Design for the Hyporheic Zone Dipole Setup

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Abstract

The hyporheic zone has been a much studied area of hydrogeology for a number of years. A research experiment is proposed over the coming years, which will allow the hyporheic mixing zone to be assessed through the use of chemical tracers. An artificial hydraulic gradient is required to generate a downward gradient that will generate a mixing zone within the riverbed sediments.

This work has developed a 'Dipole' well design that included abstracting and injecting groundwater adjacent to a stretch of river; thus generating the conditions that will facilitate the future experimental research. Optimum well positioning and operation were found to be the keys to the success of the future research.

A complete characterisation of the proposed river section was conducted that allowed the development of a numerical model using Modflow, to simulate the likely affects to the aquifer and river systems, by implementing such a well design.

Discharge rates of 1 – 4 ls^{-1} were required to generate flow reversals within a typically gaining section of river, for wells positioned at 30 – 50 m spacing.

The success of the research experiment will depend on the accurate implementation of the well design. Many of the problems associated with applying the Dipole experiment are explored in this study.

Acknowledgements

I would like to dedicate this thesis to my adorable son Jack whom I will always love.

I would like to thank my lecturers, particularly my supervisor during this project Rae Mackay for his guidance. I would like to thank Richard Greswell for his endless generosity when it comes to his time and effort with both practical and technical problems. I would also like to thank my friend and colleague Derek Conran who persevered through this experience alongside me.

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1. Chapter 1

1.1. Project Outline

This study represents the early stages of the SWITCH project which aims to improve on the growing conceptual understanding of the Hyporheic zone and its potential to naturally attenuate pollutants, and to support those theories through experimental work.

The SWITCH project has been brought about to encourage a shift in urban water management to develop sustainable, healthy and safe urban water systems.

The work will be conducted by a consortium of 32 organizations from 13 countries under coordination of UNESCO-IHE (Institute for Water Education) in partnership with the European Union. The work will run for the next three years.

The Hyporheic zone is a critical interface between groundwater and surface water environments and is a dynamic ecotone with important geochemical and microbial properties which may provide opportunities for pollutant attenuation, with a resultant reduction on the impacts of polluted groundwater on river environments, or vice versa.

This study is involves the design of three low discharge abstraction wells that require positioning adjacent to the Tame to control the head distribution beneath the river. The wells will allow several pumping tests to be conducted over the subsequent three years. The tests will involve abstracting and injecting groundwater adjacent to a section of the River Tame to allow artificial mixing zones to be generated and to assess the hyporheic zone's potential for attenuating pollutants, through the use of chemical tracers. Initial characterisation of the aquifer and the river bed is essential to meet the long term goals.

The following criteria are required in order for the wells to efficiently generate the flow reversals required to run the experimental tests.

- Re-injection of the abstracted groundwater during the pumping tests is favoured over direct discharge to the river Tame, therefore the design requires a loop system or 'dipole' setup to minimise the impact on the overall experiment.
- Optimal well spacing will be imperative to maximise the area of river affected. The abstraction – injection well setup must impact a large enough section of river (approximately 30 - 40 m) to allow adequate assessment of the hyporheic zone and its potential to attenuate various chemicals during the testing period.
- For the initial assessment of the well setup a mixing zone of half a meter must be generated within the bed sediments, over pumping periods of varying test lengths.
- A third well is required at an alternative separation to the first two wells. This well will lend itself as optional pumping-injection arrangement.
- The pumping rate must be minimised while also maximising the area of river that will be impacted.
- The well screen must be positioned at an optimal depth to facilitate the preceding requirements.
- A reliable groundwater model should be used to validate the design.

1.2. The Project Aims and Objectives

The Aim of this work is to produce a reliable well design that will meet the requirements above. The following objectives must therefore be met:

- A desk study should be conducted to gain an understanding of the local geological, hydrogeological and hydrological conditions on site and in the surrounding area.
- Data collection from all relevant sources, relating to the study site should include reviewing previous wok in the area, both academic and professional.
- A detailed characterisation of the site and the River Tame should be conducted, so that the data collected can be used to construct a groundwater flow model.
- The model should test various well designs and find the optimum Dipole setup.
- The final design should be presented clearly, to allow future work to progress.

1.3. Approach and thesis layout

The objectives were met following a combination of field investigation, desk study and computer modelling. The study area is situated along a central portion of the River Tame as it flows through the industrial city of Birmingham, UK. The field investigation work was focused along a relatively straight 1.0 km reach of the river.

The River Tame rests on top of the unconfined Triassic sandstone rocks of the Birmingham aquifer, and has long been a source of water for both domestic and industrial supply, principally industrial.

Previous work by Ellis (2003) provided a good insight to some of the methodology used during the field data collection.

A literature review of the groundwater surface water interactions was necessary to gain an understanding of the long term project plans. The theory assessed provided a foundation for the whole project and aided in the conceptual understanding of the physical processes that control the mixing zone. (Chapter 2)

A comprehensive examination of all available archive data has provided an insight into the hydrology, hydrogeology and geology, on both a regional and local scale. (Chapter 3)

The archive information supplemented data collected during this work. Successful field investigation procedures were adopted from previous research and encompassed; occasionally modified during the fieldwork programme. (Ellis, (2003); Rivett, (1989); Thomas, (2001))

Complete accounts of the methodologies used during the site investigation and characterisation stage are presented. (Chapter 4)

Chapter 5 presents the results of the aquifer and river characterisation and summarises the conceptual understanding of the current interaction between the two systems

The well design criteria are individually dealt with in order to complete the conceptual requirements for the design setup. A numerical model of the design is tested using MODFLOW to allow the optimal Dipole setup to be established through a trial and error simulation approach. A transient model is used to establish the time required for the wells to reach steady state (Chapter 6).

The results of the Modelling and optimal Dipole setup is incorporated into the well design in Chapter 7.

Conclusions and future work (Chapter 8).

2. Chapter 2

This chapter looks at the present conceptual understanding of the physical controls on the Hyporheic zone and also reviews the measurement techniques used to assess the groundwater - surface water interactions (GW-SW). The UK legalisation driving research in this area is examined and presented in brief.

The latter half of this chapter focuses on the effects of pumping next to a stream and the procedures used to estimate stream depletion.

2.1. UK Legislation

The European Water Framework Directive (Council of Europe, 2000 Environment Agency, 2002) requires a more integrated approach to catchment management in order to achieve both surface and groundwater flow and quality objectives by 2015. A particular emphasis has been put on incorporating the two systems when assessing the interface between river and aquifer, namely the Hyporheic zone. The Environment Agency's reassessments of existing definitions of the Hyporheic zone (Smith, 2005) have defined it as:

The water saturated transitional zone between surface water and groundwater.

2.2. Literature review of groundwater surface water interaction

Groundwater surface water mixing zones are present in most natural fluvial channels.
The degree of mixing depends on physical factors such as the head distribution and groundwater flow directions, riverbed permeability, structure and ecology.
Recent attempts to integrate both groundwater and surface body systems as one hydrological system, has resulted in an increased focus on these areas.
Interactions generated at these interfaces, including exchanges of groundwater and surface water, directly adjacent to the river and through the channel bed resulting in the hyporheic zone.

The effects of topography, geology, and climate have significant controls on the physics of the hyporheic zone extent and degree of mixing.
Woessner (2000) states that the exchange of groundwater and surface water (GW – SW) in a fluvial setting is controlled by

- The distribution and magnitude of hydraulic conductivities, both within the channel and the associated alluvial-plain sediments.
- The relation of stream stage to the adjacent groundwater level.
- The geometry and position of the stream channel within the alluvial plain.

The rearrangement of groundwater flow lines within fluvial sediments as they approach the river channel, results in mixing of the aquifer and river waters, which normally display individual geochemical characteristics. Contrasting geochemistry's can then be used to estimate the depth of the mixing and hence evaluate the hyporheic zone.

Mixing can occur at several scales, from a few centimetres to many meters based on bed geometry and hydraulic-potential strengths (Winter et al., 1998).

Pressure potential differences at the stream bed caused by positive relief such as ripples dunes or boulders cause the surface water to enter the channel sediments at the head of a riffle sequences and exit at the riffle base in pools (Harvey and Bencala, 1993).

On the reach scale seepage can occur through the bed of streams and is commonly related to abrupt changes in the slope of the streambed or to meanders in the stream channel. This principle is illustrated bellow in Figure 1.

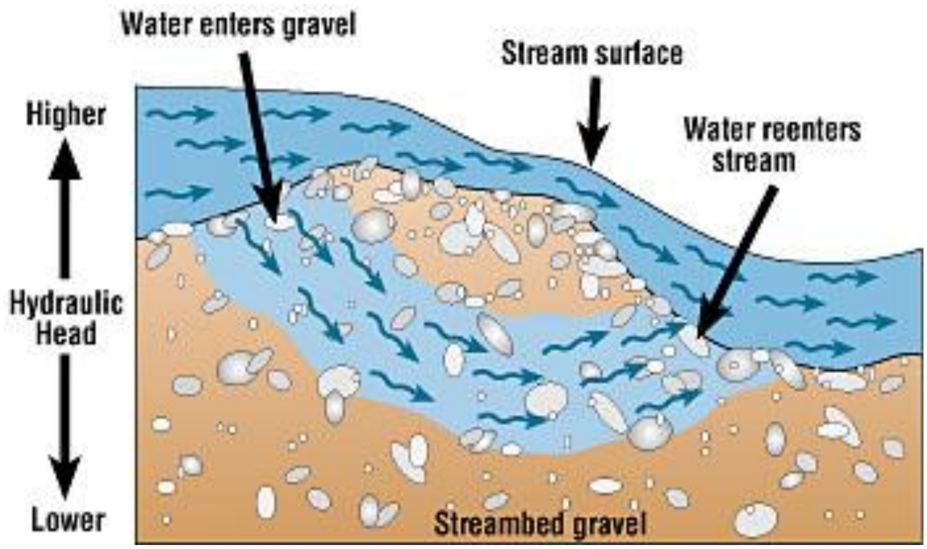

Figure 1: Hyporheic zone formed in a pool and riffle setting (University of Washington, water module, 1996)

By installing and taking measurements from riverbed piezometers, wells and piezometer nests, characterisation of the exchange between SW and GW can be achieved. This can be supported by conducting stream gauging at numerous cross-sectional intervals over a short period of time and comparing variations in discharge or recharge flux. Using these simple methods a river reach can be sub-divided into gaining, loosing, parallel or cross-flow sections the channel.

Figure 2 to Figure 5 represent the general conceptual understanding of the typical interactions that occur between streams and aquifer systems. The following are a combination of numerous

diagrams available in the literature (After: Woessner 2000, Nield et al., 1994, Spalding 1991, Sophocleous 2002, Townley et al., 1992)

Under conditions of low precipitation, Groundwater is said to be effluent to the river channel when the groundwater head at the channel interface is greater than the river stage. Base-flow in many streams constitutes a large proportion of the total discharge for most of the year.

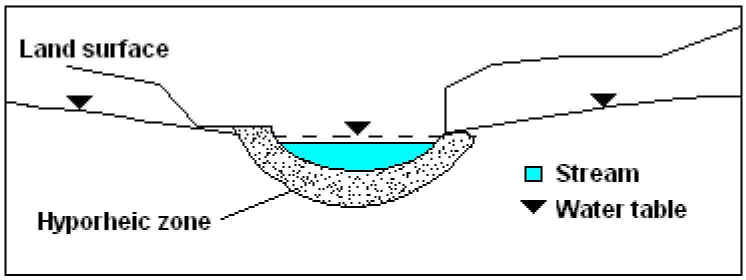

Figure 2: Groundwater is described as effluent

Equally, under conditions of increased precipitation, surface runoff and interflow gradually increase, leading to higher hydraulic pressures in the stream, which cause the river to change from effluent to influent condition, infiltrating its banks and recharging the aquifer.

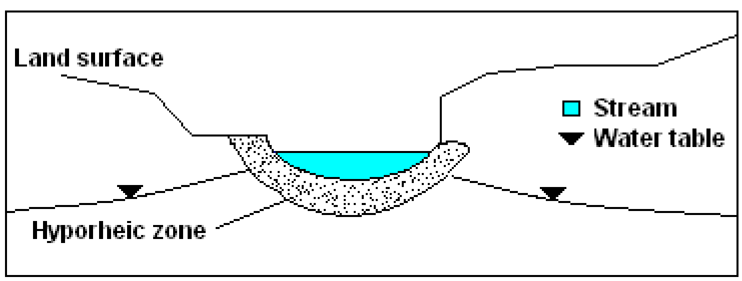

Figure 3: Influent conditions

Groundwater cross-flow may occur when the groundwater head is higher than the river stage on one bank but lower on the other (Townley et al., 1992). If groundwater flow takes place parallel to the river only partial exchange may occur.

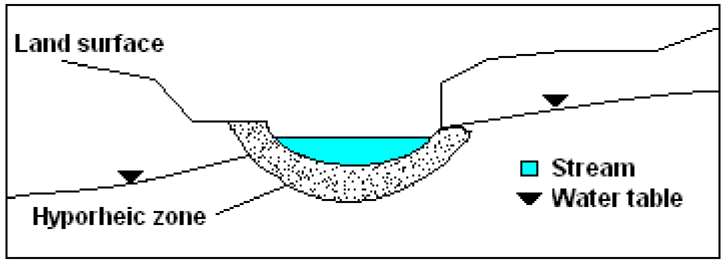

Figure 4: Cross-Flow section of river

During very arid conditions the water-table may drop. As a result it may be completely disconnected from the stream. During such conditions seepage from the stream occurs.

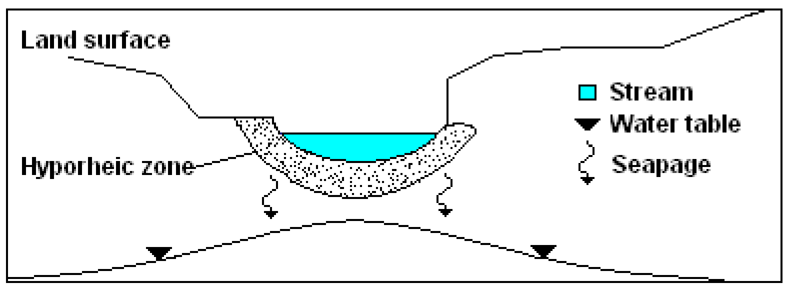

Figure 5: A disconnected section of river

Gordon et al. (1992) further classify streams into:

- Perennial streams
- Intermittent
- Ephemeral

In perennial streams, baseflow is more-or-less continuous, whereby these streams are mainly effluent and flow continuously throughout the year.

Intermittent streams receive water only at certain times of the year and are either influent (losing) or effluent (gaining), depending on the season.

In ephemeral streams the groundwater level is always beneath the channel, so they are always influent while flowing (Gordon et al. 1992).

During the course of this study a combination of flow gauging and head measurements are used to assess the exchange occurring at particular cross-sections along the reach. Difficulties can arise in interpreting such head measurements, as contradictory positive and negative, as well as neutral head differences can be observed over tens of meters of a river reach.

Woessner (2000) points out that the streambed topography and corresponding water exchange can cause localised flow systems within an overall gaining or loosing section of river.
It is also worth noting that variable flow regimes could alter the hydraulic conductivity of the sediment via erosion and deposition processes and thus affect the intensity of the GW–SW interactions. (Sophocleous, 2000)
In addition the hydraulic conductivity of the bed-sediments my become altered due to the precipitation of fine minerals, (sedimentation) as a result, clogging (colmation) can reduce the riverbed permeability (Brunke, 1999)*(See end of chapter note)
A combination of varying streambed geomorphology and colmation is likely to result in local heterogeneities.

