravel	21.48	5.65	1.98 to 4.98
BH 225	Observation in upper gravel	28.73	124.9	18.73to 23.73
BH 265	Observation in rock	20.53	20.12	-8.47 to -0.47

Table 3A - Summary of 217 BH pumping test analyses

Start Date	14th February 2006	
Finish Date	14th February 2006	
Duration of test	8 Hours	
Type of data analysed	Transducer	
Pumping Rate	1.09 litres/second	
Observation well	**$\Delta log_{10}t$ (m)**	**Transmissivity (m^2/day)**
BH 208	0.227	75.9 (UG)
BH 209	0.248	69.5 (LG)
BH 216	0.0655	263.2 (UG)
BH 218B	0.186	92.7 (LG)
BH 225	0.617	27.9 (UG)
BH 265	0.168	102.6 (Rock)

As a consequence of BH 216 being in close proximity to BH 217 the drilling will impact on the ground between the pumping and observation well, making this a higher than natural transmissivity.

The drawdown and recovery graphs used to calculate the transmissivity can be seen in Appendix 2. Pumping and observation well locations are depicted in Figure 5.5.

5.3.4. Pumping Test BH 217 (2nd test)

Table 4 - Summary of well construction for BH 217 pumping tests

Borehole number	Use of Borehole	Ground Level (m OD)	Distance from Pumping Well (m)	Screen (m OD)
BH 217	Pumping from lower gravel	21.48	-	-0.72 to 5.08
BH 208	Observation in upper gravel	21.25	96.12	6.75 to 15.45
BH 209	Observation in lower gravel	21.22	88.72	4.72 to 9.72
BH 216	Observation in upper gravel	21.33	8.68	12.33 to 15.58
BH 218B	Observation in lower gravel	21.48	5.65	1.98 to 4.98
BH 265	Observation in rock	20.53	20.12	-8.47 to -0.47
BH 225	Observation in upper gravel	28.73	124.94	18.73 to 23.73

Table 4A - Summary of 217 BH pumping test analyses

Start Date	23rd February 2006	
Finish Date	24th February 2006	
Duration of test	24 Hours	
Type of data analysed	Transducer	
Pumping Rate	5.6 litres/second	
Observation well	**$\Delta\log_{10}t$ (m)**	**Transmissivity (m^2/day)**
Drawdown data		
BH 208	0.24	368.6 (UG)
BH 209	0.27	331.1 (LG)
BH 216	0.29	305.4 (UG)
BH 218B	0.32	280.2 (LG)
BH 265	0.32	280.7 (Rock)
BH 225	0.04	2035.8 (UG)
Recovery data		
BH 208	0.23	385.0 (UG)
BH 209	0.24	366.3 (LG)
BH 216	0.35	251.22 (UG)
BH 218B	0.29	305.37 (LG)
BH 265	0.33	270.40 (Rock)

As a consequence of BH 216 being in close proximity to BH 217 the drilling process will impact on the ground between the pumping and observation well, making this a higher than natural transmissivity.

It can also be noted that the transmissivity calculated from the observed drawdown in the Upper Gravel gives a higher value than the average. This is because the Upper Gravel in general does not respond to the same degree as the underlying strata. In some observation wells in the Upper Gravel the drawdown response gives rise to a realistic transmissivity, however in most cases the calculation of transmissivity is not representative of the aquifer.

It is for the above reason that the transmissivity of 2036 m^2/day as calculated from BH 225 is not considered valid.

The drawdown and recovery graphs used to calculate the transmissivity can be seen in Appendix 3. Pumping and observation well locations are depicted in Figure 5.6.

5.3.5. Pumping Test BH 265

Table 5 - Summary of well construction for BH 265 pumping test

Borehole	Type	Ground Level (m OD)	Distance from Pumping Well (m)	Screen (m OD)
BH 265	Pumping from rock (also observation)	20.53	-	-8.47 to -0.47
BH 208	Observation in upper gravel	21.25	78.64	6.75 to 15.45
BH 209	Observation in lower gravel	21.22	106.63	4.72 to 9.72
BH 216	Observation in upper gravel	21.33	16.79	12.33 to 15.58
BH 218B	Observation in lower gravel	21.48	20.55	1.98 to 4.98
BH 225	Observation in upper gravel	28.73	124.94	18.73 to 23.73
BH 217	Observation in lower gravel	21.48	20.12	-0.72 to 5.08

Table 5A - Summary of BH 265 pumping test analyses

Start Date	28th February 2006	
Finish Date	1st March 2006	
Duration of test	24 Hours	
Type of data analysed	Transducer	
Pumping Rate	3.9 litres/second	
Observation well	**$\Delta s/\log_{10}t$ (m)**	**Transmissivity (m^2/day)**
Drawdown data		
BH 208	0.226	257.5 (UG)
BH 209	0.140	415.7 (LG)
BH 216	0.180	323.3 (UG)
BH 218B	0.252	230.5 (LG)
BH 225	0.142	408.4 (UG)
BH 217	0.191	305.5 (LG)

The drawdown and recovery graphs used to calculate the transmissivity can be seen in Appendix 4. Pumping and observation well locations are depicted in Figure 5.7.

5.3.6. Pumping Test BH 270A (1st test)

Table 6 - Summary of well construction for BH 270A pumping test

Borehole	Type	Ground Level (m OD)	Distance from Pumping Well (m)	Screen (m OD)
BH 270A	Pumping from rock	22.02	-	-0.98 to -7.02
BH 272	Observation in lower gravel	22.65	157.67	8.35 to approx. 10.35 (assumed)
BH 274	Observation in lower gravel	23.68	192.78	13.68 to 16.18
BH 290	Observation in rock	22.02	6.90	5.52 to 21.02

Table 6A - Summary of BH 270A pumping test analyses

Start Date	14th April 2006	
Finish Date	14th April 2006	
Duration of test	8 Hours	
Type of data analysed	Transducer	
Pumping Rate	13.1 litres/second	
Observation well	**$\Delta s / \log_{10} t$ (m)**	**Transmissivity (m^2/day)**
Drawdown data		
BH 272	2.11	98.4 (LG)
BH 274	1.95	106.5 (LG)
BH 290	1.91	108.8 (Rock)
Recovery data		
BH 272	1.2	173.1 (LG)
BH 274	1.435	144.8 (LG)
BH 290	1.3	159.8 (Rock)

The drawdown and recovery graphs used to calculate the transmissivity can be seen in Appendix 5. Pumping and observation well locations are depicted in Figure 5.8.

Note that a drawdown response was measured in BH265, however as this is 379m from borehole BH270, the response after only 8 hours pumping is too small to analyse.

Furthermore there was no discernable drawdown at BH 241 at 171m from the pumping well BH 270. This is likely due to the fact that BH 241 was monitored in the upper gravel which is separated from the rock aquifer by a lower permeability aquitard at this location.

5.3.7. Pumping Test BH 228

Table 7 - Summary of well construction for BH 228 pumping test

Borehole	Type	Ground Level (m OD)	Distance from Pumping Well (m)	Screen (m OD)
BH 228	Pumping from lower gravel	28.69	-	3.19 to -5.19 and 6.19 to 8.19
BH 288	Observation in upper/ lower gravel	24.56	75.47	1.56 to 9.16

Table 7A - Summary of BH 228 pumping test analyses

Start Date	4th May 2006	
Finish Date	4th May 2006	
Duration of test	8 Hours	
Type of data analysed	Transducer	
Pumping Rate	2.71 litres/second	
Observation well	**$\Delta s / \log_{10} t$ (m)**	**Transmissivity (m^2/day)**
Drawdown data		
BH 288	0.146	293.347 (UG/LG)
Recovery data		
BH 288	0.175	244.735 (UG/LG)

The drawdown and recovery graphs used to calculate the transmissivity can be seen in Appendix 6. Pumping and observation well locations are depicted in Figure 5.9.

Note that boreholes BH225, BH233, BH239, BH268 and BH241 were also monitored with pressure transducers during this pumping test. However, the drawdown response was small in these boreholes and so was not able to be accurately analysed.