2.3. Estimation of groundwater exchange

For hydraulically connected stream–aquifer systems, the resulting exchange flow is a function of the difference between the river stage and aquifer head.

This exchange mechanism is based on Darcy's law, where flow is a direct function of the hydraulic conductivity and head difference, can be expressed as

$$q = K\,(\Delta h)$$

Where

- $\Delta h = h_A - h_R$, (h_A is aquifer head, and h_R is river-head)
- q is flow between the river and the aquifer

[(+) For baseflow for gaining streams; and (–) for river recharge for losing streams]

- K is a constant representing the streambed leakage coefficient (hydraulic conductivity of the semi-impervious streambed stratum divided by its thickness).

2.4. Estimating Groundwater exchange through numerical modelling

The arrival of readily available computing power has led to an increase in the use of numerical models to simulate groundwater/surface water interactions.

Wroblicky et al. (1998) investigated the groundwater flow and seasonal variations in the hyporheic zone for two, first order mountain streams in the US, the Rio Calaveras and The Aspen Creek. The streams were in alluvial material with different hydraulic properties derived from bedrock consisting of welded tuff and sandstone. Modflow was used to simulate unconfined transient spatial variation in the magnitude and direction of stream-groundwater exchange.

Sensitivity analyses indicated that changes in the hydraulic conductivity of the alluvial and streambed sediments and variation in recharge rates have greatest impact on the magnitude, direction, and spatial distribution of stream/groundwater exchange.

Kasahara and Wondzell (2003) simulated Hyporheic exchange flows using MODFLOW and MODPATH to estimate relative effects of channel morphologic features on the extent of the hyporheic zone, on hyporheic exchange flow, and on the residence time of stream water in the hyporheic zone. The influence of pool-riffle sequences and channel shape was examined. Results confirmed that channel morphology significantly controlled exchange flows in most channels.

Most models use idealised stream geometry and the Darcy equation to simulate transfer of water across the streambed sediments. Groundwater discharge = hydraulic conductivity x hydraulic gradient x cross sectional area.

This may often be an oversimplification of the natural system.

The realistic simulation of transient surface water flows and groundwater flows across a region is complex. Surface water and groundwater flow equations may be coupled and solved simultaneously by iteration. However, more commonly the surface water heads are set (from field data or separate modelling) or river baseflow is determined as output from a groundwater model (Younger, 1989). The widely used groundwater flow model MODFLOW (Macdonald and Harbaugh, 1986) has been adapted to incorporate a 1D simulation of unsteady flow in open-channel networks, but still utilises the simple Darcy equation for water transfer.

Models have been used to evaluate the effect of groundwater pumping on stream flow for water resources and environmental impact purposes (Eberts and Blair, 1990).

Eberts and Blair, (1990) used Modflow (1989) to address the concerns in Columbus, Ohio regarding the vulnerability of a series of radial collector well from two nearby quarry dewatering wells and five uncontrolled land fills. The model was used to predict the effects of halting the dewatering wells to reconstruct the likely migration of lechate. The radial collector wells were drawing water from beneath rivers Scioto and Big Walnut Creek, resulting in a disconnected loosing stream. The model was used to assess the quantity of water that was being derived from the riverbed as apposed to the underlying aquifer.

2.5. Approaches used to assess the effects of pumping next to a stream

The effect of pumping from a well near a stream has long been of interest.

Pumping can lower groundwater levels and potentially reduce water flow within a stream.

Analytical solutions for estimating stream depletion from groundwater pumping are often unable to accurately predict complex interactions between groundwater and surface water.

Groundwater modelling has allowed these interactions to be analysed and solved iteratively.

This project aims to use a mathematical modelling approach to derive a design solution for the Dipole setup. The Groundwater Vistas model MODFLOW has been chosen because it allows seepage to the aquifer from the overlying river.

The following is a collected understanding of the effects of placing a well next to a stream. Both empirical methods and mathematical modelling have been used to predict and recreate the complex hydraulics of such conditions.

2.6. Analytical Approaches

Theis (1941) was the first to approach the problem of positioning wells next to streams; using an analytical solution, the Theis solution depicted the river as a long straight line, and was able to estimate the quantity of water derived from the river due to pumping, using the superposition principle. The solution included the assumptions that the stream and well completely penetrated a homogeneous aquifer, with zero drawdown. Groundwater was assumed to move horizontally, and vertical movement of groundwater was not included.

In the 1950s many authors focused their attention on the quantity of water coming from the river while pumping and defined the stream depletion as the ratio between this amount of water and the water extracted from the aquifer (qs/Q).

Theis' integral was later evaluated by Glover and Balmer (1954) using the complimentary error function (erfc). This solution is provided below in Equation 1.

$$\frac{\Delta Q}{Q_w} = erfc\left(\sqrt{\frac{S * l^2}{4 * T * t}}\right)$$

Equation 1: Glover and Balmer (1954) following Theis (1941) solution to stream depletion

Where:

- ΔQ [L^3/T] is stream depletion flow rate
- Q_w [L^3/T] is pumping flow rate
- t is time [T]
- T is aquifer transmissivity [L^2/T]
- S is aquifer storage coefficient [L^{-1}]
- *l* [L] is perpendicular distance from the well to the stream edge.

Their solution computed the stream depletion for a well located in a horizontal unconfined aquifer located near a fully penetrating stream with a highly permeable streambed (Figure 6: Glover and Balmer (1954) Conceptual aquifer - stream system analyzedFigure 6).

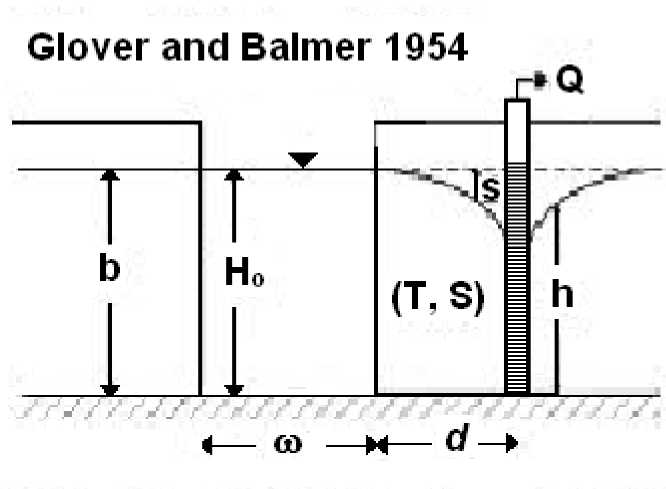

Figure 6: Glover and Balmer (1954) Conceptual aquifer - stream system analyzed

Hantush (1965) went a step further and provided an analytical solution for a fully penetrating stream and identical set of conditions considered by Theis (1941) and Glover & Balmer (1954) with the addition of a vertical layer of semi-permeable material lining the stream bed. This solution is provided below in Equation 2.

$$\frac{\Delta Q}{Q_w} = erfc\left(\sqrt{\frac{S*l^2}{4*T*t}}\right) - \exp\left(\frac{T*t}{S*L^2} + \frac{l}{L}\right) * erfc\left(\sqrt{\frac{T*t}{S*L^2}} + \sqrt{\frac{S*l^2}{4*T*t}}\right)$$

Equation 2: Hantush (1965) solution to stream depletion

Hunt (1999) proposed a similar solution to Hantush, additionally including the assumption that streambed penetration of the aquifer and dimensions of the streambed cross section are relatively small. This solution is provided below in Equation 3.

$$\frac{\Delta Q}{Q_w} = erfc\left(\sqrt{\frac{S*l^2}{4*T*t}}\right) - \exp\left(\frac{\lambda^2*t}{4*S*T} + \frac{\lambda*l}{2*T}\right)*erfc\left(\sqrt{\frac{\lambda^2*t}{4*S*T}} + \sqrt{\frac{S*l^2}{4*T*t}}\right)$$

Equation 3: Hunt (1999) solution to stream depletion

Where, λ (L/T) is a constant of proportionality between the seepage flow rate per unit distance (along the stream) through the stream bed and the difference between river and groundwater levels at the stream centre.

Hunt et al. (2001) carried out a field experiment to test the formulation in of the Hunt equation (1999). A pump test was conducted using a well located 55 m from the nearest edge of a long, straight portion of Doyleston Drain, New Zealand. The drain is 2.5 m wide with a silt and gravel lined stream bed approximately 1 m below the ground surface. The aquifer, composed of unconsolidated sand and gravel, is about 20 m thick and is capped on top with 2.8 m of less permeable material. Water was abstracted from the well at a constant rate of 0.0175 m3/s for a period of 10 hours. During this time, water levels were measured in nearby observation wells, while flow measurements were taken in the drain with the use of installed weirs. Values for T and S were estimated from observation well drawdown at early times, and λ was estimated using measured stream depletion flows at later times. Reasonable agreement was obtained for values of these parameters from data measured in four observation wells. The authors point out that these methods for determining parameters are dependent upon accurate measurements of flow in the river. While this criterion is well suited to small channels, it would be difficult to apply these methods at larger streams.

Jenkins (1968) later used an adaptation of Glover and Blamer's equation for a fully penetrating stream with a low conductivity streambed (Figure 7), to develop a set of graphical curves that could be used by practitioners to analyze water rights problems without the use of complex mathematical functions. Example calculations demonstrating how the graphical curves are applied in different situations are also described in Jenkins (1970). The graphical method makes the following assumptions: 1) Transmissivity does not change with time (drawdown is negligible compared to saturated thickness), 2) temperature of groundwater and stream water are equal and constant, 3) the aquifer is isotropic, homogeneous, and semi-infinite in aerial extent, 4) the stream is straight and fully penetrates the aquifer, 5) water is released instantaneously from storage, 6) the well is open to the full saturated thickness of the aquifer, and 7) pumping rate is steady during pumping.

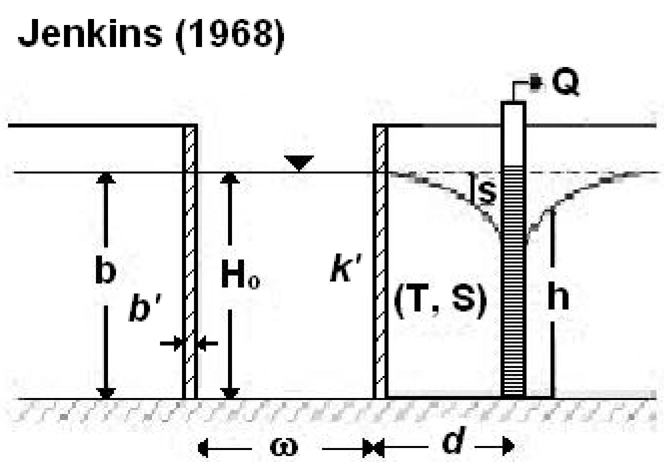

Figure 7: Jenkins (1968) Conceptual aquifer - stream system analyzed

Grigoryev (1957) and Bochever (1966) developed a steady-state model of stream-aquifer interactions that includes a simplified representation of a partially penetrating stream. The model is based on the assumptions that the river penetrates only a small proportion of the full aquifer

thickness and that the stream and aquifer are separated by a thin zone of relatively low hydraulic conductivity (Figure 8)

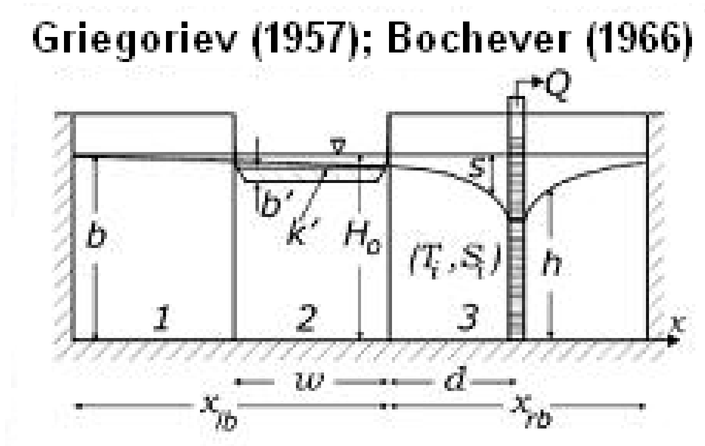

Figure 8: Griegoriev (1957) Conceptual aquifer - stream system analyzed

Wilson (1993) analyzed stream depletion for a fully penetrating stream fed by a unit inflow (q_u) under steady-state conditions (Figure 9), obtaining an equation giving the exact value of the stream depletion.

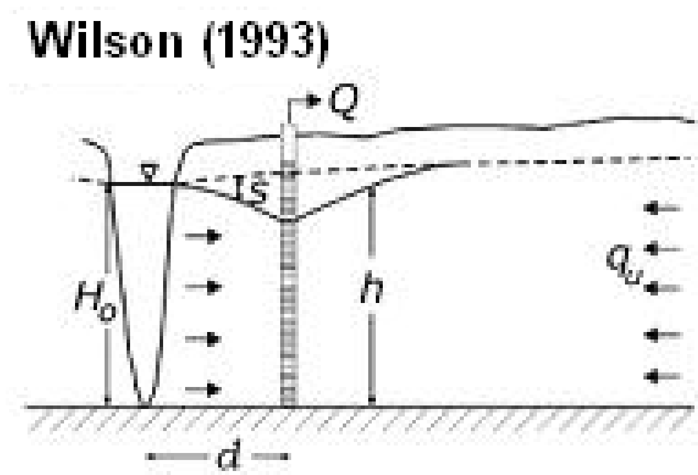

Figure 9: Wilson (1993) Conceptual aquifer - stream system analyzed

Sophocleous et al. (1995) tested the solution originally proposed by Theis (and updated by Balmer and Glover (1954), Jenkins (1970)) by comparing results from the analytical solution to results generated using numerical simulations with MODFLOW. Assumptions regarding local

aquifer homogeneity were problematic and stream bed clogging was a major factor in leakage calculations (Sophocleous et al., 1995). The assumption of full penetration of a stream into an aquifer was also a major factor, and the degree of partial penetration of the stream into the aquifer could dramatically change leakage estimates. Large scale aquifer heterogeneity was significant, and the analytical solution was found to be risky for layered systems. In contrast, estimates of aquifer properties such as storativity and hydraulic conductivity, and assumptions regarding partial or full penetration of the well were less sensitive to stream leakage calculations.

Butler et al. (2001) made an important contribution to the issue by analyzing a multilayer hydrogeological system connected to a river with a low hydraulic conductivity streambed and different thicknesses (Figure 10)

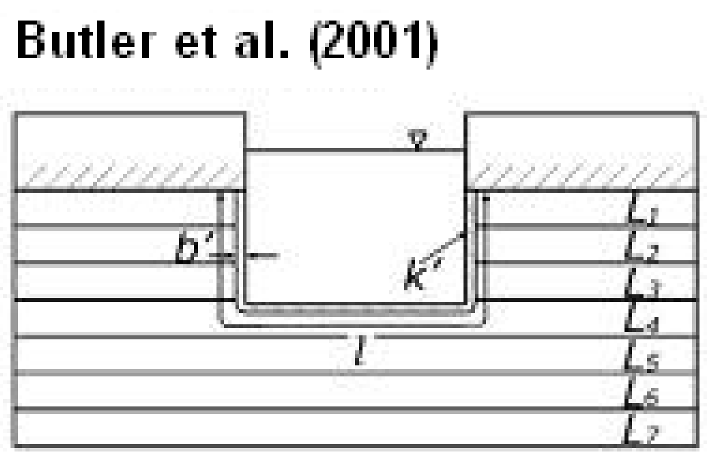

Figure 10: Butler (2001) Conceptual aquifer - stream system analyzed

Matteo and Dragoni (2005) estimated stream depletion under steady-state conditions for a variety of hydrogeological systems. A finite differences model was used to analyze several hydrogeological situations, and for each of these, the stream depletion was estimated using an advective transport method. They were able to derive an empirical equation that allowed stream depletion to be estimated, for the case of a stream that partially penetrates the aquifer and a pumping well that is screened over a portion of the aquifer. They found that their equation was valid for both isotropic and anisotropic conditions.