5.3.8. Pumping Test BH 224

Table 8 - Summary of well construction for BH 224 pumping test

Borehole	Type	Ground Level (m OD)	Distance from Pumping Well (m)	Screen (m OD)
BH 224	Pumping from lower gravel	28.62	-	4.62 to -6.62 and 7.62 to 9.62
BH 218B	Observation in lower gravel	21.48	107.27	1.98 to 4.98
BH 234	Observation in rock	27.38	147.27	-3.87 to 4.38
BH 265	Observation in rock	20.53	126.91	-8.47 to -0.47
BH 266	Observation in rock	24.05	47.85	-10.05 to -1.65
BH 288	Observation in upper/ lower gravel	24.56	106.69	1.56 to 9.16

Table 8A - Summary of BH 224 pumping test analyses

Start Date	8th May 2006	
Finish Date	8th May 2006	
Duration of test	8 Hours	
Type of data analysed	Transducer	
Pumping Rate	3.51 litres/second	
Observation well	**$\Delta s/\log_{10}t$ (m)**	**Transmissivity (m^2/day)**
Drawdown data		
BH 218B	0.234	237.2 (LG)
BH 234	0.500	111.1 (Rock)
BH 265	0.253	219.4 (Rock)
BH 266	0.260	213.5 (Rock)
BH 288	0.330	168.2 (UG/LG)
Recovery data		
BH 218B	0.255	217.9 (LG)
BH 265	0.238	233.7 (Rock)
BH 266	0.256	216.6 (Rock)
BH 288	0.238	233.7 (UG/LG)

The drawdown and recovery graphs used to calculate the transmissivity can be seen in Appendix 7. Pumping and observation well locations are depicted in Figure 5.10. The response from BH268 was too small to be analysed.

5.3.9. Pumping Test BH 217 (3rd test)

Table 9 - Summary of well construction for BH 217 pumping test

Borehole	Type	Ground Level (m OD)	Distance from Pumping Well (m)	Screen (m OD)
BH 217	Pumping from lower gravel	21.48	-	-0.72 to 5.08
BH 216	Observation in upper gravel	21.33	8.68	12.33 to 15.58
BH 218B	Observation in lower gravel	21.48	5.65	1.98 to 4.98
BH 266	Observation in rock	24.05	68.71	-10.05 to -1.65

Table 9A - Summary of BH 217 pumping test analyses

Start Date	13th May 2006	
Finish Date	19th May 2006	
Duration of test	6 Days	
Type of data analysed	Transducer	
Pumping Rate	9.66 litres/second	
Observation well	**$\Delta s/\log_{10}t$ (m)**	**Transmissivity (m^2/day)**
Drawdown data		
BH 216	0.185	825.7 (UG)
BH 218B	0.240	636.5 (LG)
BH 266	0.250	611.0 (Rock)
Recovery data		
BH 216	0.363	420.8 (UG)
BH 218B	0.357	427.9 (LG)
BH 266	0.385	396.8 (Rock)

Note that boreholes BH233, BH270A, BH269, were also monitored with pressure transducers during this pumping test. However, the drawdown response was small in these boreholes and so was not able to be accurately analysed.

As a consequence of BH 216 and BH 218 being in close proximity to BH 217 the drilling will impact on the ground between the pumping and observation well, making this a higher than natural transmissivity. The interpretation of the results in BH 266 appears to be of high quality and gives a transmissivity of 611m^2/day. This is higher than previously measured but the test is over a longer duration. One of the reasons for this is that the slope may be flatter because of the leakage from the upper layers. If this is the case then the transmissivity is over-estimated. Nevertheless, a transmissivity of 611m^2/day must be considered possible even with the recovery test giving 397m^2/day.

The drawdown and recovery graphs used to calculate the transmissivity can be seen in Appendix 8. Pumping and observation well locations are depicted in Figure 5.11.

5.3.10. Pumping Test BH 270A

Table 10 - Summary of well construction for BH 270A pumping test

Borehole	Type	Ground Level (m OD)	Distance from Pumping Well (m)	Screen (m OD)
BH 270A	Pumping from rock	22.02	-	-0.98m to 7.02
BH 234	Observation in rock	27.38	104.29	-3.87m to 4.38
BH 238B	Observation in lower gravel	22.11	16.70	7.61 to 16.98
BH 239	Observation in upper gravel	21.96	3.34	17.96 to 18.36
BH 252A	Observation in upper gravel	26.38	111.28	19.38 to 20.88
BH 266	Observation in rock	24.05	47.85	-10.05 to -1.65
BH 268	Observation in rock	28.59	198.73	-16.21 to -6.91
BH 274	Observation in lower gravel	23.68	192.78	13.68 to 16.18
BH 290	Observation in rock	22.02	6.80	5.52 to 21.02

Note that the calculations below demonstrate a high level of transmissivity calculated from observation drawdowns in BH 239 and BH 252. However these transmissivities are not considered valid as they are based on drawdowns that are not representative of the change in water head in the aquifer as described previously.

From the remaining results, there are two distinctive patterns. The results from boreholes to the northeast of the site, i.e. BH 234, BH 238B, BH 274 and BH 290, produce an average transmissivity of 79.4 m^2/s. However the boreholes to the southwest of the site, i.e. BH 266 and BH 268, give rise to an average transmissivity of 335 m^2/s.

In OGI's opinion it is most likely that the interpretation of this response is that the response at the northern observation wells is governed by the transmissivity of the rock. However, at the location of BH 266 and BH 268, the rock is overlain by the transmissive sand and gravel layer dominant at the southeast of the site.

Because of the heterogeneous conditions present, it is not feasible to state that the average transmissivity 335 m^2/day calculated at BH 266 and BH 268 is valid, as the conditions for a Jacob analysis are not met. However, this value is clearly more representative of the sand and gravel layer and confirms the clear distinction between the groundwater flow in the rock and that in the sand and gravel.

Note that boreholes BH228 and BH265 were also monitored with pressure transducers during this pumping test. However, the drawdown response was small in these boreholes and so was not able to be accurately analysed.

Table 10A - Summary of BH 270A pumping test analyses

Start Date	22nd May 2006	
Finish Date	5th June 2006	
Duration of test	14 Days 18 Hours	
Type of data analysed	Transducer Readings	
Pumping Rate	11.3 litres/second	
Observation well	**$\Delta s / \log_{10} t$ (m)**	**Transmissivity (m^2/day)**
Drawdown data		
BH 234	2.867	62.7 (Rock)
BH 238B	1.87	96.153 (LG)
BH 239	0.57	315.4 (UG)
BH 252A	0.27	666.0 (UG)
BH 266	0.55	326.9 (Rock)
BH 268	0.525	342.9 (Rock)
BH 274	2.35	76.5 (LG)
BH 290	1.725	104.2 (Rock)
Recovery data		
BH 238B	2.3	78.2 (LG)
BH 274	2.9	62.0 (LG)
BH 290	1.9	94.6 (Rock)

Excluding the Upper Gravel these results demonstrate that the average transmissivity calculated for the rock is lower than that observed in the sand-gravel aquifer.

The drawdown and recovery graphs used to calculate the transmissivity can be seen in Appendix 9. Pumping and observation well locations are depicted in Figure 5.12.

5.3.11. Pumping test BH 234

Table 11 - Summary of well construction for BH 234 pumping test

Borehole	Type	Ground Level (m OD)	Distance from Pumping Well (m)	Screen (m OD)
BH 234	Pumping from rock	27.38	-	-3.87m to 4.38
BH 269	Observation in rock	24.75	47.23	-7.35 to -0.35
BH 231	Observation in lower gravel	25.03	25.79	6.23 to 10.23

Table 11A - Summary of BH 234 pumping test analyses

Start Date	10th June 2006	
Finish Date	11th June 2006	
Duration of test	24 Hours	
Type of data analysed	Transducer	
Pumping Rate	0.55 litres/second	
Observation well	**$\Delta s/\log_{10}t$ (m)**	**Transmissivity (m^2/day)**
Drawdown data		
BH 269	0.252	34.499 (Rock)
BH 231	0.533	16.326 (LG)
Recovery data		
BH 231	0.94	9.249 (LG)

The drawdown and recovery graphs used to calculate the transmissivity can be seen in Appendix 10. Pumping and observation well locations are depicted in Figure 5.13. The calculated transmissivity of the rock appears to be low, however it is likely that because of the low pumping rate, the influence on the observation wells is low and the drawdown will be influenced by other factors such as leakage and rainfall.

Note that boreholes BH225, BH233, BH239, BH268 and BH241 were also monitored with pressure transducers during this pumping test. However, due to the low permeability of the rock in this pumping location, the maximum abstraction rate from the borehole was 0.55lit/s. Under these pumping conditions, the drawdown responses were small in the observation boreholes and so were not able to be accurately analysed.