The derived equation, expresses stream depletion as a function of the unit inflow to the river, the discharge of the pumping well, the well screen length, the distance between the river and pumping well, the wetted perimeter, and a new parameter called ‘‘overlap,’’ which is defined to be the distance between the riverbed and the top of well screen.

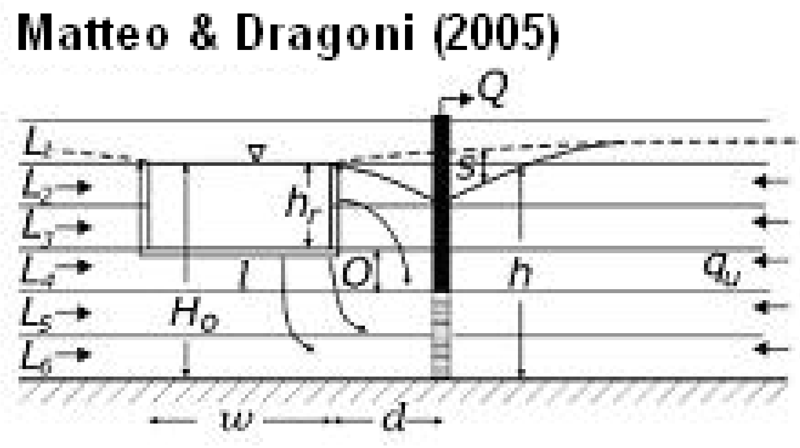

Figure 11: Matteo & Dragoni (2005) Conceptual aquifer - stream system analyzed

The overlap parameter makes it possible to consider indirectly the vertical component of flow, which is emphasized when the well is screened below the streambed. The formula is believed to be useful in deciding where to locate a pumping well and to decide the appropriate length of its screen.

2.7. Conclusions

Groundwater/surface water flow interactions are complex and are spatially variable between and within catchments. Further work is required to better understand the link between quality and flow processes. A great deal work has been done on assessing the effects of pumping next to a river. This review found that modelling allows the best means of predicting the effects of installing the Dipole pumping system adjacent to the river Tame. However the model is only as reliable as the amount and quality of the data available.

A systematic approach is therefore required during the data collection stage, to minimise inaccuracy in the data.

The key references identified in the review of groundwater-surface interactions were, Sophocleous, 2000, Eberts and Blair, 1990, Winter et al. (1998), EPA (2005), and Wroblicky et al. (1998)). A Useful case study of groundwater quality and flow interactions was Kasahara and Wondzell (2003).

*Note colmation can result from straightening, embankment works or canalization. In 2000 flood defence work was carried out along the River Tame. The works included realignment and restructuring of many of the embankments. It is important to consider the consequences of such action on the bed permeability. Such works could cause alterations in the discharge regime.

3. Chapter 3 The Study Setting

3.1. Introduction

The River Tame is an eastward flowing river which forms part of the larger drainage system of the River Trent and is the major river system in the West Midlands conurbation, which eventually discharges to the North Sea.

The unconfined Triassic sandstone aquifer underlies the study site and part of the city of Birmingham, dominating approximately 111 square kilometre area. The unconfined aquifer is roughly six to eight km wide and 18 km in length. (Thomas, 2001)

The 1 kilometre study reach lies between Rayhall and Water Orton, close to Witton, just upstream from where Hockley Brook and Witton Brook merge with the River Tame. See Figure 12.

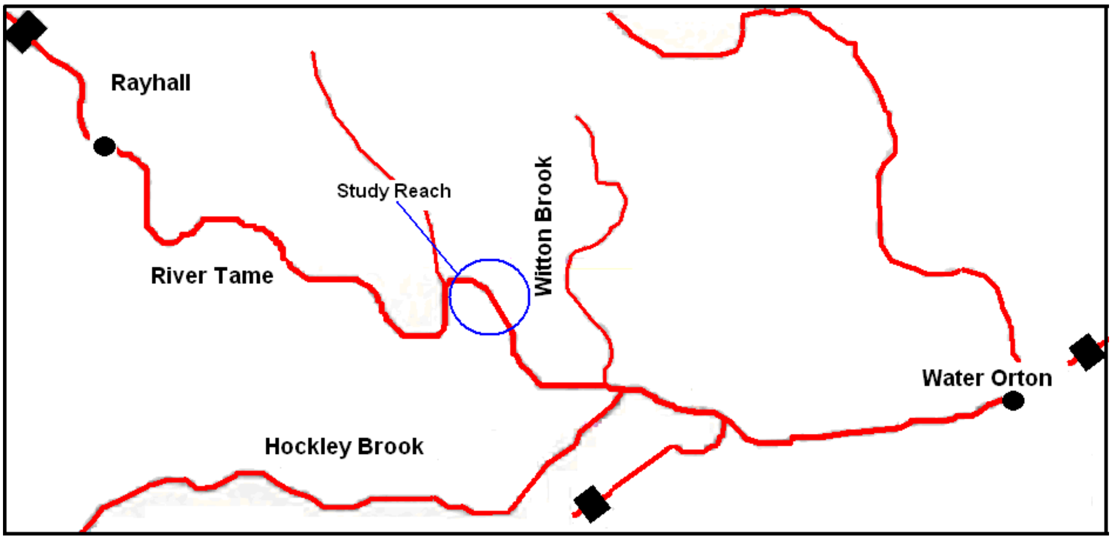

Figure 12: Rayhall and Water Orton with reference to the study site

The section of river selected for the proposed work was accessible from the Holford Industrial Estate, where negotiations are in currently in progress to obtain access to portions of the river that have been classified suitable locations for drilling rigs to enter. Further

requirements include a long term power supply in close proximity of the drilling site. The proposed power supply location is shown later on Figure 30.

The industrial estate satisfied both of these requirements.

3.2. Site Location

The Study area comprises a 1 km reach of the River Tame, which flows along the north of Birmingham city centre.

Access to the site was gained via Tameside drive, (Holford Industrial Estate) just off the A 4040 heading north of Witton, approximately 4.0 miles from Birmingham city centre (Figure 13 and Figure 14). A rough grid reference for the centre of the site is as follows: SP 078886.

Figure 13: Study location with reference to Birmingham City Centre, UK

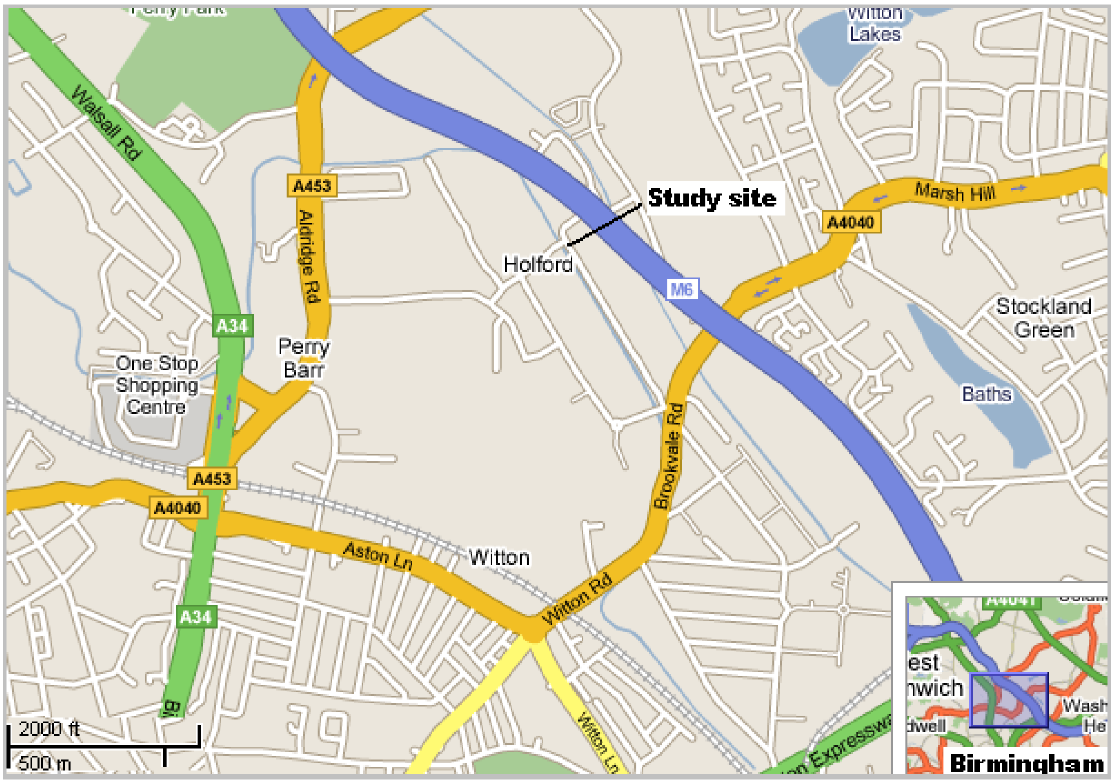

Figure 14: Study site location with reference to A4040 and Witton

3.3. Regional Geology

The study site is situated along the river Tame and is located roughly at the centre of the Knowle Basin; which consists of Mesozoic Triassic Sandstone, commonly known as the Sherwood Sandstone Group. The Knowle Basin is bound to its east and west by faults; the western and south western side of the unconfined aquifer unit is bounded by the Upper Carboniferous and strata of the Warwickshire Coal Field. The eastern and south eastern boundary of the unconfined aquifer is nearly straight and represented by the Birmingham Fault.

The Mesozoic Triassic strata are subdivided into the following units:

- Sherwood Sandstone Group
    - Wildmoor Sandstone Formation
    - Kidderminster and Hopwas Breccia Formation
    - Bromsgrove Sandstone Formation

The Triassic strata are distinguishable by their red colour, resultant from digenetic oxidation of detrital ferromagnesian silicates and iron bearing clay minerals to iron oxide. (Haematite) (Powell et al. 2000) They are predominantly continental, fluvial and lacustrine, the Triassic sediments were deposited during arid to semi arid conditions.

The Triassic Sherwood Sandstone has a gently undulating topography which is mirrored by a 0 to 5 m covering of superficial deposits, which follows the shape of the Triassic strata.

The Tame originates from a major watershed along the line of highest ground in the west, known broadly as the Russell's Fault. The river flows to the east for about seven kilometres, where it crosses the Birmingham Fault.

The Birmingham Fault is a major geological discontinuity which downthrows the Mercia Mudstone Group and marks the transition from the unconfined in the west to the confined Sandstone aquifer in the east.

The river Tame continues east across the Mercia Mudstone Group until it reaches Hams Hall, where the Tame takes a sudden turn northwards to eventually flow into the River Trent which discharges to the North Sea.

Information on the solid and drift geology was obtained from recent geological maps and memoirs (Geology of the Birmingham Area (Powell et al., 2000) of the British Geological Survey. (See Figure 15)

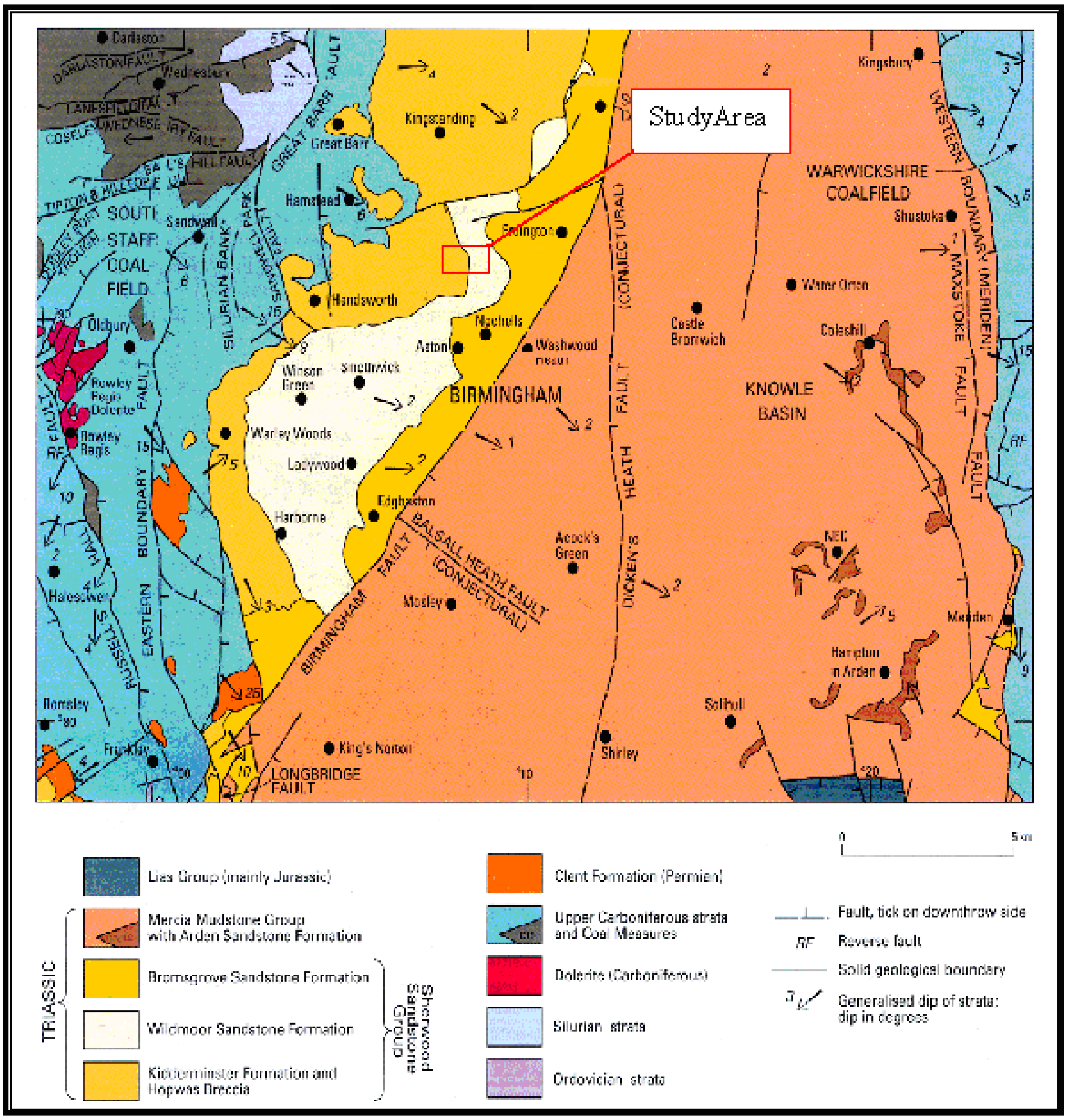

Figure 15: Geology of the Birmingham Area (Powell et al., 2000)

3.4. Site Geology

The study site is located approximately 1.0 kilometre NW of the Birmingham Fault. Historic borehole logs published by Butler and Lee (1943) from 11 deep abstraction sites within a 900 meter radius of the site show a variation in depths of the Triassic sandstone rocks of between 71 and 80m, with a median thickness of 76 m.

The Triassic Sherwood Sandstone Group display distinct lithological characteristics that allow stratigraphical division between the individual Triassic strata. (See Table 1)

Formation	Description
Kidderminster Formation	Mainly course to medium grained, pebbly, cross- bedded sandstone, with infrequent thin mudstone beds. The formation coarsens with depth frequently consisting well rounded pebble conglomerate, indicative of its fluvial deposition.
Wildmoor Sandstone Formation	Fine grained with occasional intercalations of mudstone.
Bromsgrove Sandstone Formation	Massive medium to course grained Sandstone with inter-bedded marls and occasional conglomeratic beds.