A further pump test was conducted from borehole BH 238 with monitoring at BH237C, BH290, BH231, BH 239, BH292 and BH293. Again due to the low permeability of the rock in this pumping location, the maximum abstraction rate from the borehole was 0.40 lit/s. Under these pumping conditions, the drawdown responses were small in the observation boreholes and so were not able to be accurately analysed.

5.4 Computer Simulation of Pumping from BH 270A

5.4.0. On 14 April 2006 Borehole BH 270A was pumped at a rate of 1135 m^3/d with the water level observed in observation well BH 272, a distance of 174m from the pumping well. The observed piezometric drawdown in observation borehole 272 is shown in Figure 5.14.

5.4.1. Because a substantial lowering in the piezometric head was observed at such distance from the pumping well, a computer simulation of the results was undertaken. OGI's transient groundwater model CVM was used to simulate the drawdown over the period of the test by matching the observed data with a simulated response. The resulting best match fit is presented in Figure 5.15.

5.4.2. To fit this observed drawdown required the input of transmissivity, T, as 80 m^2/d, and a storage coefficient, S, of 0.0007. This value of T is very close to the average value of 79.4 m^2/day derived from the Jacob method for the BH 270 test.

5.4.3. The low value of S calculated demonstrates a low compressible aquifer system which implies that the aquifer is both fully saturated resulting in artesian conditions and that the rock aquifer is of low compressibility. This low rock compressibility implies that there would be very little expectation of deformation due to effective stress changes in the rock.

5.4.4. This test illustrates that the regional drawdown resulting from dewatering this lower rock aquifer can be very widespread. Even after pumping for a period of only eight hours, a drawdown of 1.7m is observed at a distance of 174m from the pumping well.

5.4.5. To confirm the calculation of transmissivity from the BH 270 14 day pumping test, OGI's CVM model was again used to reanalyse the data.

5.4.6. The CVM model allows the field results to be simulated with the added condition that there is leakage to the aquifer from above and below as a result of the lowered piezometric head in the aquifer.

5.4.7. Figure 5.16 demonstrates a fit between the drawdown observed in Borehole 274 with the simulated drawdown. The parameters used from this best fit are as follows:

Transmissivity = 55 m^2/day

Storage Coefficient = 0.0006 = 6×10^{-4}

Leakanace * = 2.85×10^{-3} day^{-1}

(* Leakance Coefficient = Aquitard permeability/Aquitard Thickness
Anderson & Woessner, Applied Groundwater Modeling, 1992)

5.4.8. These results demonstrate a lower value of transmissivity than the Jacob Method which does not take into account leakance. The results also demonstrate that there is leakage through the layers above the rock down into the rock aquifer caused by the reduction in piezometric head in the aquifer caused by pumping.

5.4.9. Four rock pumping test results were simulated (BH234, BH238B, BH274 and BH290) using the CVM model which are presented in Appendix 11. The results from BH270, presented in Table 12 below demonstrates that after analysis with the CVM Model, the results were very similar the Jacob analysis.

Table 12 - Modification of BH 270A pumping test analyses

Start Date	22nd May 2006	
Finish Date	5th June 2006	
Duration of test	14 Days 18 Hours	
Type of data analysed	Transducer Readings	
Pumping Rate	11.3 litres/second	
Observation well	**Transmissivity (m^2/day) Jacob Analysis**	**Transmissivity (m^2/day) CVM Analysis**
BH 234	62.7 (Rock)	58.5 (Rock)
BH 238B	96.5 (LG)	96.2 (LG)
BH 274	74.5 (LG)	55.0 (LG)
BH 290	104.3 (Rock)	104 (Rock)
Average Transmissivity	**84.3 (Rock)**	**78.4 (Rock)**

5.4.10. Matching the data with the CVM model as presented in Appendix 11 also gives rise to other parameters that govern the groundwater response. These parameters include the Storage Coefficient and the Aquifer Leakance. The parameters are presented below for the 14 day, 18 hour pumping test in BH270A conducted on 22nd May 2006, together with the 8 hour test conducted on 14 April 2006 as observed in BH 272.

Table 13 – Results from BH270A pumping test analyses using CVM Model

Pumping Well 270A Observation well	**Transmissivity (m^2/day) CVM Analysis**	**Storage Coefficient CVM Analysis**	**Aquifer Leakance (d^{-1}) CVM Analysis**
BH 272 (14 April)	80.0 (Rock)	0.0007 (Rock)	0.0 (Rock)
BH 234 (22 May)	58.5 (Rock)	0.00029 (Rock)	0.000185 (Rock)
BH 238B (22 May)	96.2 (LG)	0.007 (LG)	0.0025 (LG)
BH 274 (22 May)	55.0 (LG)	0.0006 (Rock)	0.00285 (Rock)
BH 290 (22 May)	104 (Rock)	0.035 (Rock)	0.012 (Rock)

5.4.11. Test results from pumping in the gravel at the southwest end of the site were analysed using the CVM model as presented in Appendix 12. These parameters include the Storage Coefficient and the Aquifer Leakance. These parameters are presented below for the 24 hour pumping test in BH217 conducted on 22 February 2006.

Table 14 – Results from BH217 pumping test analyses using CVM Model

Pumping Well 217 Observation well	**Transmissivity (m^2/day) CVM Analysis**	**Storage Coefficient CVM Analysis**	**Aquifer Leakance (d^{-1}) CVM Analysis**
BH 208 (22 February)	368.6 (UG)	0.00106 (UG)	0.0 (UG)
BH 209 (22 February)	331.1 (LG)	0.00106 (LG)	0.0 (LG)
BH 216 (22 February)	305.4 (UG)	0.03400 (UG)	0.0 (UG)
BH 218B (22 February)	280.2 (LG)	0.02000 (LG)	0.0 (LG)
BH 265 (22 February)	280.7 (Rock)	0.00420 (Rock)	0.0 (Rock)

5.4.12. By comparing the above two tables, it can be seen that the 14 days pump test in the rock borehole BH270 indicates leakage from the overlying ground. However, the response from the pumping in the gravel aquifer at the southwest does not indicate a leaky aquifer. This confirms the overall view that the southwest aquifer is an unconfined water table aquifer, with the northeast aquifer being confined with leakage from an overlying layer.

5.5 Summary and Interpretation of Pumping tests

5.5.0. Numerous groundwater pumping tests have been conducted at the Raith Junction site designed to provide estimates of transmissivity at varying locations within and surrounding the construction excavation.

5.5.1. Pumping tests are extremely valuable in determining the groundwater conditions as they are in effect mini dewatering operations. By measuring the pumping rates together with the observed drawdowns in the surrounding strata, governing properties can be calculated. From these properties, computer simulations can be conducted to predict the total pumping dewatering requirements together with an estimate of the wider drawdown impact resulting from the dewatering operation.

5.5.2. The analysis of results has demonstrated a range in calculated values of transmissivity. In particular there is a significant difference between the transmissivity of the sand & gravel aquifer to the southwest of the M74 and the rock to the northeast of the M74.

5.5.3. It was inappropriate to analyse the data for all observation readings, either because they did not conform to the conditions of the analysis, or because there was no response in the observation well during the test. As a consequence, the transmissivity values considered valid are those values from the tests which are considered appropriate for analysis. This does not mean that the tests are not valid, only that some of the data measurements are not suitable for transmissivity calculation.

5.5.4. The highest transmissivity has been identified in the sand & gravel layers at the southeast of the site. In this location the transmissivity is expected to range between 19 m^2/day to 611 m^2/day, with a mean value calculated as approximately 200 m^2/day.

5.5.5. This variability in transmissivity is quite normal within an aquifer as the ground is usually heterogeneous. Furthermore, during the tests, pumping was conducted at different rates and during varying weather patterns.

5.5.6. The transmissivity calculated in the underlying rock to the northeast of the site (calculated to be no more than 173 m^2/day) is clearly lower than in the gravel layer at the south of the site. The mean transmissivity to the northeast is calculated as approximately 79.4 m^2/day with the computer simulations producing a best fit transmissivity of 80 m^2/day and 78.4 m^2/day.

5.5.7. Note that high flows have been observed in the underlying rock even with a lower transmissivity. This is because of the initial high flowing artesian head present in this aquifer.

5.5.8. Whilst there have been distinctly different ground conditions encountered between the southwest and northeast of the sites, it is considered that there is a connection, even if partial, between the southwest gravel and the northeast rock aquifers.