Table 1: lithological characteristics of the Sherwood Sandstone Formations

However problems arise at the contact between the base of the Sherwood Sandstone (Kidderminster Formation) and the overlying Wildmoor Formation.

The base of the Kidderminster Formation passes gradationally upwards through the finer Wildmoor Sandstone Formation and then the lesser persistent Bromsgrove Sandstone Formation.

Present British Geological maps (Powell, 2000) show the study site to rest on the Kidderminster formation, however the contact boundary between the Wildmoor and Kidderminster formation is inferred and borehole evidence in the area is more inclined to support the Wildmoor formation along the proposed drilling site, and will be discussed further in Chapter 5.

The base of the Sherwood Sandstone Group (Kidderminster formation) rests disconformably on the Hopwas Breccia which consists of yellow brown and red brown, medium to course grained pebbly sandstone and breccia of Palaeozoic origin reaching a thickness of 10 m.

The Kidderminster formation predominantly comprises of weakly cemented to friable, medium to coarse grained pebbly sandstones, showing a fining upwards sequence from the base of the formation which supports its depositional history; owing its origin to fluvial deposition by a north flowing braided river system. (Powell et al., 2000)

The Wildmoor Formation (Earlier nomenclature Upper Mottled Sandstone.)

The formation is typically orange-red often referred to as ‘foxy red’, fine grained, micacous soft sandstone with less frequent thin beds of mudstone and very rare pebbles. The lower boundary of the formation is gradational as it rests conformably on the Kidderminster Formation.

The fine grained morphology of the Wildmoor formation has been principally used to distinguish between the two formations. Grain size analysis shows 40 to 67 per cent medium sand and 28 to 46 per cent fine sand, with less than 2 per cent course sand. (Powell et al. 2000)

3.5. Drift cover

It is approximated that 75 per cent of the District is covered in superficial deposits. (Powell et al 2000) The study site is completely covered in till and sandy till. The drift cover is of a glacial and alluvial origin and shows a complex distribution reflective of the glacial and post-glacial periods of the Pleistocene and Recent times (Ellis, 2003, Thomas, 2001).

3.6. Drift Deposits

Much of the study area is covered in Quaternary glaciofluvial deposits (sand and gravel) that varies in thickness from 1 – 2 m. Melt water streams originating from the ice sheets deposited glaciofluvial sand and gravel as outwash deposits. The glaciofluvial deposits comprise primarily poorly sorted red, orange and yellow sand, clayey sand, pebbly sand and gravel (Thomas, 2001).

Detailed information regarding drift cover for the exact study area was not available. The site has had a long industrial history and much of the area has under gone redevelopment over the past decade and continues today. Flood Defence work 2000, in addition re-development work along the area surrounding the site has resulted large areas of made ground. Overall the drift cover in the district is complex and heterogeneous. The complexity and diverse nature of the drift cover is illustrated well by Thomas (2001) in Figure 16.

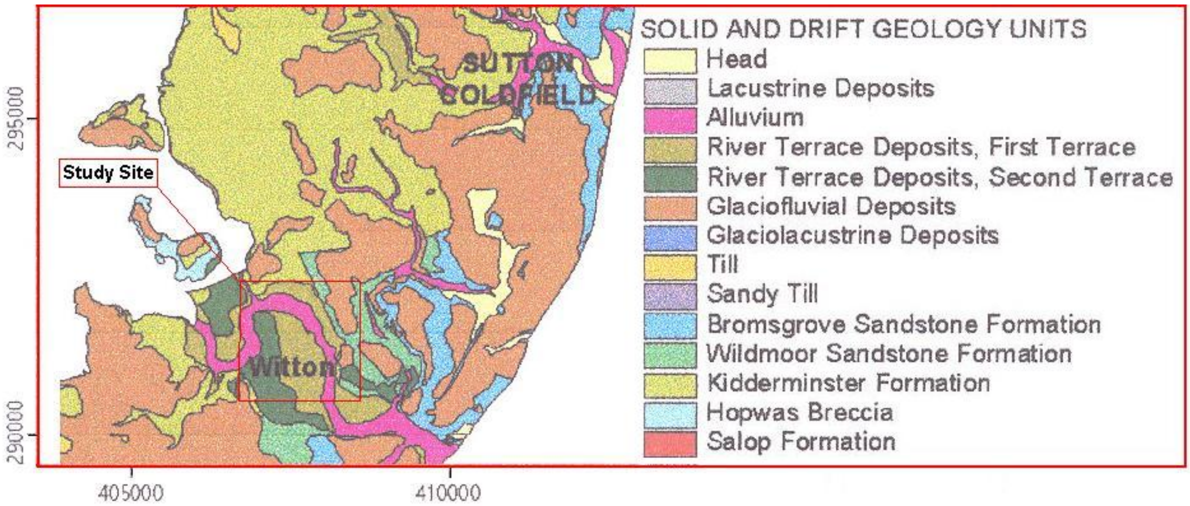

Figure 16: Drift and solid geology across the unconfined section of the Birmingham Aquifer. (Thomas, 2001)

3.7. Aquifer Recharge

The extensive drift cover is expected to control recharge to the aquifer system.

The average regional recharge to the sandstone aquifer is estimated to be about 0.36 mm/d.

Knipe et al., (1993)

Greswell (1992) calculated an average recharge estimate of 0. 45 mm/d based on the following relationship:

Recharge = (Effective rainfall x f_d x f_u) + (Urban return flows x f_d)

Where:

- **Effective rainfall** encompasses precipitation minus actual evapotranspiration

  $\mathbf{f_d}$ is a modification factor to account for the effect of drift which will reduce the amount of water reaching the aquifer.

- $\mathbf{f_u}$ is a modification factor for urban industrial cover which will divert water from reaching the aquifer.

- **Urban return flows** are losses from public water supplies and sewers which will augment natural recharge.

A comprehensive study into groundwater recharge and pollution to the unconfined Birmingham aquifer by Thomas (2001) found that recharge depths to the aquifer beneath the study site ranged from 0.167 cm to 0.347 cm during a 22 mm rainfall event.

3.8. Regional Hydrogeology

The principal groundwater source for the Birmingham region is the Sherwood Sandstone Aquifer.

The eastern edge of the unconfined aquifer is bounded by the Mercia Mudstone Group, which is generally considered impermeable, following work by Jackson and Lloyd, (1983) which highlighted distinct contrasts in levels of piezometric head across the Birmingham Fault.

The western and south western aquifer boundary is irregular and thins out as the Triassic sandstone rocks rest unconformably on top of the Carboniferous coal measures to the west.

The Permian Client Formation (Butler and Lee, 1943) is generally considered as the effective base of the Birmingham aquifer. Composed of red marl, sandstone, and breccia, the Client Formation is overlain by the laterally discontinuous Hopwas breccia (Probably of Triassic age also consists of angular blocks of quartzite in a sand or clay matrix). These two units are in hydraulic continuity with the overlying Sherwood Sandstone strata. (Powell et al., 2000)

Only limited hydraulic data is available for the unconfined Birmingham Aquifer. The Triassic sandstones are in general relatively low permeability/ high storativity aquifers: typical permeability and specific yield are of the order of 1m/d and 0.1, respectively. (Ford & Tellam, 1994)

3.9. Kidderminster Formation

This formation is generally very friable to weakly cemented, although locally harder calcite cemented beds are present. It is generally regarded as being the most permeable of the Sherwood Sandstone sequence.

The upper portion of the formation is dominated by upward fining sandstone cycles, red-brown and yellow-brown fine to course grained sandstone with occasional small pebbles and pebble lenses.

The formation has a median porosity of 29 per cent, and a hydraulic conductivity of 3.8 to 7.7 meters per day (m/d). Local cementation may reduce the porosity value closer to 20 percent and hydraulic conductivity closer to 0.6 m/d. (Powell et al., 2000)

Lovelock (1977) assigned median values of hydraulic conductivity and porosity to the Kidderminster Formation (Pebble Beds – earlier nomenclature) based on laboratory analysis as follows:

Formation	$K_{horozontal}$ m/d	$K_{vertical}$ m/d	Porosity (n)
Kidderminster	3.5	2.7	0.285

Table 2 : Hydraulic properties of the Kidderminster Formation of the Birmingham Aquifer, after Lovelock (1977).

Transmissivity values of the Kidderminster Formation range 60 – 600 m2/d (Rivett, 1988), and it is considered the most permeable member of the Triassic Sandstone sequence.

Taylor et al., (2003) document extensive fissuring of the Kidderminster Formation within a borehole drilled at a site in Witton, Birmingham between depths of 9 and 16 meters below ground level (mbgl.). An inter-granular transmissivity of 70 m^2/d is common, assuming an aquifer thickness of 20m.

Lovelock (1977) estimated the inter-granular transmissivity to be 268 m^2/d with a total transmissivity of 2771 m^2/d, suggesting fracture flow is prevalent.

According to Knipe et al., (1993) a value of 0.15 for specific yield is thought to be representative of the unconfined Sherwood Sandstone Formation.

Butler and Lee (1942) report thicknesses between 70 and 86 meters based on borehole data to the north west of the Birmingham Fault.

3.10. Wildmoor Formation

The Wildmoor Formation is characteristically fine grained, relatively friable sandstone. The formation is smaller in aerial extent compared to the Kidderminster formation. The formation can reach thickness of up to 120m, displaying thick beds of soft sandstone with rare thin beds of marl. (Powell et al., 2000)

The formation has a median porosity of 27 per cent, and a hydraulic conductivity of between 0.06 and 0.6 m/day. Powell et al. (2000) suggest a median horizontal hydraulic conductivity of 1 m/d and a vertical conductivity of 0.83 m/d.

Lovelock (1977) estimated values of permeability and porosity based on laboratory analysis of limited amount of material. The formation displayed porosity values of 25 per cent and a permeability range of 0.1 to 0.01 m/d.

Hydraulic data from pumping tests conducted by Severn Trent Water Authority was analysed by Lovelock and is displayed in Table 3 below.

Allen et al., (1997) suggest that the intergranular hydraulic conductivity values of this formation ranges from 3.1 X 10-4 to 12 m/d.

Formation	Transmissivity m^2/d	Hydraulic conductivity m/d	Analysis method used
Wildmoor/Kidderminster	500	7.8	Jacob
Wildmoor	130	1.2	Jacob

Table 3: Hydraulic data from Severn Trent Water Authority after Lovelock (1977)

3.11. Site Hydrogeology

The study site is situated on the unconfined portion of the aquifer. The site is entirely contained within the bounds of the Holford Industrial Estate off the A4040 Brookvale road, near Witton. Most of the hydrogeological interpretation of the underlying Triassic structure is based on available borehole data. Flow is typically from West to East but flows towards the River Tame along the study reach (Figure 17).

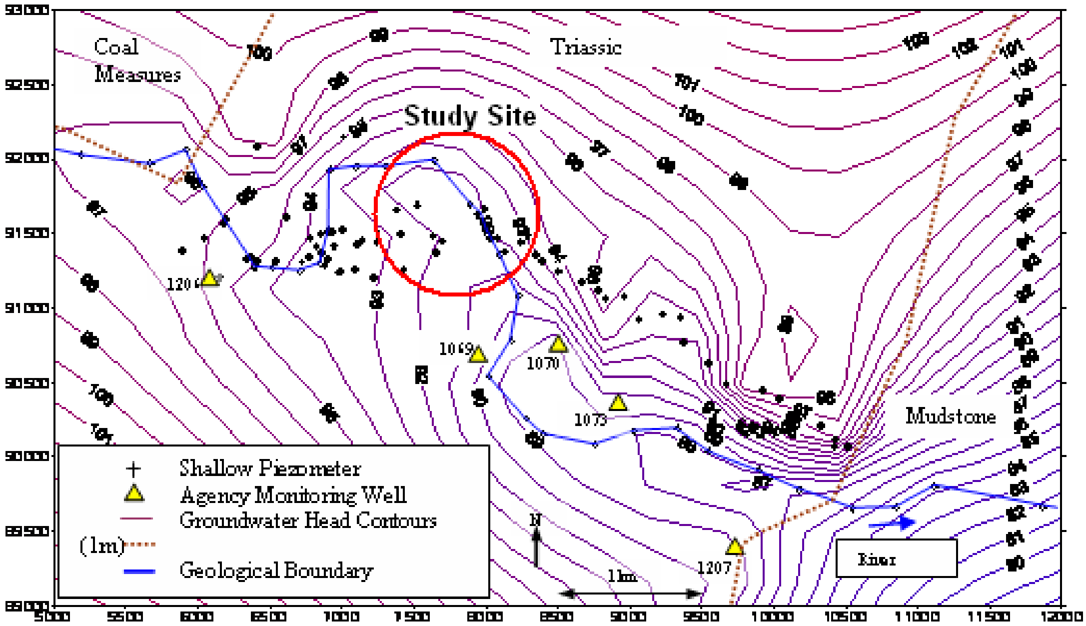

Figure 17: Groundwater Head distribution taken from Ellis 2003

Slug tests conducted by the Environment Agency following completion of boreholes P11 and P23 along Tameside drive were conducted at depths consistent with weathered sandstone. Both tests yielded a permeability value of 0.00004m/s (0.3 m/d). (See Appendix 3) Additional falling head tests conducted on the riverbank deposits immediately adjacent to the River Tame yielded conductivity values of between 4 and 6 m/d.

Stratigraphy	Lithology	KH/KV Ratio	KH (median) m/d	Sy	θ
Glacio-Fluvial Deposits	Sand and Gravel	1.54	20	0.24	0.33
River Terrace deposits (first terrace)	Sand and Gravel	4.26	20	0.24	0.33
Wildmoor Fm	Sandstone	1.88	1	0.13	0.27
Weathered Wildmoor Fm	Sandstone	1.88	1	0.13	0.27
Kidderminster Fm	Sandstone	1.81	3.5	0.19	0.29

Table 4: Literature hydraulic parameters for the modelled stratigraphy after Thomas, 2001,Allen et al., 1997

These hydraulic parameters are used as a first estimation during the modelling portion of this work.

3.12. Hydrology of the River Tame

The River Tame is an eastward flowing river which forms part of the larger drainage system of the River Trent and is the major river system in the West Midlands conurbation, which eventually discharges to the North Sea. The Tame rises to the east of a topographical high along a ridge, broadly known as the Russell's Fault**,** which represents a major watershed of central England.

The Tame originates from two principal source locations in the 'Black Country' towns of Oldbury, and Wolverhampton before joining at Bescot and flowing in an overall eastwardly direction to the north of Birmingham City centre.

Over much of the Tame's length, particularly in the urban areas, the river can be described as "greatly modified". In other words the river bears little resemblance to its original form in terms of its channel characteristics and/or its course.

The River Tame ranges from 7.7 – 11.4 m wide, 0 to 1.3 m deep with average dry weather flow velocities of 0.1-0.8 ms-1 along the study reach.

The Southern bank of the River has a gentle longitudinal slope of 0.0018 following the gradient of the river bed. The northern bank is more elevated, on average 3m above the river bed, and displays a more undulating topography. Elevations along north of the site vary between 95.48 and 92.15 mAOD.

The river stage maintains a fairly constant shallow gradient of 0.0013 over the studied section. Ordnance Datum (mAOD) was obtained from known several known borehole levels along Tameside drive. Three known boreholes were surveyed into one another to ensure the levels were accurate. Borehole 23 provided an elevation of 93.86 mAOD, from this a temporary benchmark was positioned at the south bank bridge abutment beneath Gavin Way bridge, which provided a convenient reference point for all levelling during the field work period. The South bank bridge abutment at Gavin Way was assigned a value of 92.257 mAOD. (See Figure 62 later in Chapter 7)

Initial characterization of the river was conducted by taking 12 No. cross-sections between the north of the site and Gavin Way Bridge. These profiles were used in conjunction with profiles taken by Severn Trent during the Perry Barr to Gravelly Hill sewer site investigation (S.I.) to provide bed elevation, stage, and river width data for the groundwater model. In total 22 No. cross sections were used in the groundwater model. All cross sections used are referenced in Appendix 1.