5.5.9. The pumping test performed in BH224 (pumping from the gravel at 303 m^3/day) demonstrates that drawdown is observed on both sides of the M74, i.e. in observation wells both to the northeast and southwest.

5.5.10. Specifically, 300 – 400mm drawdown was observed in boreholes in the southwest, with 470mm drawdown observed in BH288 (northeast upper/lower gravel) and 150mm drawdown observed in BH234 (northeast rock).

5.5.11. These observations confirm that there is some connection between the southwest gravel aquifer and the northeast rock aquifer, although this certainly may be a partial connection, restricted by the presence of a possible fault.

5.5.12. To illustrate the range in transmissivity derived from the pumping test analyses, a normal distribution has been applied to both the derived transmissivity and the derived log of transmissivity.

5.5.13. Furthermore the range of transmissivity has been separated into those values derived from the test in the northeast (rock) and those in the southwest (predominantly sand and gravel).

5.5.14. Table 15a and Table 15b below presented the mean, standard deviation, and the 5% and 95% values of transmissivity, and log transmissivity, for the southwest and northeast respectively.

Table 15a – Results from pumping test analyses from boreholes in the southwest

Definition	Transmissivity (m^2/day)	Log_{10} (Transmissivity)	Transmissivity (m^2/day)
Mean Value	250.4	2.305	201.8
Standard Deviation	126.2	0.350	n/a
Lower 5%	42.8	1.729	53.6
Upper 5%	458.1	2.881	760.2

Table 15b – Results from pumping test analyses from boreholes in the northeast

Definition	Transmissivity (m^2/day)	Log_{10} (Transmissivity)	Transmissivity (m^2/day)
Mean Value	89.1	1.858	72.0
Standard Deviation	46.6	0.348	n/a
Lower 5%	12.4	1.285	19.3
Upper 5%	165.8	2.430	268.9

6. Impact of Dewatering

6.0.0. Based on groundwater properties as derived from the pumping tests, it is possible to input these parameters into a model of the site dewatering. The resulting output will include the required pumping rate from a dewatering well arrangement, together with the wider drawdown surrounding the underpass excavation.

6.0.1. It must be emphasised that the ground investigation conducted at the Raith site demonstrates clearly that the geology and hydrogeology is complex and certainly multi-layered. The conceptual drawing of the geology by Mouchel – Fairhurst (Drawing No. 53213/002) clearly demonstrates the complex and multi-layered nature of the ground.

6.0.2. Furthermore, the dominant aquifer at the southwest end of the site is the lower gravel layer, with the rock aquifer being the dominant aquifer at the northeast end of the site.

6.0.3. However, the purpose of the modelling is to assess whether the dewatering of the underpass excavation is feasible, i.e. to assess if the ground of a condition to allow dewatering to work in practice. To establish this, it is not necessary to predict the precise abstraction rate, but a range of likely abstraction rate. From this information, further detailed design may be conducted in advance of the dewatering operation.

6.0.4. To provide this assessment, it is the skill of the groundwater modeller to choose the appropriate model that will provide sufficient accuracy for which engineering designs can be produced. Furthermore, in making the decision of which model to use, account must be taken of the level of data available which is input into the model, together with sensitivity the of the output to the level of input detail.

6.0.5. It is for this reason, that whilst it is accepted that the geology is multi-layered, it is considered that the application of a multi-layered numerical model is an unnecessarily complex solution to simulating the critical groundwater behaviour at Raith relevant to a dewatering operation.

6.0.6. For the purpose of the modelling of groundwater from the surrounding hydrogeological environment to the dewatered excavation, including the calculation of the drawdown at distance from the excavation, OGI considers that a single layer model to be a pragmatic approach during this stage of the investigation. Justification of the application of this model to the multi layered aquifer at Raith is presented in Appendix 13.

6.0.7. During the detailed design stage when the precise method of dewatering is chosen, it is recommended that a more complex model be applied to the groundwater regime. The purpose of a more complex model is to develop a dewatering solution that provides the required stability within the excavation, but minimises the groundwater lowering in the surrounding aquifer.

6.0.8. For the purpose of calculating the expected range of flows to the dewatering system OGI's groundwater model GEMOS (Groundwater Engineering & Management Optimisation Software) is applied. This model simulates a single aquifer layer and is a model developed specifically to predict groundwater behaviour surrounding a series of groundwater abstraction wells.

6.0.9. The drawdown simulations presented are based on the assumption that dewatering is taking place from wells located within the excavation as illustrated in Figure 6.1. This dewatering approach will have less impact on the groundwater lowering than the scenario in which wells are installed outside the central roundabout and construction area.

6.0.10. The location of the dewatering boreholes is not a design by OGI, but certainly an approach that needs to be considered. Whilst it is accepted that there is more inconvenience with boreholes within the cut-off walls, this approach is widely used within industry, provides an in-built safety factor against basal heave, and has less impact of drawdown on the surrounding aquifer.

6.0.11. From the analysis of the pumping tests, the transmissivity of gravel aquifer has been found to be in the range $19m^2$/day to $611m^2$/day (excluding those values which are considered not to be valid).

6.0.12. However, the transmissivity at the northeast end of the site has been found to be generally lower, i.e. in the range $62m^2$/day – $104m^2$/day, most likely because the groundwater flow is dominated by the flow in the rock aquifer.

6.0.13. The initial condition of groundwater water level prior to pumping from the boreholes is not constant, with a downward gradient observed from the northeast to the southwest.

6.0.14. OGI's model simulation is based on the initial pre-construction groundwater level as shown in Figure 6.2, which also depicts some of the groundwater level measurements along the new underpass section. Figure 6.2 was based on data provided by MFJV and was taken from the long term transducer data for the period of December 2005 to January 2006. (See table A14.2 in appendix 14 for the source of the data used.)

6.0.15. Because one of the objectives is to minimise the drawdown impact outside the underpass, dewatering wells have been positioned inside the secant/diaphragm cut-off walls. This will have the effect of lifting the effective dewatering level as felt by the surrounding ground, even though the water table can be maintained at 14m OD (the required depth of excavation) within the excavation. This can be achieved only if the ends of the excavation are also cut-off with an impermeable wall to the same depth as the side cut off walls.

6.0.16. The required depth of cut-off wall to reduce the impact will depend upon a number of factors, most significantly the presence of horizontal layers that would give rise to anisotropy of ground conditions.

6.0.17. Figure 6.3 illustrates the computer generated flow net of equipotential lines and flow paths for the case of groundwater flow beneath a cut off wall installed to 7m OD. This is deeper than the proposed cut off wall in the south, shallower than intended for the north, but typical of the wall depth proposed over the central section of the underpass.

6.0.18. It most be noted that the flow lines and equipotential lines do not intersect at 90° in this flow net because the ground has been modelled as being anisotropic. For this simulation, the ratio of horizontal to vertical permeability is in the ration 4:1. This is considered conservative with the vertical permeability being substantially lower than the horizontal permeability due to the stratified clay/silt/sand deposits within the gravel.

6.0.19. Based on the computer simulated flow net, the mean predicted water table outside the excavation cut-off wall can be calculated. Figure 6.4 depicts the effective water table lowered to a level of 16m OD outside the cut off wall, resulting from the water level drawn down to 14m OD in the internal wells installed within the sheet pile wall.

6.0.20. These simulations demonstrate that there is a significant reduction in flow to the excavation, this being in the order of a 25% reduction, under the condition that the open ends of the underpass are sealed to prevent horizontal groundwater flow to the underpass excavation.

6.0.21. As a consequence of the reduction in flows, if further amelioration of the drawdown outside the site proves to be critical, extending the cut-off walls further into the sand may be considered.

6.0.22. Based on a required effective dewatering level to 16m outside the excavation, i.e. 14m OD inside the excavation, the GEMOS model has simulated the surrounding resulting drawdown impact.

6.0.23. Figure 6.5 presents the simulated steady state piezometric head level based on the lowering of the water level to 14.0m OD within the underpass, 16m OD outside the underpass. This figure also presents the initial level from which the reduction in (i) water table in the southwest, and (ii) piezometric head in the northeast can be assessed.

(Note that this section is along the section from Easting 270721, Northing 657811, to Easting 272770, Northing 660000 as illustrated in Figure 6.6).

6.0.24. Figure 6.7 presents a contour plot of the computer simulated drawdown superimposed on a map of the area. It must be noted that the drawdown presented here is the reduction in piezometric surface below the initial level, even if this level is elevated above ground level. For example, if the initial piezometric surface is 3.0m above ground level and is reduced to 1.0m above ground level, the simulated result is a 2.0m drawdown.