Riverbed samples were taken across profiles XSL 1 to 12 to provide a better understanding of the spatial variability of the river bed sediments and to provide river bed conductivity estimates. A complete discussion about the River Tame characterisation will be presented in Chapter 4.

The river has been extensively modified from its original form as a meandering braided river system on a broad flood plain. The study reach has undergone engineering works for flood defence purposes, including strengthening bank sides and straightening some sections of the river. The channel bottom remains natural and unlined over most of its length.

Bed materials range from sub angular cobbles through gravel, sand and silt and include many artefacts.

Weed growth within the channel is limited during the winter months but is abundant during the summer and is thought to have an effect on the hydrological regime by reducing flow velocities (Clay, 1999).

4. Chapter 4 Characterisation of River Tame and Monitoring methods

4.1. Introduction

Data collection and the methodologies used are the facts upon which later conclusions can be drawn. It is important therefore to have sound methodologies and a systematic approach to data collection otherwise subsequent analysis of hydrological and hydrogeological data and the use of such data in hydrogeological modelling may be unreliable. This chapter looks at the methods used during data collection. Piezometers and piezometer nests have been used in several studies to classify gaining and loosing sections of river.

The data for this study was collected between 01 June 2006 and the 31 August 2006.

A river-bed piezometer network was specifically installed for the study and used in conjunction with two previously installed shallow groundwater monitoring network in the vicinity of the River Tame.

4.2. Archive data

Several sources of information were available for consultation following work conducted by Ellis (2003). Site investigations conducted by Severn Trent Water Company and the Environment Agency in 1993 and 2000 respectfully; provided detailed information about the geology, groundwater monitoring and licensed abstraction data along the study reach.

- Environment Agency
    - Flood defence survey data of 10 river cross-sections were relevant to this particular site. (Appendix 1).

- Licensed discharge and actual abstraction information from the Environment Agency for the Birmingham Aquifer (Appendix 2).

- Geological logs, hydraulic conductivity estimates and sieve analyses from 5 No. shallow piezometers drilled on the banks of the Tame for flood defence investigations (Appendix 3).

4.3. Severn Trent Water Company

- Geological logs from 14 boreholes drilled within a 600m radius of the study site, as part of the Black Country Trunk Sewer Extension (BCTSE) site investigation along the Tame valley (Appendix 4).
- Dipping records (1993 –2000) from piezometers installed for the BCTSE
- 9 No. relevant riverbed cross-sections along the 1km study reach (Appendix 1).

4.4. Location of data collected

An initial reconnaissance allowed the site to be divided into 50m sections using a 30m tape measure and marking each 50m with a wooden stake. Measurements were taken from the south embankment, starting at the down-stream edge of a prominent discharge outlet constructed from red brick; the location of the Discharge outlet is clearly marked as Outlet "A" in Figure 18. Note Outlet "A" is approximately 600m upstream of Gavin-way Bridge. Twelve river cross-sections labelled XSL 1 to XSL 12 were surveyed, to provide information of the Tame Hydrology and structure for use in later modelling. These profiles were used in conjunction with profiles taken by Severn Trent during the BCTSE to provide bed elevation, stage, and river width data for the groundwater model. In total 22 No. cross sections were used in the groundwater model. All levels presented for the riverbed cross-sections are given in meters above ordnance datum.

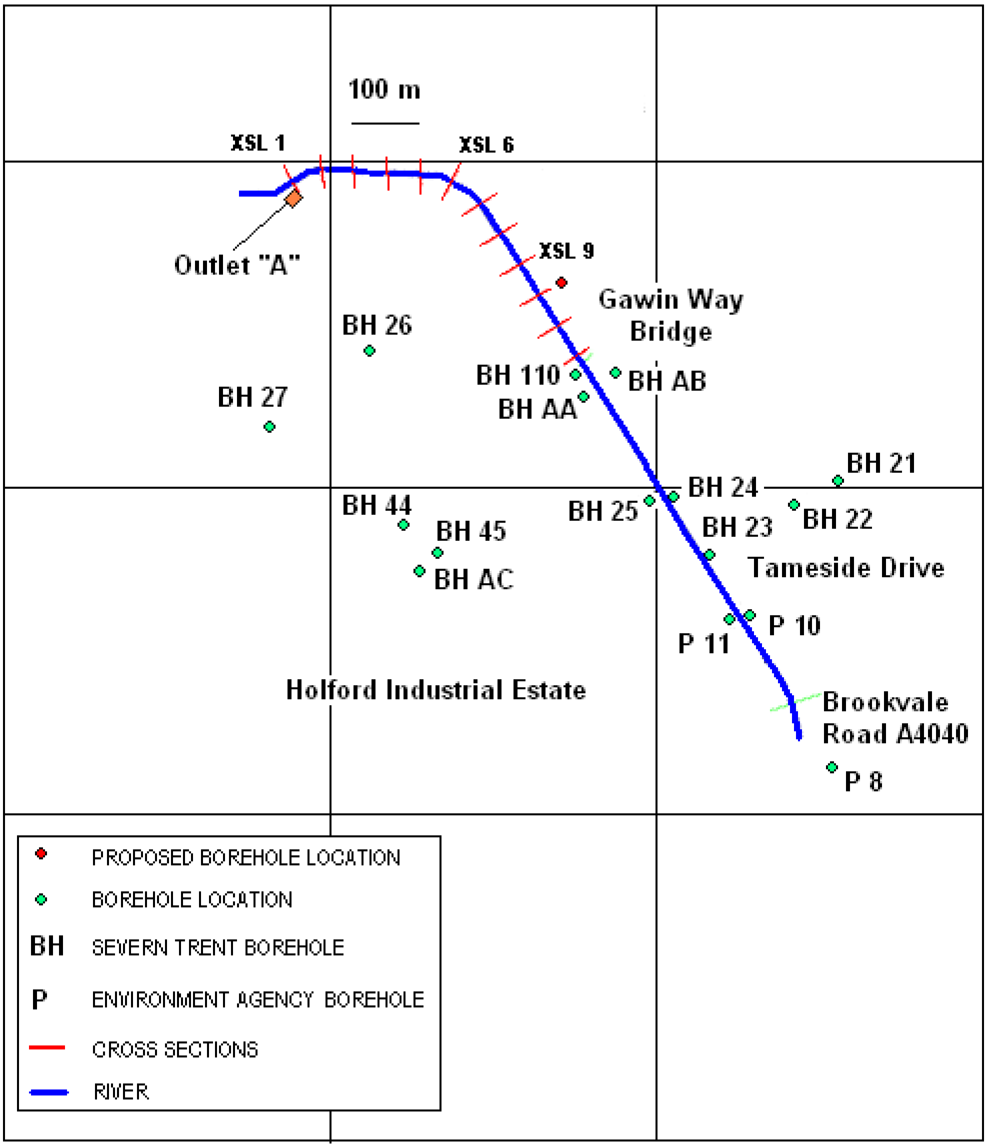

Figure 18: Location of cross-sections surveyed during study with reference to all available boreholes in the locality.

4.5. Mini Drive Point Piezometers (MDPs)

The Piezometer design implemented during this project was adopted from Ellis (2003). Mini Drive point piezometers were used to conduct in-situ falling head tests and measure shallow head in the aquifer directly underlying the river bed.

These measurements were used to calculate estimates of river bed flux and for use in the local ground water model. Moreover head differences between groundwater and stage allowed individual sections of river to be classified into influent, effluent or cross-flow portions of river.

4.6. Construction of Riverbed Mini Drive Point Piezometers (MDPs)

The MDPs were constructed of flexible 13mm OD (10mm ID) high density Polyethylene (HDPE) tubing with 3 No. 30mm (10mm ID) penny washers held in place using a 35 mm long stainless steel M10 bolt screwed into the base of the open tube(Figure 20). The M10 bolt provided a drive-point with ample endurance during the installation process. Directly behind the bolt was a 100mm open section of drilled holes screened with nylon mesh (100 microns) secured by stainless steel wire.

The MDPs varied in length but typically ranged between 1.7 and 1.90m.

Figure 19: Typical MDP installed in the riverbed

Shallower piezometer nests were later constructed. The nests consisted of two MDPs of average length 1.7m. The piezometers were placed in series, with the shallow piezometer upstream, and the deep piezometer downstream. The shallow piezometers were installed at 0.30 m. This allowed estimates of conductivity for the shallower bed sediments to be determined. The deep nested MDPs were designed to give conductivity estimates at an intermediate depth with respect to the first deep MDPs and the shallow MDPs at 0.30 m. The deep nested MDPs were therefore installed at exactly 0.50 m below the riverbed surface.

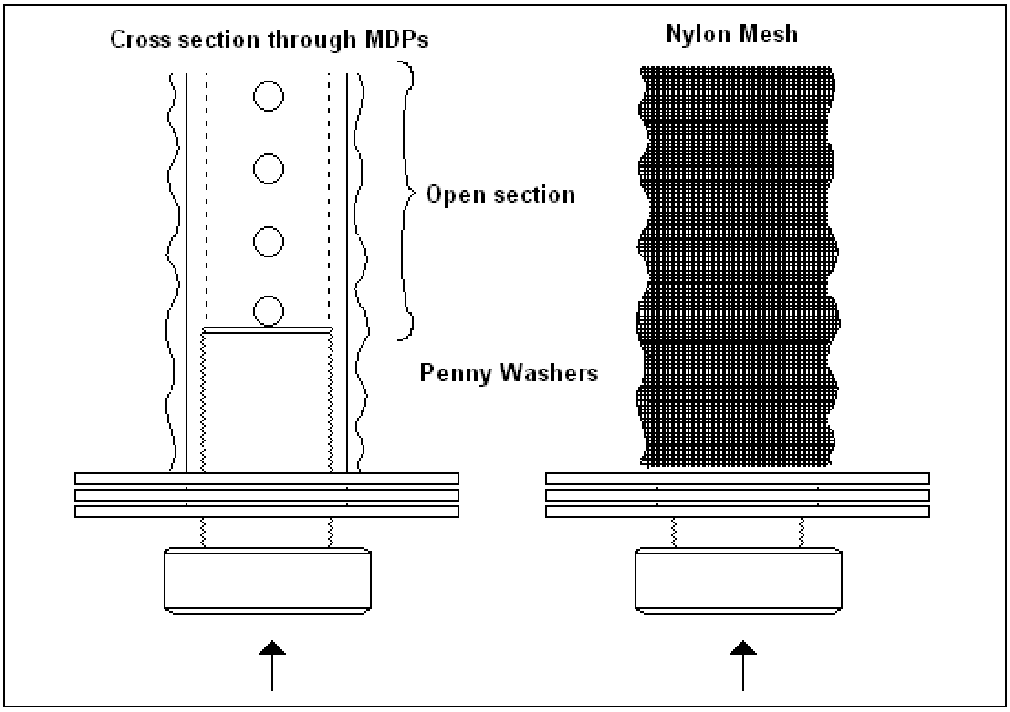

Figure 20: Schematic of MDP with penny washers, M10 bolt and open section location complete with nylon

The following include the construction materials used, and are presented below in Figure 21 and 22 below.

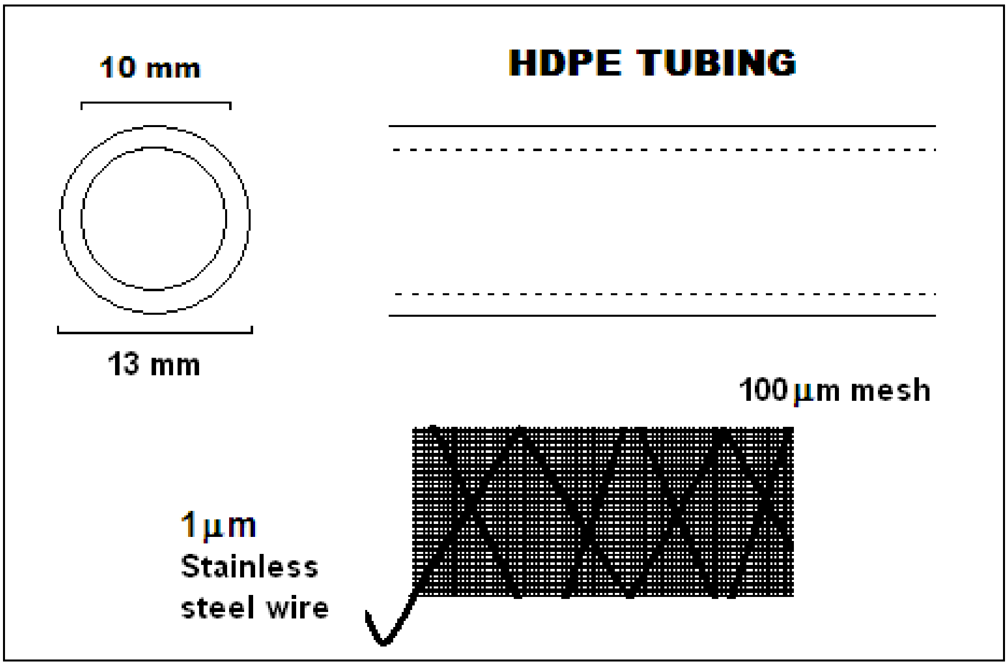

Figure 21: Construction materials for MDPs & dimensions of HDPE tubing

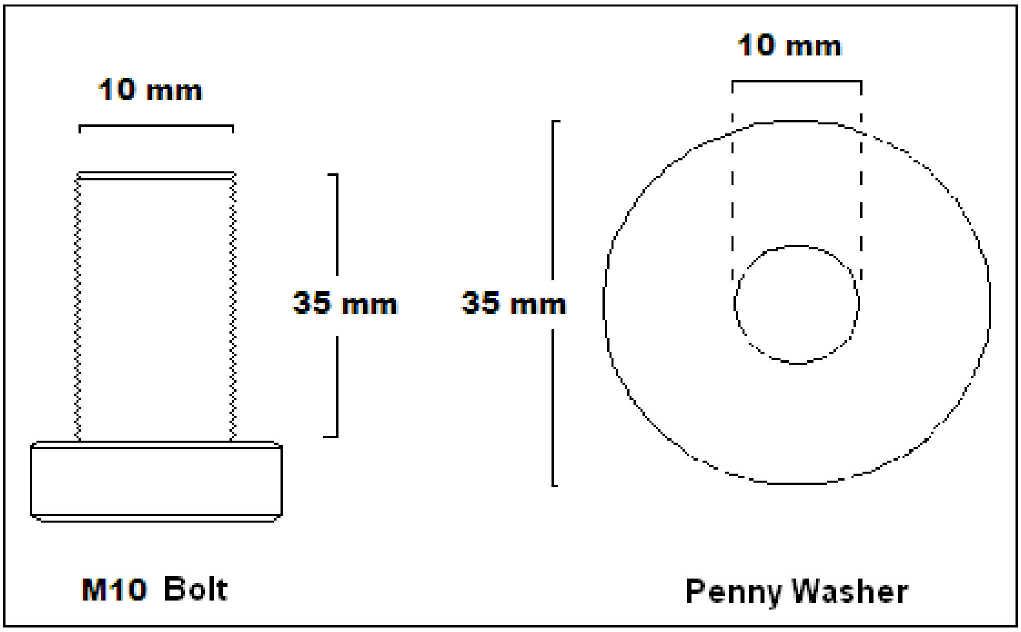

Fig Figure 22: Construction materials for MDPs

4.7. Installation of MDPs

Piezometers were inserted inside a 2 m length (20mm ID) steel tube and driven by hand into the riverbed using a 15 kg fence post driver to depths varying from 0.30 to 0.932 m (Figure 23). The driving tube was removed leaving the MDP securely in place. The MDP was sealed with a rubber stopper to prevent the growth of algae within the tube and to prevent artesian flow which would disturb the natural system. The MDPs offer flexibility during varying river flow and allowed detached vegetation to slide over them during the summer period, when an increase in vegetation growth was observed. Sand and gravel bed sediments of the river Tame provided varying resistance to the installation of the Mini Drive-point piezometers.