6.0.25. Note that the widest contour area plotted is for a 1.0m drawdown and demonstrates a large impact area of approximately 3000m diameter. However, this is considered to be the worst case simulation as conditions that ameliorate the drawdown cone, such as natural or artificial groundwater recharge, are not included in this model.

6.0.26. As this predicted area of drawdown is relatively small, Figure 6.8 has been drawn to a larger magnification to illustrate the area of artesian head reduction or water table drawdown in more detail.

6.0.27. The steady state groundwater abstraction rates required to maintain the piezometric level at 14.0m OD within the cut-off wall (effective drawdown to 16m OD immediately outside the cut-off wall), will be predominantly dependent on the value of Transmissivity.

6.0.28. The steady state abstraction rate is also dependent upon the boundary level of the artesian head, this being related to the water level in the River Clyde, together with the rate of percolation of surface infiltration into the aquifer.

6.0.29. To assess the required steady state pumping rate, two computer models were applied to the Raith hydrogeology. The first model, GEMOS, is a steady state single layer analytical 2-dimensional model. The second model, SEFTRANS, is a more flexible steady state or transient 2-dimensional finite element model

6.0.30. Under steady state conditions, for an effective 16m piezometric head at the excavation and under the ambient artesian head conditions encountered at the time of pump testing, the simulated required dewatering rates using both models are as follows:

Transmissivity (m^2/day)	GEMOS Model Steady State Pumping to reach 16m OD (lit/s)	SEFTRANS Model Steady State Pumping to reach 16m OD (lit/s)
20	3.562	3.549
50	8.910	8.873
100	17.81	17.75
200	35.62	35.49
300	53.43	53.24
400	71.24	70.99
500	89.05	88.73
600	106.9	106.5
700	124.7	124.2
800	142.5	142.0

Transient Modelling

6.0.31. Calculations of the transient behaviour of the drawdown have been conducted using two-dimensional single layered finite-element model, SEFTRANS with a transmissivity of 300m^2/d. These calculations demonstrate that for a storage coefficient of 0.2, the required pumping rate at 30 days following the commencement of pumping is between 140% and 200%, with the required pumping rate at 100 days as between 114% and 140% of the steady state pumping value. This results in the following calculation of the required abstraction rates at 30 days and 100 days after the commencement of pumping.

Transmissivity (m^2/day)	Abstraction rate at 30 days to 16m OD (lit/s)	Abstraction rate at 100 days to 16m OD (lit/s)	Steady State Pumping to reach 16m OD (lit/s)
150	52	38	27
300	87	67	53
600	148	121	106

6.0.32. The abstraction rate indicated above is within the capacity of a dewatering system. No specific design of a system has been presented in this report, however, for an initial abstraction rate of 148 lit/s, two rows of wells at 10m spacing over a 300m length would result in 62 No. wells, each requiring a pumping capacity of 2.4 litre/sec.

6.0.33. At the northern section of the site, the main source of the artesian pressure is from the rock aquifer. This being the case, to prevent heave in the ground beneath the excavation it is only necessary to lower the artesian head in the rock, or in any other confined layers, to a level that ground heave will not occur, or that water will not force up the side of the diaphragm wall.

6.0.34. Preventing ground heave to achieve stability will not necessarily prevent groundwater from seeping to the excavation surface. However, such seepage can be managed locally by usual techniques used to manage surface water or near surface groundwater.

6.1 Impact of dewatering on forming piles and diaphragm wall

6.1.0. During the installation of a diaphragm wall or secant pile wall it may be necessary to reduce excess artesian pore water pressure to facilitate installation. In this instance the dewatering requirements are less than that required for full excavation thereby allowing staged installation of the full dewatering scheme running at a reduced efficiency until the full dewatering is required.

6.1.1. Removal of the excess artesian pore water pressure at the northern end of the site can be achieved by installing groundwater abstraction boreholes together with installing pumps within the boreholes to lower the artesian head in the area of the wall construction.

6.1.2. If the diaphragm/secant walls are installed from the existing ground level of approximately 22m OD, then the lowering of the artesian head to 21m OD can be achieved by pumping from the gravel and rock layers.

6.1.3. To provide an indicative impact of first stage pressure relief, a computer simulation of this dewatering stage has been conducted in which the artesian level has been reduced to 21m OD using dewatering wells installed within the footprint of the underpass construction between the cut-off walls.

6.1.4. Figure 6.9 presents the simulated steady state piezometric head level based on the lowering of the artesian head to 21.0m OD within the underpass footprint.

(Note that this section is along the section from Easting 270721, Northing 657811, to Easting 272770 Northing 660000 as illustrated in Figure 6.6).

6.1.5. Figure 6.10 presents a contour plot of the predicted reduction in artesian pressure head superimposed on a map of the area. Note that the largest contour area is for a 1.0m drawdown.

6.1.6. As this predicted area of drawdown is relatively small, Figure 6.11 has been drawn to a larger magnification to illustrate the area of artesian head reduction in more detail.

6.1.7. The model used to produce the enclosed drawdown contours is a steady state model for which a single transmissivity is applied. As such it is considered that this is a conservative model as the model itself does not have the capability to simulate time dependent behaviour, or the flow between different hydrological layers.

6.1.8. In regard to the sensitivity of the predictions to different levels of transmissivity, there are two key factors. The first is impact on the required abstraction rate. This issue has been discussed with the required abstraction being effectively directly proportional to the transmissivity.

6.1.9. The other impact of a higher transmissivity would be the prediction of a wider drawdown area with the same time scale. However, the current groundwater model simulations present only a steady state (long term) response in which there is little sensitivity of transmissivity on drawdown. If the transient model is used to predict drawdowns, inevitably there will be less impact predicted on the surrounding aquifer, but this will also be dependent on the time scale of the construction period, as well as the storativity of the aquifer.

6.1.10. OGI's GEMOS model has been considered appropriate as it is flexible and efficient to use for the assessment of the feasibility of dewatering with a conservative prediction on dewatering impacts.

6.1.11. Should further detail be required then OGI has various other models that can be applied. However, these require a substantial investment in time and cost. If the time is available, the ability to apply a more powerful model inevitably results in substantial benefits, including cost savings and increased productivity to the project.

6.2 Impact of temporary dewatering on surface water bodies

6.2.0. From the pumping tests conducted over the last six months it has been possible to provide a calculation of the groundwater abstraction rate needed to lower the artesian/piezometric head to the required level for the construction of the underpass.

6.2.1. During the dewatering of the ground at Raith, the majority of the water abstracted is from:

a) removal of water stored in the ground, and

b) removal of water from, or which would have flowed to, the River Clyde.

6.2.2. However there are four significant water bodies within the predicted area of drawdown (see Figure 2.1) that need to be commented on, and they are:

- Strathclyde Loch.
- Pond No. 5, which is to the north of Junction 5.
- Pond No. 1 which is located at the southern end of the SSSI.
- The River Clyde.

6.2.3. The water level in the Strathclyde Loch is at a level of approximately 23m OD, which is approximately 4m above the level of the River Clyde, in the vicinity of the site. This also results in the loch level being at an estimated distance of 4m above the piezometric level in the underlying aquifer. The water levels in the River Clyde and the Strathclyde Loch were provided by Mouchel Fairhurst JV.

6.2.4. The model prediction indicates a maximum drawdown of 0.5m beneath the centre of the Loch, (see Figure 6.7). However the fact that there is already a 4m difference between the Loch and the River Clyde implies that the Loch is in ‘hydraulic isolation’ and the base of the Loch is sealed or has silted up over the years.

6.2.5. As a result in either case there is unlikely to be any significant lowering of the water level in the Strathclyde Loch as a result of a 0.5m lowering of the piezometric surface in the aquifer beneath the loch.

6.2.6. The source of water for Pond No 5 is likely to be from three sources:

- Direct rainfall to the pond.
- Surface water running into the pond.
- Groundwater seeping into the pond from shallow groundwater flow in the upper aquifer.

6.2.7. In addition to the above three sources, it is theoretically possible for there to be an upward component of groundwater flow to the pond as a result of the upward hydraulic gradient between the underlying rock aquifer and the surface water elevation. However, in light of the low permeability layer separating the upper aquifer from the rock aquifer, this groundwater flow to the pond is likely to be insignificant.
Figure 6.12 demonstrates this concept.