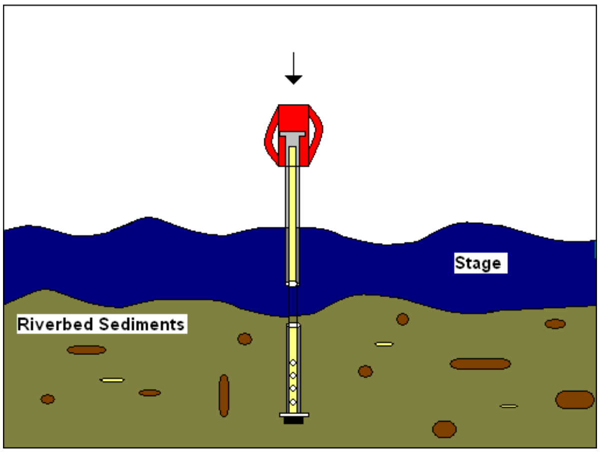

Figure 23: Installation of MDPs using drive post

4.8. Shallow monitoring wells

Groundwater head measurements were obtained from two sets of shallow piezometers (<13m) within the sandstone aquifer and overlying drift deposits located adjacent to the River Tame, for later use during the model construction.

The 'Black Country Trunk Sewer Extension Project' was conducted during the second half of 1993. Around 70 No. holes were drilled by rotary and percussion techniques with an average borehole diameter of 121 mm to depths of between 10 to 35 metres. The holes were then back filled with arisings to the bottom of the response zone within the sandstone. The response zone comprised a one metre section with a bentonite seal at top and bottom, filled with a sand filter and containing a PVC piezometer (ID20 mm) with a 30 cm screen installed 30 cm from the base of the zone (Gabriel, 1993).

The hole was covered with a standard 0.13 x 0.13 m stop-cock cover. Attempts at locating 14 No boreholes within a 600 m radius of the study site, proved unproductive as many of the holes are no longer accessible. A total of 6 out of the 14 holes were found intact and in good working condition, during the course of the project.

Details of the Boreholes located and their current operational condition are presented in appendix 5.

The second set of piezometers was drilled during summer 2000 as part of the Environment Agency River Tame Asset Survey. The holes (ID 150 mm) were drilled to depths of up to 10.5m by light percussion rigs within 15 metres of the riverbank. The holes were back filled with arisings to the base of the response zone.

The response zones ranged from 1 to 3 metres, within the sandstone, with a bentonite seal at top and bottom, filled with a sand filter and containing a PVC piezometer (ID19 mm) (Caudell, 2000).

4.9. Deep abstraction wells

It was necessary to look all local abstractions within a 1 km radius to determine their likely impact on groundwater head distribution in the area, as any significant drawdown would impact model calibration at a later stage.

A comprehensive review of all deep abstraction sites was undertaken during the project. Many of the sites containing boreholes have undergone redevelopment over the last number of years resulting in a halt to some groundwater abstractions.

Historical Borehole logs from wells bored prior to 1942 were available for consultation. 11 No boreholes were found to be located within a 1 km radius of the study site. The logs provided some insight into drawdown and well yield data for 5 of the boreholes. However much of the yield data is for the Kidderrminster Formation and the proposed site is situated on Wildmoor Formation. An excel file was constructed to analyse the data (See Appendix 5).

4.10. Borehole Locations

All 19 Site investigation boreholes used and referenced during the modelling portion of this project can be found in (Figure 24).

The details of the Severn Trent piezometers included collar locations and elevations according to ordnance datum. These boreholes were used as fixed reference points from which the flood defence boreholes, riverbed piezometers and river stage were all surveyed, using a prismatic level with stadia. Maps at 1:1250 scale were available from Severn Trent showing borehole locations. Environment Agency maps were available at a scale of 1:1000 showing the River Tame and surrounding area and marking the location of surveyed river sections and flood defence boreholes. Copies of the maps are available for reference in Appendix 3 and 4.

Temporary bench marks were setup along the riverbank for convenience.

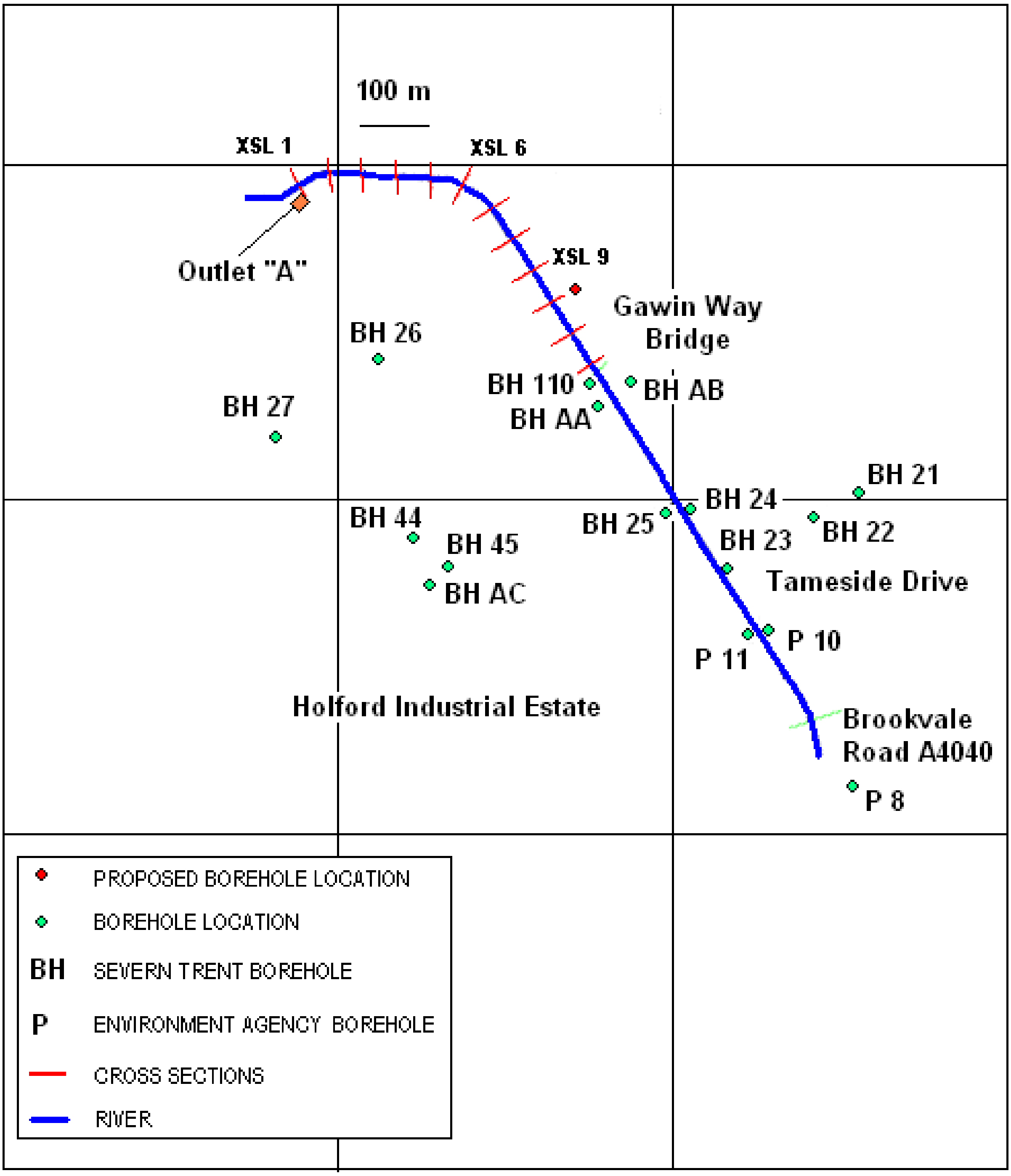

Figure 24: Combined Borehole Location Plan (Environment Agency boreholes and Severn Trent Water) with reference to the Tame study reach

4.11. Groundwater surface water head measurements

Groundwater head measurements were obtained from operational boreholes across the area, using a standard 30 m Solinst water level dip meter.

Depth to water measurements were taken, and then subtracted from the collar elevation to get absolute mAOD.

A similar method was employed for stage measurements and head measurements in riverbed piezometers. In all cases the piezometer tips were levelled, to avoid any errors caused by fluctuations in the riverbed thickness (as a result of fluvial transport mechanisms).

4.12. Characterisation of the Riverbed Sediments

A combination of in-situ falling head tests conducted in riverbed piezometers and sieve analyses of riverbed material were carried out to establish estimates of hydraulic conductivity for the riverbed sediments. Estimates of porosity were also obtainable from the samples collected. The methodologies adopted in the analysis are presented herein.

4.13. Sediment size Analysis

Hydraulic conductivity can be estimated by particle size analysis of sediment.

Using empirical equations, the hydraulic conductivity of unconsolidated sediment relates to the packing of the particles and the void spaces between them.

By analysing the constituent sizes of the grains in the river bed samples, a grain-size distribution curve was constructed. The Tame river-bed sediments displayed a layered stratigraphy of unconsolidated sands and gravels. (See Figure 25)

Figure 25: Layered bed-sediments -medium-coarse sand and gravel (yellow) on top of fine to medium red sand

31 No. samples from eleven sampling points at cross sections XSL2 to XSL12 were sieved. 3 No. samples were taken from each cross-section of the river Tame, with the exception of XSL 2 where an elevated river stage only permitted one sample to be taken. Sampling depths ranged between 0 – 0.50 m below the river bed. In each case samples were taken 2, 4 and 6 meters out from the south bank channel edge. Any deviations from the sampling distances are clearly marked on the cross-sections in Appendix 6.

4.14. Conducting the sieve analysis

The samples were individually dried using a (GALLENKAMP) hot box industrial oven with fan, at 110 degrees centigrade for 24 hours, following which they were weighed using a (Spartorius 4000g) balance which was sensitive to one gram.

The samples were then passed through a set of 8-in (203-mm) brass sieves which were shaken by hand until no fine material passed the constituent sieve mesh.

Sieve openings ranged from 13 mm to 0.053 mm in diameter.

The full set of sieve-sizes used along with photos of the equipment used can be obtained from Appendix 7.

The sieves were shaken in a circular motion with some up and down movement, accompanied by jarring to keep the material moving on each sieve and prevent clogging. (Driscoll 1986).

During the sieving process, a percentage of the total sample weight is retained on each sieve, with the finest material collecting in the bottom tin. A semi-log plot of sieve-size versus cumulative percent retained was plotted in each sample case.

A cumulative weight was calculated to verify that it matched the total sample weight which was taken for a second time at the end of the analysis to reduce error through loss of sample. The cumulative weight in all cases was within two grams of the total weight.

The cumulative percentage retained was calculated in Microsoft Excel by dividing the cumulative weight by the total weight.

Following calculation of the cumulative percentage retained, a grain-size distribution curve was constructed by plotting the cumulative percentage retained against the sieve openings in millimetres, on semi-log paper. In each case the size of the sieve openings were taken to represent the diameter of the smallest particle retain on each sieve.

A smooth curve was fitted to the points to permit estimation of the percentage of each soil constituent. The samples taken were logged in the field and latter corrected using the results of the sieve analysis. Sample descriptions are based on British standards (BS5930: 1991) and presented along with the frequency distribution curves in appendix 7.

4.15. River Discharge calculations

Values of flow velocity were collected from 1.0 meter intervals across each river section, during a period of dry weather flow.

These measurements were required to support flux measurements calculated using simple Darcian flow equations, to classify loosing and gaining portions of the river reach. All measurements were taken within a single afternoon to minimise errors caused due to fluctuations in baseflow.

Figure 26: Velocity measurements conducted at one meter intervals across the River Tame

A flow propeller was used in collecting the velocity measurements.

Areas of minimal flow disturbance were chosen to reduce inaccuracies caused by vegetation. However some loose vegetation occasional got caught on the propeller skewing results. In this instance the propeller was cleaned and the measurement repeated. The river stage was measured at each interval, so that the effective placement depth for the flow meter could be calculated. The velocity at 0.6 m of the total depth is considered representative of the average velocity occurring over the total depth interval.

The total discharge for that location can then be calculated using the following formula:

$$Q = \sum_{i=1}^{n} (X_{i+1} - X_i)(U_i Y_i + U_{i+1} Y_{i+1}) / 2$$

Where:

Q= total discharge

X_I = distances to successive velocity measurements from the river bank

U_i= velocity measurement at each interval

Y_i= depths at each interval

4.16. Estimation of bed permeability from sieve analysis

The curves were used to determine effective or average grain-sizes, often represented in soils by the d_{10} (Hazen's effective grain-size). The d_{10} symbolizes a grain diameter greater than that of 10% of the particles by weight.

The Uniformity coefficient (C_u) is the ratio of the grain size that is 60% finer by weight to the grain size that is 10 % finer by weight (Hazen's effective grain size).

$$C_u = \frac{d_{60}}{d_{10}}$$

A well graded soil has a lower C_u. A soil with a uniformity coefficient smaller than 2 is well sorted, see Table 5.

Cu	**Sorting**
< 2	Well sorted
2 -- 4	Moderately Well sorted
4 -- 6	Poorly sorted
> 8	Very Poorly sorted

Table 5: Uniformity Coefficient values relating to the degree of sorting

4.17. Methods Used in calculation of hydraulic conductivity

Using empirical equations by Hazen, (1911) and Shepherd, (1989) estimates of hydraulic conductivity for the bed sediments were determined.

4.18. Hazen (1893)

Hazen's method is appropriate for sandy sediments where the d_{10} is between 0.1 and 3.0 millimetres.

$$K = C\ (d_{10}^2)$$

Where:

K = Hydraulic conductivity (mms^{-1})

C is an empirically-determined coefficient based on sorting see Table 6. (mms^{-1})

d_{10} = the effective grain-size, (10 % finer by weight) (mm)

Sediment Character	Range of C coefficient (cms^{-1})
Very fine sand, poorly sorted	40 – 80
Fine sand with appreciable fines	40 – 80
Medium sand, well sorted	80 – 120
Course sand, poorly sorted	80 – 120
Course sand, well sorted, clean	120 -150

Table 6: Representative Values of the Hazen coefficient for different grain sizes and degrees of sorting (Fetter, 1994).

4.19. Shepherd (1989)

Shepherd developed an empirical formula for approximating hydraulic conductivity K (feet day^{-1}) from grain size analyses. He developed a relationship between grain size and conductivity based on field and laboratory data from 18 published studies. The general formula for the relationship is as follows:

$$K = C\,(d_{50})^{j}$$

C = a shape factor

d_{50} = the mean grain diameter (mm)

j = an exponent

Representative values of the Shepherd shape factors and exponents for different levels of sediment maturity are shown in Table 7. The shape factor and exponent are greatest for texturally mature sediments which are characteristically well sorted with uniformly sized particles with a high degree of roundness and sphericity.

Note: hydraulic conductivity values derived will be in feet day^{-1}but are converted afterwards to md^{-1} for the results presented in Chapter 5.

Sediment Character	**Shape factor 'C'**	**Exponent 'j'**
Texturally mature		
Glass spheres	40000	2
Dune deposits	5000	1.85
Beach deposits	1600	1.75
Channel deposits	450	1.65
Consolidated sediments	100	1.5
Texturally immature		

2. Table 7: Shepherd shape factors and exponents for different levels of sediment maturity

4.20. Riverbed Coring

Riverbed cores were conducted during the course of the field work.

The cores were designed to improve on understanding the heterogeneities observed in bed sediments, paying particular attention to identifying a distinct difference in bed sediment layers observed during the sample collection reconnaissance work.

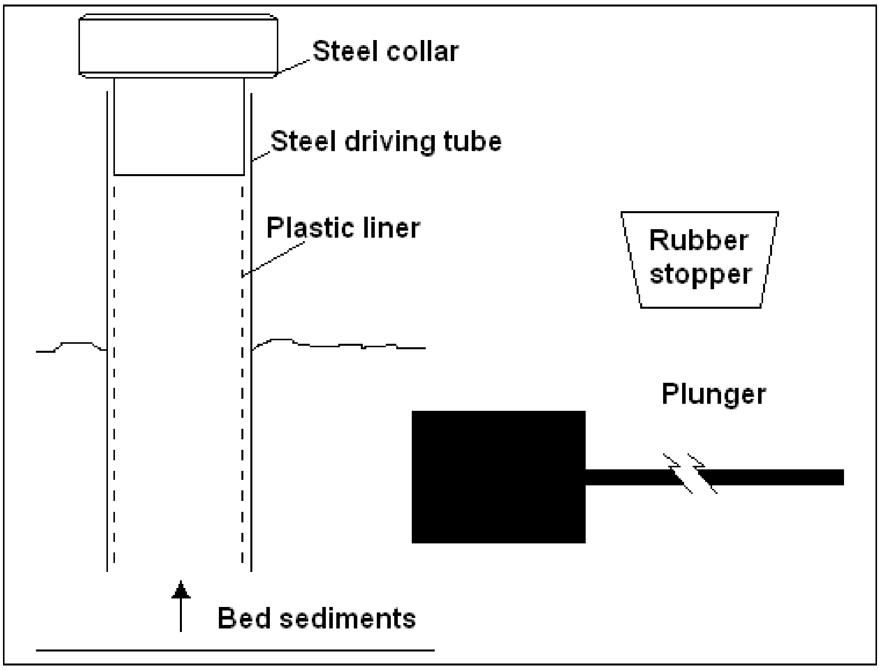

Figure 27: Schematic of the riverbed coring device used during the field investigation

A steel pipe (ID 50 mm) 0.70 m in length was lined with 48 mm plastic core liners.

The steel pipe contained flat collar which attached to the top end of the pipe to minimise damage caused to the driving pipe. (Figure 27)

The mode of operation was to drive the steel tool, using a 14 pound sledge hammer, into the bed sediments and then extract the sediment core from inside the steel driving pipe using a rubber stopper in the top end of the pipe where the collar sits. The rubber stopper was used to create enough suction during the withdrawal process, to prevent the sediment from falling out the base of the tool. A plunger consisting of a long steel rod with a cylindrical block (48 mm

in diameter) was used to push the inner liner containing the sediment core sample out of the driving tube.

4.21. Estimates of riverbed conductance

Permeability (or hydraulic conductivity) of the bed sediments represents the ease with which water can flow through the river bed. The river stage and piezometer head differences were used in combination to derive a reasonable estimate of flux to the river using the Darcy equation. The following approach to estimating the riverbed flux was adopted.

4.22. Hydraulic gradient

Groundwater head measurements were made by finding the depth to water within each piezometer from the respective piezometer tip.

The piezometers are constructed of clear HDPE which permits head measurements to be made directly. The piezometers were held upright, without the rubber stopper plug, to allow time for the head to equilibrate with atmospheric pressure.

All measurements were taken with respect to the piezometer tip, subsequent river stage measurements were taken at each piezometer location.

The head difference between the stage and groundwater measurement permitted the calculation of the hydraulic gradient (i) as follows.

$$i = \frac{h_2 - h_1}{L}$$

Where:

h_2 = water level in piezometer

h_1 = river stage

L = installation depth of piezometer in riverbed

4.23. Groundwater flux

A one dimensional analytical solution for Darcy flow was used to gain an estimate of the flux of the groundwater to the River Tame.

$$q = K\,(i)$$

Where:

q = the specific discharge (m/d)

K = hydraulic conductivity (m/d)

i = hydraulic gradient (-)

To obtain an estimate flux over the whole river reach the specific discharge for a piezometer can be multiplied by the river width in order to get a flux per unit width. This flux is then applied upstream and downstream to the midpoint between the nearest piezometers. When this is done for each piezometer a flux estimate for the whole reach can be calculated. However it must be noted that there is considerable ambiguity when using the above method for estimating fluxes as the piezometers only provide single point estimates as a very small volume of the sediment is tested. The sediments are highly heterogeneous and extrapolating values obtained to large areas of river will result in inaccurate results.

4.24. Porosity

Sand samples taken at discrete depths within the river bed were used to gain an estimate of the river-bed porosity. For each sample, the porosity was calculated by dividing the volume of water added it by the total volume of the material.

$$n = \frac{V_v}{V} x100\%$$

Where:

V_v corresponds to the volume of void space occupied by water

V corresponds to the total volume of sample

The results were expressed as a percentage. In all cases a sample of weight greater than 1.5 Kg was used, the large samples were considered large enough to yield representative results.

4.25. Falling Head Tests

The determination of in-situ permeability from the mini drive point piezometers involved the application of a hydraulic pressure, in the piezometer different to that in the ground, and the subsequent measurement of the rate of flow due to this difference.

In all cases the pressure in the piezometer was increased by introducing water into it.

Fines that accumulated within the piezometer were abstracted using a small peristaltic pump (Masterflex pump Model 70 15 – 20), twenty four hours prior to testing to reduce possible resistive effects caused by a filter cake forming.

Even after this action, it is hard to be certain of clean water in the piezometer, since some suspended material always remains and settles into sediment under stagnant conditions. (Cey et al., 1998) have recognised that clogging of the riverbed piezometer screen may reduce the estimated conductivity.

Filter cakes can form of only a few grain thicknesses, resulting in errors of an order or more in magnitude in the assessment of permeability. (BS5930: 1999)

A 5 mm diameter tube, connected to a pressure transducer, was inserted to the piezometer base below the static water level, prior to the start of the test. The water level in the piezometer was then allowed to equilibrate so that a static water level could be recorded by the transducer.

De-aired water was fed directly into the top of the piezometer using a funnel that attached directly to the outer lining of the piezometer tip at the start of the testing procedure.

The pressure transducer was attached to a Hobo (4 channel) data logger which, allowed accurate measurement of the pressure and hence head difference in the piezometer over slow and most importantly rapidly falling head tests.

4.26. Analysis of the Falling Head Test data

The falling head tests conducted in the riverbed were analysed using the following modification of the Hvorslev equation following Hvorslev (1951), to obtain estimates of hydraulic conductivity.

$$K = \frac{r^2 Ln(Le / R)}{2 LeT_0}$$

K = hydraulic conductivity cm s^{-1}

r = radius of well casing (0.5 cm)

R = radius of well screen (0.65 cm)

Le = length of well screen (10 cm)

T_0 = Time for water level to fall to 37% of the initial level

The pressure transducer gave out puts in volts (V), which could be converted to a corresponding head of water, following calibration of the equipment.

The pressure transducer was calibrated by inserting the open-end tube into a tall column of water. The open-end tube was successively inserted 0.05 m every 30 seconds, to a total depth of 1.55 m. The voltage readings for each insertion step were averaged for the 30 seconds and plotted against depth intervals of 0.05 m.

A trend-line through the results produced an equation of a line as follows:

$$Y = 0.154X + 0.4418$$

Where:

X is the depth in meters

Y is the corresponding voltage reading

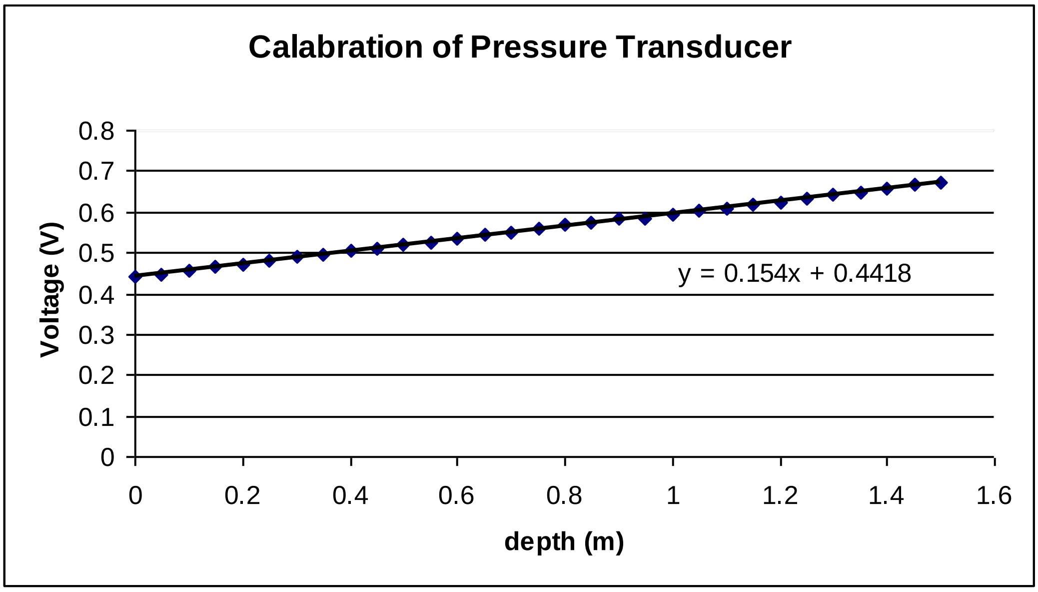

Figure 28: Calibration of Pressure transducer for Falling head tests

An Excel spreadsheet was set up to convert all voltage readings to depth of water in meters. The results of each falling head test was plotted as **(h – wl) / (h_0 – wl)** on the y-axis (log scale) against time in seconds on the x-axis.

Where:

h = water level at time t,

h_0 = water level at the start of the test,

wl = the original head of water before the test began.

A straight line was fitted to the points using the trend line function in Excel to derive two constants describing the line in the form:

$$y = ae^{-bx}$$

T_0 was then derived using the equation:

$$T_0 = \frac{(Log(0.37/a))}{-b}$$

A spread sheet was used to calculate the value of K for the bed sediments which would contain both vertical and horizontal components.

4.27. Conclusion

Most of the methods presented in this chapter have been tried and tested in recent years during MSc and PHD projects. Many of the authors document success using these methods. None of the methodologies applied here are stand alone methods and where possible were supported by more than one method.

The results of the data collected are presented in the subsequent chapter.

5. Chapter 5 Fieldwork Results

5.1. Deep Abstraction Boreholes

The Environment Agency abstraction database provided an insight into the likely abstractions surrounding the study site. These discharges are important to consider as any significant drawdown could potentially interfere with the proposed experiment.

3 No. sites were visited following a detailed inspection of current licensed abstraction data provided by the Environment Agency (End of 2005).

One site was found to have suspended pumping completely (Grid: SP069915).

Two sites within a 500 m radius of the River Tame were found to be operational and licensed to abstract up to 2.27 ML/d in total, from boreholes up to 100 m in depth.

The area adjacent to the study reach (Holford Industrial estate) and surrounding district has had a long industrial past, which exploited groundwater from the Birmingham Aquifer from between the 1850's until the 1960's. A reduction in abstraction rates was observed in line with the post war decline in manufacturing industries in the Birmingham area.

The maximum reduction in abstraction rates was achieved during the 1960's which resulting in rising water table and increased base-flows to the Tame. (Powell et al., 2000), (Greswell et al., 1994), (Knipe et al., 1993)

For a number of years throughout this extended period of abstraction, increased drawdown resulted in various parts of the River Tame to become effluent to the underlying Birmingham Aquifer (Land, 1966, Jackson et al., 1983).

Due to the reduction in Abstraction much of the River Tame is gaining.

Knipe et al. (1993) states that under natural conditions groundwater flow is towards the River Tame.

A study by Thomas (2001) found that abstraction rates for the unconfined portion of the aquifer saw a reduction from 45 ML/d in 1993 to just 11 ML/d in 2001.

Current revision of abstractions for the unconfined portion of the aquifer found that pumping is now at 1.8 ML/d for 2006.

This calculation is based on actual abstractions returned and includes abstraction rates not returned to the Environment agency for the 2006 period. 13 No. companies containing wells situated on the unconfined aquifer were reviewed. 6 No. companies were found not to be abstracting in 2006. The companies that are abstracting are shown in Appendix 2 along with their current abstraction rates.

An old works at grid: SP076910 is currently under redevelopment and abstraction rates for 2005 were not returned to the EA; however a meeting with the site manager confirmed that only two boreholes (BH “5” and “6” are still abstracting groundwater) The remaining boreholes on the work site have since been decommissioned with the exception of two boreholes that remain as monitoring boreholes as requested by the Environment Agency. This site is of particular interest due to its close proximity to the proposed drilling site. Unfortunately, current abstraction rates for the two operational boreholes were not available, so a best estimate was calculated.

The boreholes are being used to supply storage tanks for onsite industrial use; they are estimated to pump for 10 h/d, and for six days a week. This constitutes around 35.5 percent of its allocated allowance, giving the works an estimated abstraction of 0.48 ML/d. This brings the total estimated abstraction rate for the surrounding area (500m radius) to 0.806 ML/d. The two licensed boreholes are located to the south and south west, and are on the opposite sides of the river to the proposed Dipole setup.

The abstractions are not thought to significantly impact the future site.

5.2. Stratigraphic correlation of Borehole logs.

9 No Severn Trent Water boreholes records were used in conjunction with 5 No. Environment Agency boreholes to establish

- Likely thicknesses of the expected strata
- Depth to principal aquifer
- Nature of the overlying sediments
- Any problems encountered during drilling
- Drilling methods used

The 14 No. engineer's logs were consulted to estimate the principal strata expected along the study reach. Four primary layers were found.

Made ground was found to overlie a mixture of sands and gravels. The sands and gravels sit directly on-top of weathered Triassic Sandstone before reaching weak red brown sandstone.

The local made ground typically consist of dark brown, slightly sandy gravely clay with occasional brick, concrete and slag fragments.

Sands and gravels consist of medium dense dark brown silty gravely Sand. The gravel consisting of sub angular to rounded quartzite, with occasional cobble.

The Triassic weathered sandstone is typically recovered as very dense red brown fine to medium Sand.

The major aquifer unit being the Wildmoor formation consists of weak red brown sandstone.

A stratigraphic correlation between adjacent boreholes allowed a conceptual understanding of the geology and its gradients to be determined.

From these logs an average thickness for the four units was calculated and a good estimate of the expected geology is presented in .