6.2.8. To maintain the water level within the pond, the recharge of the pond using the abstracted groundwater can be considered. The MFJV Contamination report will need to be consulted to assess the suitability of the groundwater for recharging the surface water bodies.

6.2.9. Prior to discharge to the hydrological environment, the water may be required to pass through a filtration system and be chemically tested to ensure that the water is not contaminated.

6.2.10. The level of the Pond No. 1 water is approximately 17.8m OD, which is approximately 1.2m below the level of the River Clyde and at the same level as the ambient groundwater level as observed in BH201.

6.2.11. The fact that there is over a metre difference in water level between Pond No. 1 and the River Clyde suggests that they are not connected.

6.2.12. As a result, the source of the water to the pond is likely to be from three sources:

- Direct rainfall to the pond.
- Surface water running into the pond either directly or from a small burn.
- Artesian water rising to the pond resulting from the upward hydraulic gradient between the underlying aquifer and the surface water elevation.

6.2.13. As a consequence any discharge of abstracted groundwater from the dewatering system into the burn will recharge Pond No. 1. Again the MFJV Contamination report will need to be consulted to assess the suitability of the groundwater for recharging the surface water bodies.

6.2.14. Based on a qualitative assessment of the location of the surface water bodies, the following is concluded:

(i) There will be negligible impact on the Strathclyde Loch because it is in hydraulic isolation from the underlying aquifer.

(ii) There will be no impact on the River Clyde because the amount of water lost from the Clyde is insignificant compared with the flow in the Clyde, and that all the abstracted groundwater will eventually find its way back to the Clyde resulting in no net loss.

(iii) Some removal of flow from Pond No. 5 is possible due to the reduction in pressure in the underlying rock. However, the removal rate from the pond is likely to be low. If this is indeed the case, recharge of water to the pond from the dewatering system is recommended. Note that prior to discharge to the hydrological environment, the water will be required to pass through a filtration system and be chemically tested to ensure that the water is not contaminated. MFJV contamination report will need to be consulted to assess the suitability of the groundwater for recharging the surface water bodies.

(iv) Impact on the Pond No. 1 can be considered negligible for two reasons. The first is that it appears to be in hydraulic isolation from the Clyde. The second is that the current surface flow from the land surrounding the Junction 5 flows to Pond No. 1 and then out to the Clyde. If the discharge from the dewatering system is directed back to Pond No. 1, then the required water level will be maintained. MFJV contamination report will need to be consulted to assess the suitability of the groundwater for recharging the surface water bodies.

7. Effects of dewatering on ground consolidation

7.0.0. The above sections have demonstrated that it is feasible to lower the water table and to reduce the artesian pressure to a sufficient level to construct the underpass. However, as a consequence of the dewatering operation, the resulting cone of depression extends away from the site area and so will reduce the pore water pressure at distance from the excavation area.

7.0.1. Reduction in pore water pressure within the ground has the effect of increasing the effective stress within the soil matrix. This is governed by the equation:

$$\sigma' = \sigma^T - u_w \qquad (1)$$

where σ' is the effective stress, σ^T is the total stress on the and u_w is the pore water pressure.

7.0.2. Such a change in the effective stress is a cause of consolidation, with the magnitude of the consolidation governed by the compressibility of the soil at that particular stress state, as well as the change in effective stress itself.

7.0.3. The degree of consolidation will also depend upon whether the ground is being consolidated for the first time (known as virgin consolidation) or the ground has previously been consolidated, with the effective stress state subsequently relaxed. Under these conditions the soil is in an “over-consolidated” condition.

7.0.4. To evaluate the change in effective stress change in the soil at the northeast end of the site, it is important to consider the change in pore water pressures from the initial pressure to the post dewatered level.

7.0.5. The following is an example to illustrate the conditions expected in the vicinity of the northeast end of the site.

Ground level	22m OD
Artesian Piezometric head	25m OD
Top of Rock	0m OD
Bottom of rock	-10m OD

7.0.6. The dewatering is considered in two stages as follows:

Stage 1 Lowering of artesian pressure in rock aquifer to ground level, i.e. from 25m to 22m OD.

Stage 2 Further lowering artesian pressure in rock aquifer to final level, from 22m to 18m OD.

7.0.7. Note that the reduction to 18m OD is an arbitrary level for illustration purposes only. However, it is likely to be in this order to avoid uplift pressures on the excavation base. MFJV is scheduled to assess the settlement based on OGI's drawdown predictions.

7.0.8. From Figure 7.1 the stage lowering of the piezometric level can be calculated by subtracting the individual heads as shown. Note that this figure assumes that there are no negative pore pressures and associated effective stresses occurring in the zone above the reduced water table.

7.0.9. Based on the dependence of artesian piezometric head on recharge over a catchment area, the initial lowering of artesian head down to ground level at 22m OD is likely to have negligible impact in terms of consolidation at the ground surface.

7.0.10. Although this reduction in pore water pressure does indeed increase the effective stress, the soil under this effective stress range will almost certainly be in an over-consolidated condition. Even dewatering to levels below ground level will have limited impact on the dense gravels and the rock, these layers being of low compressibility.

7.0.11. Because of the high artesian heads in the underlying rock, there is an upward gradient of hydraulic head which results in an upward flow of water from the rock aquifer to the upper strata as depicted in the conceptual drawing Figure 7.2.

7.0.12. On dewatering of the rock there will be a reduction in water flowing upward to the upper strata as shown in Figure 7.3.

7.0.13. Conventional methods of mitigating the effects of drawdown on ground consolidation include the recharge of ground using the abstracted groundwater. There are a number of different methods of groundwater recharge appropriate to Raith as follows:

(i) Groundwater recharge via recharge wells,
(ii) Shallow wells surrounding specific buildings, and
(ii) Distributed drawdown to maintain water table levels.

These measures have the ability to maintain the ambient water table in the upper gravel layers surrounding critical buildings where there is an impermeable layer above the rock as is the case in the north. Where the aquifer is phreatic, i.e. there is a water table, recharging the ground with wells or trenches is more effective.

7.0.14. <u>Groundwater recharge via recharge wells</u>. Figure 7.4 illustrates the principle where some of the abstracted groundwater is pumped back into the aquifer so maintaining a higher water table surrounding the excavation.

This approach is more appropriate to the amelioration of groundwater at the south of the site. In this location water table conditions are present which makes recharge more feasible.

Figure 7.5 presents possible indicative locations where recharge using wells or trenches could be feasible if water table conditions are present. Note that it may also feasible to recharge some of the groundwater back to the upper gravel at the north of the site, but it is not expected that this will take a substantial amount of recharge.

7.0.15. Shallow wells surrounding specific buildings. In critical areas where it is essential to maintain a high water table in the upper deposits, it may be possible to provide recharge to the ground using shallow wells linked to a simple trench or conduit recharge system. This can ensure that the water table is maintained at ground level if required.

7.0.16. Distributed drawdown to ameliorate water table levels. To ameliorate the water table over an extended area, it is feasible to discharge the abstracted groundwater over a general wide area. This will allow the groundwater to infiltrate into the ground, especially where the water table has been reduced and where the ground is more receptive to percolation to the water table. Clearly, if there is a cohesive impermeable layer at the ground surface, a distributed recharge is less effective. Under these conditions a more intrusive system of recharge is required such as shallow wells or trenches.

7.0.17. To the southwest of the M74 the water bearing aquifer is generally unconfined, that is having a water table or "phreatic surface". As such, this aquifer can be recharged with a shallow well system or a trench recharge system if appropriate.

7.0.18 To the northeast of the M74, the main water bearing aquifer is the rock which is under artesian conditions. At this location it is more technically challenging to maintain the artesian conditions by artificial recharge.

However if the water table in the upper gravel is perched above a low permeable layer, it is likely to be recharged from surface run off, or near surface flow. Under these conditions it is unlikely that a lowering of the artesian head in the rock aquifer will have any significant impact on the water table in the upper gravel.

8. Other Issues

8.0.0. It is not recommended to decommission the ground investigation wells at this time. In view of the considerable investment in the installation of these wells, it is recommended that these wells are maintained for future groundwater testing, dewatering trials or as contributing to the full scale dewatering operation.

8.0.1. Should the wells be decommissioned now, they will need to be backfilled with grout or bentonite. During the decommissioning process SEPA guidelines for decommissioning wells need to be followed.

8.0.2. Should the wells not be decommissioned, it is recommended that a programme of maintenance be developed, which includes the long term monitoring of groundwater behaviour. Also required is the documentation of the required eventual decommissioning plan.

8.0.3. If the wells are to be retained for future use or monitoring purposes the wells will need to be capped with sealable removable well heads to prevent water from bleeding out of the top of the well casing, if the piezometric surface is above the ground level. Sealing the wells with a well head will also prevent debris from entering the well.

8.0.4. The wells will also need to be monitored on a regular basis to check for a build up of silt. If silt is building up in any particular well, the well should be pumped from on a regular basis to remove the silt build up.

8.0.5. It may be that not all the wells will need to be retained for future use or monitoring purposes and these can be sealed as described 8.0.1.

8.0.6. During construction of the base slab, a procedure needs to be in place to provide a seal of the dewatering wells following the curing of the concrete base slab.

8.0.7. Such a procedure can comprise the sealing of the annulus of each well over an appropriate length of the unslotted well casing. Over the depth of the concrete base slab, a well casing can be fitted which is designed to allow the temporary sealing with a screw-in plug to prevent the inflow of groundwater under pressure. Once this is plugged, cement grout can be poured into a hole in the slab with a bentonite waterstop installed to prevent any leakage through the concrete plug.

8.0.8. At the northeast end of the site, depressurisation wells may be installed either on the inside or outside of the excavation footprint depending on the required target piezometric level in the underlying rock.

9. Discussion

9.0.0. OGI has been engaged by the Mouchel-Fairhurst Joint Venture on behalf of Transport Scotland, to assess the feasibility of dewatering required for the construction of the new road underpass at the M74, Junction 5, Raith.

9.0.1. As part of the assessment MFJV has undertaken a comprehensive ground and groundwater investigation to determine the geotechnical and groundwater conditions at the site.

9.0.2. Of particular significance to the dewatering assessment is the 90 investigation boreholes that have been installed with either observation piezometers or with well casings suitable for groundwater pumping.

9.0.3. Groundwater pumping tests have been conducted from a series of pumping wells, with observed drawdowns measured in surrounding boreholes and piezometers.

9.0.4. To date, the field pumping tests have demonstrated that the highest transmissivity* occurs within the gravel aquifer at the southeast of the site, i.e. in the area of borehole BH217. The rock layer present beneath the gravel appears to have a low transmissivity based on the results of the pumping tests in the underlying rock.

> * The transmissivity is the average permeability of an aquifer multiplied by the effective thickness of the same aquifer. The groundwater abstraction rate required to lower the water table or piezometric surface in the aquifer is directly proportional to the transmissivity. In summary, the higher the transmissivity, the higher the required pumping rate.

9.0.5. To the northeast of the site a distinctly different picture was observed. The rock aquifer was measured as having the dominant transmissivity, with the overlying alluvial deposits having a low transmissivity.

9.0.6. Of major significance, the artesian pressure in the underlying rock aquifer was measured at approximately 26m OD, this being in the order of 4m above ground level. This is known as "flowing" artesian conditions.

9.0.7. The transmissivity of this rock layer in the northeast of the site is generally lower than that measured in the gravel aquifer to the south. This is most likely as a result of the main flow through the rock being through the upper weathered and fractured zone. Whilst the rock is possibly more permeable than the sand, as transmissivity is defined as permeability x effective aquifer saturated thickness, if the rock has a thin fractured zone, it has a lower transmissivity.

9.0.8. The first stage lowering of the piezometric pressure is required for the installation of the diaphragm or secant pile walls. This is required to remove the excess high artesian pressure that could potentially displace the grout during wall construction. Because the grout has a higher density than water, the complete removal of the artesian pressure is not normally required for diaphragm or secant pile wall construction.

9.0.9. As a consequence, it is unlikely that any first stage lowering of the water table is required at the southwest end of the underpass excavation, leaving artesian pressure reduction only required at the northeast end of the site.

9.0.10. The artesian pressure reduction at the northeast end of the site can be achieved by the installation of groundwater abstraction wells either inside or outside the underpass footprint. In either case it is recommended that the wells are positioned so that the same dewatering wells can be used to abstract groundwater for the second stage dewatering required for the excavation of the ground to formation levels.

9.0.11. For the second stage dewatering to excavation formation levels, based on the information gathered from the groundwater investigation and testing, the recommended dewatering strategy is different at the two ends of the underpass.

9.0.12. At the southwest end it is necessary to lower the water table in the exposed sand/gravel strata to the excavation formation level within the diaphragm/secant piled cut-off walls. If the water table is not lowered, the exposed sand will “boil” resulting in soil instability.

9.0.13. OGI recommends installing dewatering wells into the sand aquifer to a sufficient depth and frequency to ensure stable conditions. At this point in time, OGI does not consider it necessary to install boreholes into the rock aquifer at the southwest end of the excavation where the overlying sand and gravel layers are permeable.

9.0.14. The current underpass construction design has cut-off walls partially penetrating the sand-gravel layer. This partial penetration reduces the required groundwater abstraction rate to reach the target drawdown and so reduce the potential impact on the water table outside the excavation. To make use of this cut-off wall, the ends of the underpass, i.e. the 32m wide south opening, may also need a cut-off wall of similar depth.

9.0.15. At the northeast end of the site, a different dewatering strategy is recommended. MFJV drawing No. 53213/002 shows that the alluvial deposits in the northeast contain horizons that are of a higher clay-silt content and as such probably will have a significantly lower permeability. As a consequence, OGI considers that only limited dewatering of the alluvial deposits is required, such as the drainage of surface water, or some shallow wells to intercept vertical seepage of groundwater into the excavation.

9.0.16. This limited dewatering within the upper alluvium at the northeast is only possible if the excess artesian pressure, as was observed in BH 270, is removed from within the underlying rock.

9.0.17. Because of the depth of the rock aquifer below the underpass excavation, it not necessary to lower the piezometric water table to the same level as the excavation formation. Instead, the artesian pressure is only required to be reduced to a level such that a sufficiently high factor of safety against ground heave is achieved.

9.0.18 Assessing the appropriate safe artesian pressure is critical for the following reasons:

(i) The artesian pressure beneath the excavation needs to be reduced to a sufficiently low level to prevent ground heave.

(ii) If the artesian pressure is lowered excessively, there will be a consequential excessive drawdown outside the excavation. Therefore the higher the artesian pressure can remain beneath the excavation, the less the impact will be outside the excavation.

(iii) If the artesian pressure is lowered excessively, a large groundwater abstraction rate is required. As a consequence, if the artesian pressure is lowered just enough to ensure ground stability, this approach will minimise the abstraction rate of groundwater.

(iv) Following the same reasoning as (iii) above, if less groundwater is abstracted, less treatment of potentially contaminated groundwater will be required.

9.0.19 In terms of impacts caused by the lowering of the water table, amelioration of drawdown can be considered as follows:

(i) Some recharge to the sand aquifer is feasible should the recharge zone be at an appropriate distance from the excavation. Note that water recharge close to the excavation is not recommended.

(ii) Some recharge to the rock aquifer at distance from the excavation is feasible only to maintain piezometric surface at ground level. It is not practical to consider recharging the ground above hydrostatic pressure to maintain an elevated artesian head above ground level.

(iii) Distributed surface recharge to the ground at key locations. This will reduce the impact on the pore water pressure by changing the boundary conditions at the ground surface. This surface recharge can be enhanced with a shallow well or trench system to ensure infiltration of the recharge water to the ground.

9.0.21 The required groundwater abstraction rate will depend on a number of factors. The key factors are as follows:

(i) The regional transmissivity of the ground strata surrounding the site

(ii) The target water table/artesian levels below the excavation

(iii) The depth that the surrounding cut-off walls penetrate the aquifer

(iv) The water level of the River Clyde

(v) The intensity of percolation of surface water into the gravel aquifers

(vi) The storage coefficient of the ground

9.0.22 The groundwater abstraction rates required to lower the piezometric head to below the excavation formation have been calculated to be in the range of 52 – 148 litres /second after a 30 day pumping period, based on a range of uniform transmissivity of 150 - 600 m^2/day, a storage coefficient of 0.2, together with a full connection of the River Clyde with the underlying gravel aquifer.

9.0.23 However, whilst a higher transmissivity will give rise to a wider cone of depression of water table, this wider impact is only partly sensitive to transmissivity. This is because the lowering of the water table is predominantly governed by the required drawdown beneath the excavation together with the initial water table level and the water level in the River Clyde and other surrounding other water features.

9.0.24 This particular hydrogeological problem is known as a boundary value problem. That is the problem is governed by set levels, not by set flows such as a water supply requirement where pumping rates are specified.

9.0.25 This magnitude of groundwater abstraction required to lower the water table and artesian beneath the excavation to the required levels, is well within the capacity of dewatering systems that OGI has previously designed and installed. Examples of relevant dewatering operations that demonstrate this can be achieved safely in practice include the following.

OGI has designed and installed a pressure relief system to lower the piezometric surface by over 25m, as required for the construction of the London Millennium Bridge.

Also in London, OGI was responsible for the dewatering along the A13 Widening for a road underpass and numerous pipelines. Water table was lowered by up to 7 metres without disruption of traffic flow.

Furthermore, OGI designed and installed a system required to abstract groundwater at a rate of over 1000 litres/second, as required for the construction of a water storage tank at Forfar for Scottish Water.

9.0.26 It is OGI's opinion that the impact on the nearby water features can be ameliorated by recharge to the ponds of the abstracted water. The groundwater is abstracted at source and can be pumped to the various water features to maintain the water levels at the required levels, under the conditions that the pumped water is of sufficient quality.

9.0.27 It is OGI's opinion that there can be further substantial benefits to the project made by limiting the extent of artesian pressure reductions in the rock aquifer at the northern end of the underpass construction.

These benefits include (i) the reduction in the abstraction rate of groundwater, together with (ii) the reduction in the extent of surrounding groundwater drawdown influence.

However, the residual artesian pressure does need to be lower than the downward force from the overlying ground throughout the entire soil mass between the excavation formation and the rock aquifer.

9.0.28 The optimum level of dewatering and artesian pressure reduction can be assessed by more detailed design using a more powerful computer model than used for the current assessment. It is recommended that such a model be applied in conjunction with the structural design during the detailed design stage.

9.0.29 If a more powerful model is applied during the detailed structural design, this can be used to assess the impact of installing drainage below the road structure in order to assess the benefits of equalising the artesian pressures. The benefit of this approach is to permanently reduce the high artesian pressures beneath the structure by releasing these pressures into the lower pressure aquifer on the southwest of the site.

9.0.30 Throughout the report, OGI has stated clearly that the dewatering of the structure is feasible based on the current conditions encountered during the site investigation period. However, the challenge during the construction period is to minimise the level that water table or artesian head is reduced. This has the benefit of (i) reducing the volumes abstracted, (ii) reducing the potential water volumes that need treatment, (iii) reducing the wider impact of drawdown outside the excavation and (iv) reducing the amount of artificial recharge required to maintain a sufficiently high external water table and artesian head.

10 Conclusions

10.0 Groundwater Assessment

10.0.1 This assessment is based on a comprehensive programme of ground and groundwater investigation which included rigorous field testing to establish the groundwater behaviour in the vicinity of the proposed underpass. It is considered that the scope of investigations, and the quality of the findings are sufficient to enable competent D&B construction contractors to design a practical dewatering scheme which will allow the envisaged construction methods to be employed.

10.0.2 The following potential impacts of dewatering have been considered and are addressed in turn:

- *risks to construction*
- *risks to permanent works*
- *risks to surrounding property*
- *risks to groundwater regime*
- *risks to surface water environment*

10.0.3 Construction risks are concerned with a) installation of cut-off piles and walls, and b) construction of an effective base slab. The first of these is considered to be relatively straightforward. This will require a dewatering arrangement local to the underpass which will reduce piezometric heads to near ground level and so allow conventional pile/wall construction techniques to be utilised.

10.0.4 Because only a limited number of dewatering boreholes are required for this operation, it will be possible to use these boreholes to further test the groundwater behaviour locally. This in turn will allow a refinement of the design and so avoid an over-design of the dewatering system.

10.0.5 The construction of the base slab will require more comprehensive dewatering measures to draw down groundwater levels sufficiently to remove the threat of base heave and to achieve stable working conditions. It has been demonstrated in this assessment that such an operation is well within the capability of a specialist dewatering contractor. Typical well arrangements have been suggested within the report.

10.0.6 Differing dewatering techniques are suggested for the northeast and southwest sections of the underpass because of the distinctly different ground conditions. If necessary, groundwater lowering via the Sands & Gravels in the southwest may be assisted by sacrificial, slurry cross-walls and/or by extending the underpass walls to rockhead.

10.0.7 Artesian conditions in the northeast may be dealt with most effectively by limited pumping from the bedrock (to reduce the artesian heads to a safe level). To ensure stability of the silt/clay strata above the rock to the ground surface, pumping from the rock can be supplemented by slender vertical pressure relief wells through the soft clay layer to dissipate the uplift pressures on the underside of the clay and completely remove the risk of heave of the excavation base.

10.0.8 In the permanent condition the slender pressure relief drainage wells could be linked to a horizontal drainage blanket beneath the slab and thus cater for any uncertainties in long-term groundwater behaviour.

10.0.9 In all the above cases, an observational approach to the construction is advocated whereby information gathered during the installation of the system is used to optimise the final system installed.

10.0.10 On completion of the permanent works and cessation of pumping, groundwater levels will gradually return to normal. Provided the works are appropriately designed and constructed to resist uplift pressures, and to be watertight, then no ongoing groundwater control measures are considered necessary.

10.0.11 There is a clear potential risk to adjacent property through the implementation of the necessary dewatering scheme. The form of the resulting cone of depression means that the risk is greatest near to the pumping wells (i.e. to the road infrastructure around Junction 5), but somewhat less at the distance of the closest property (i.e. the restaurant in Strathclyde Park). Expected drawdown curves have been presented within this report and these illustrate that drawdowns of 6m beneath the M74 and 4m beneath the restaurant are possible.

10.0.12 Based on these predicted drawdowns, the resulting settlements are estimated in the Geotechnical Interpretative Report.

10.0.13 Conventional measures of mitigating the effects of drawdown on ground consolidation have been suggested herein and these include:

(i) Groundwater recharge via recharge wells,
(ii) Shallow wells surrounding specific buildings, and
(ii) Distributed drawdown to maintain water table levels.

These measures have the ability to ameliorate the ambient water table in the upper gravel layers surrounding critical buildings where there is an impermeable layer above the rock as is the case in the northeast. Where the aquifer is phreatic, i.e. there is a water table, recharging the ground with wells or trenches is more effective.

10.0.14 Any risk of detrimental permanent impact on the groundwater regime itself is considered to be insignificant. It is a dynamic system fed by a substantial catchment and the effects of dewatering are expected to be short-term and completely recoverable.

10.0.15 Risks to the surface water environment are a primary concern given the sensitive nature of the local ecology. This assessment has concluded that these risks are extremely low given the manner in which the surface water bodies are fed, and because proposals are made for returning abstracted water to the ponds to ensure a balance is maintained. Clearly these proposals are dependent on an acceptable quality of returning water and this aspect is addressed in the groundwater chemistry section of the Interpretative Contamination Report.

10.1 Residual Risks and Recommendations

10.1.0 Whilst it is considered that a practical dewatering scheme can be designed and implemented to effect safe and reliable construction, it is emphasised that success is fundamentally dependent upon the experience and capability of the construction team. Careful selection of the contractor is paramount.

10.1.1 Other than 10.1.0 above, the principal perceived residual risks to the feasibility of a safe and effective dewatering scheme are:

- *Potential disruption to dewatering due to unexpected flooding*
- *Unusually long periods of rainfall and resulting high groundwater pressures*
- *Long-term groundwater behaviour; observations to date are only a snapshot and the regional hydrogeology could change; potential minewater influences in particular are not understood.*
- *Highly variable natural ground conditions*
- *Unexpected man-made features within the ground*
- *Unforeseen contamination*

10.1.2 A substantial investment has been made in the ground and groundwater investigations to date and an extensive array of groundwater pumping and monitoring wells remain in the ground. These present both an opportunity (in terms of further observation, testing and later construction) and a liability (in terms of possible future leakage and resulting ground loss). Decommissioning costs will also be substantial and for this reason alone it may be preferred to include this process within the construction contract. If this is the case then it is recommended that a regular inspection programme be established to ensure prompt remedial action in the event of any deterioration.

10.1.3 Initiating a regular groundwater monitoring programme is recommended to provide a longer-term picture of the conditions.

10.1.4 Consideration should be given to retaining selected wells for future testing at, or prior to, construction tender stage in order to optimise the dewatering design.

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