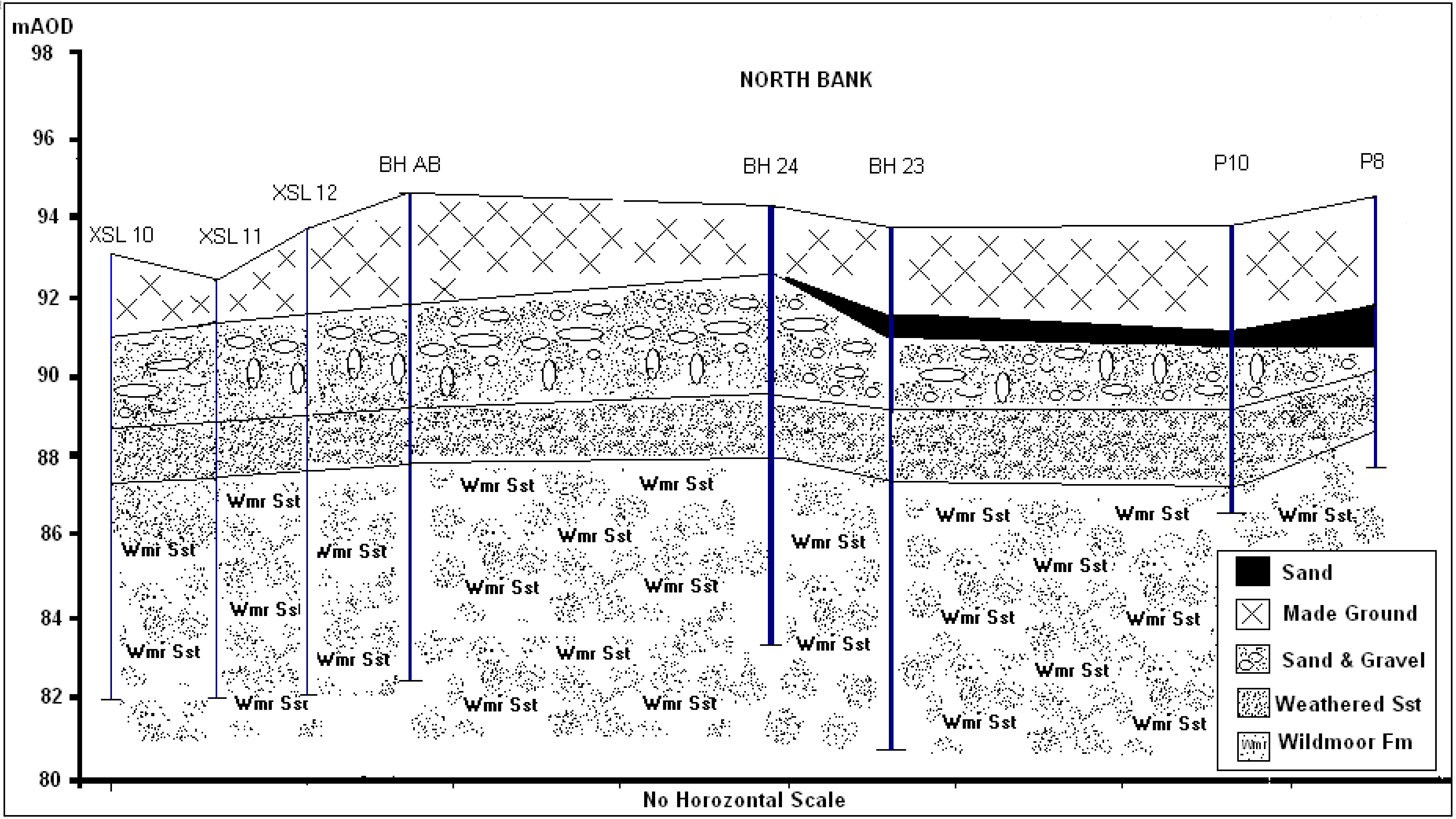

Figure 29 : Expected geology at XSL 11 and XSL 12 from stratigraphic correlation of Severn Trent Water and Environment Agency borehole logs

Dipping records from 1993 until 2000 were available after Ellis (2001) from Severn Trent Water. Average water levels over this monitoring period were used in the model calibration. Although variation up to 0.50 m in groundwater levels had occurred over this period, the average water levels corresponded well with measured groundwater levels during the study period, and the averaged values were used to supplement the incomplete head data in areas >13m from the River Tame.

A complete log of the groundwater level data collected by Severn Trent and levels collected during the study can be found in Appendix 8.

The average ground water values were used to calculate hydraulic gradients between boreholes. Boundary conditions for the modelling portion of the work were assigned based on averaged groundwater levels at exact points and, based on extrapolation of the gradients out to the required bounds.

5.3. Mini Drive-point Piezometer installation (MDPs)

The total number of MDPs installed came to 37. No. The MDPs were installed as lateral profiles at 12 different locations along the study reach. Figure 30 illustrates the location of the piezometer installations across profiles 7 to 16.

Profiles 8, 9 and 11 contain multiple installations, including nested piezometers; these are shown in Figure 31to Figure 34.

A complete set of cross sectional profiles 7 to 12, displaying installation depth of each piezometer is included in Appendix 6.

Installation depths for the MDPs ranged from 0.30 m to 0.932 m.

The MDPs were driven on average to a depth of 0.573 m. Penetration of the bed sediment became increasingly difficult below this depth due to the nature of the strata.

The nested MDPs were installed to depths of 0.3 (shallow) and 0.5 m (deeper) below the riverbed surface. The piezometers are distinguishable in the field as the deeper piezometers were installed upstream 15 cm from the shallow installation, and extruded 0.2 m less than the shallower piezometers.

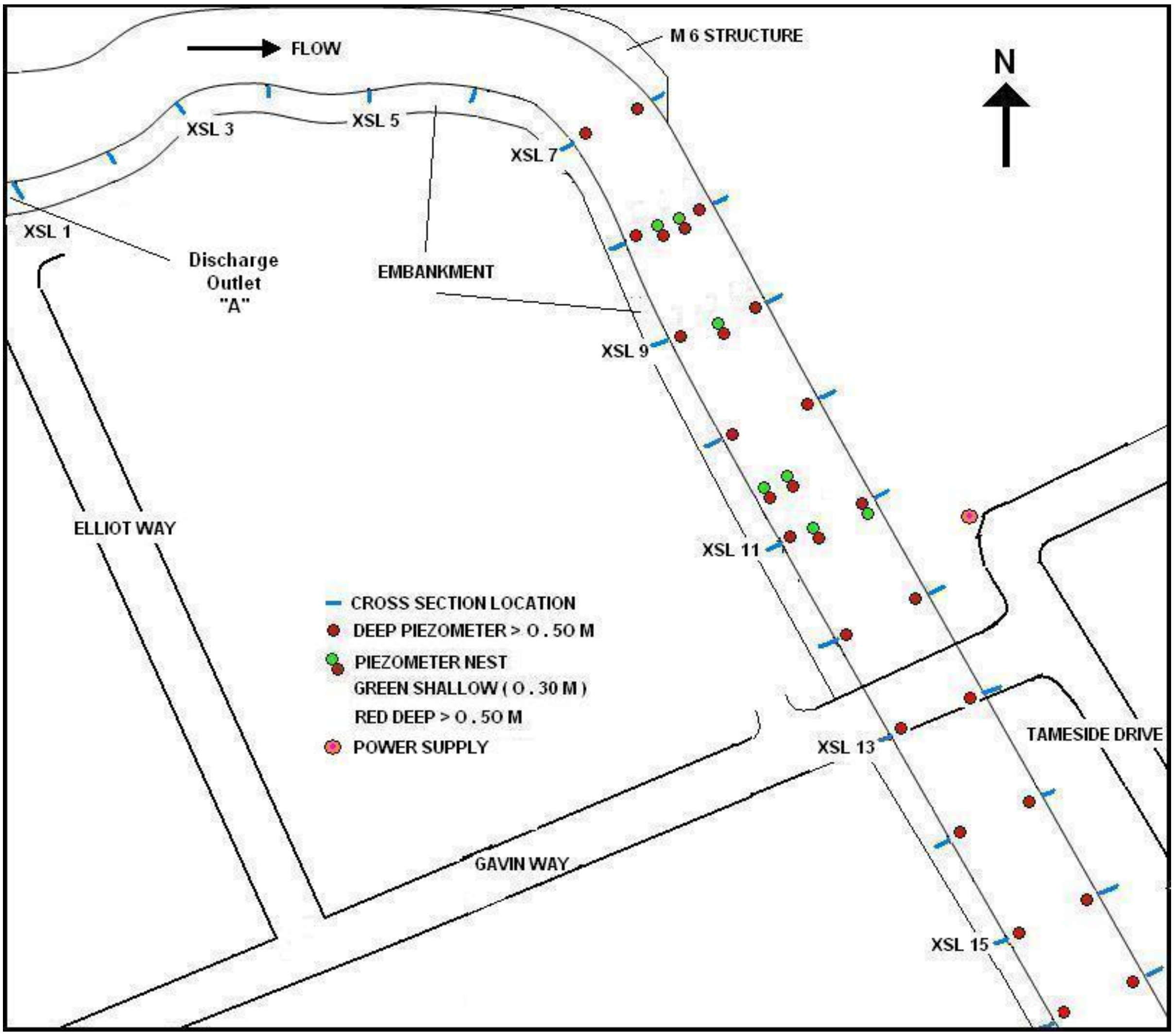

Figure 30: Location of installations along the study reach. (Tame River)

The piezometers were positioned across the channel to assess the heterogeneities within the riverbed. The MDPs were installed along the channel bed approximately 2 meters from the north and south banks. Although modelling by Ellis (2003) highlighted the need for some installations within the banks and seepage faces to pick up likely high flow zones, installations in these areas were not achievable during the course of this project as the embankment material mainly consisted a mixture of made ground and fluvial sediments,

(mostly consolidated sands and clays) emplaced during the Environment agency flood defence site investigation work in 2000. The installations of piezometers in these areas were therefore outside the scope of this project.

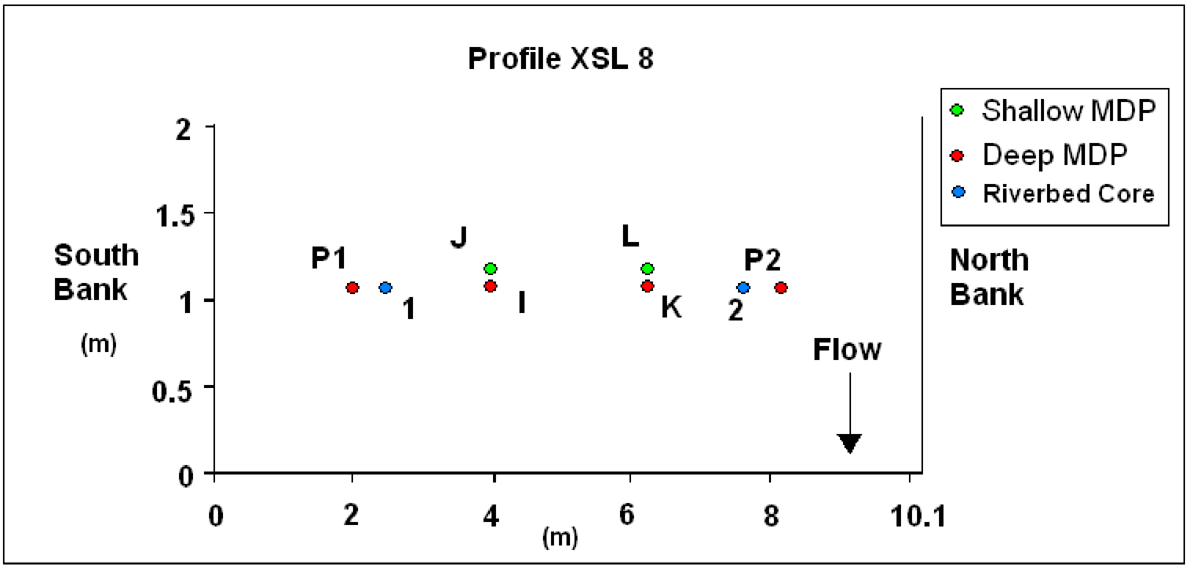

Figure 31: Location of MDP at XSL8 installations and riverbed core samples

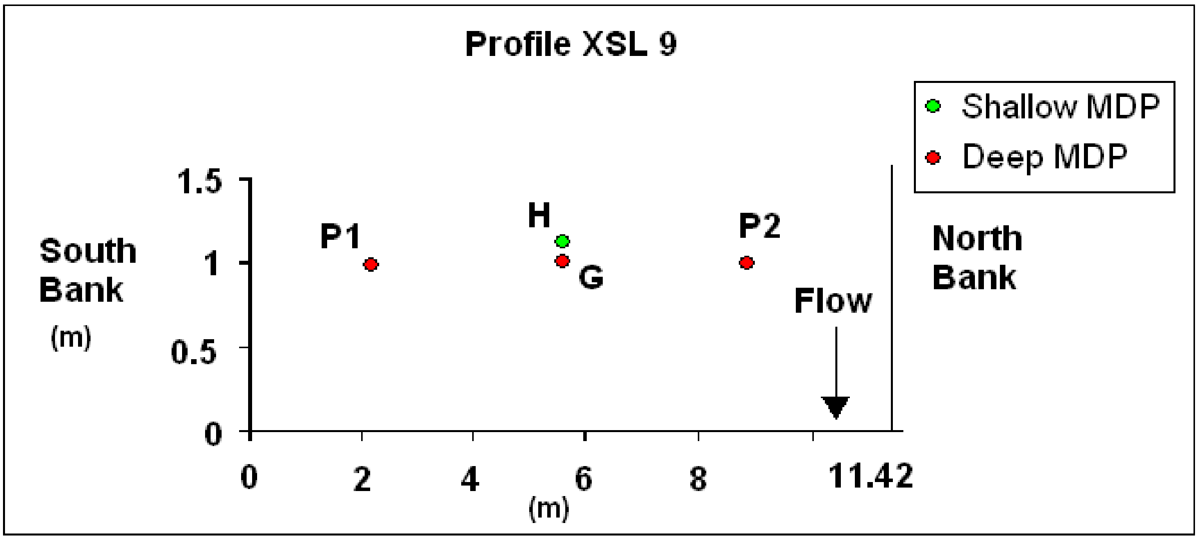

Figure 32: Location of MDP installations at XSL9 and riverbed core samples

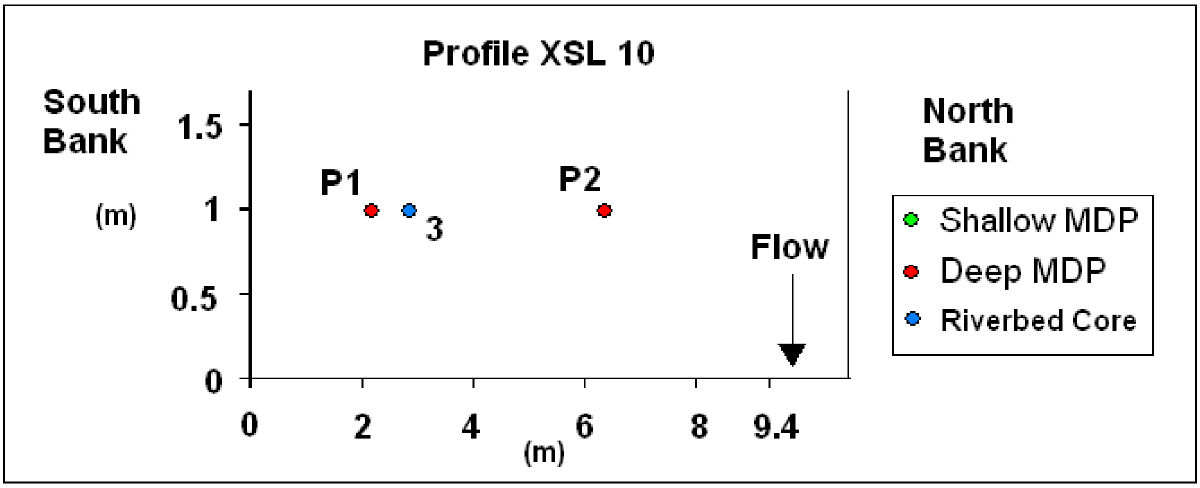

Figure 33: Location of MDP installations at XSL10 and riverbed core samples

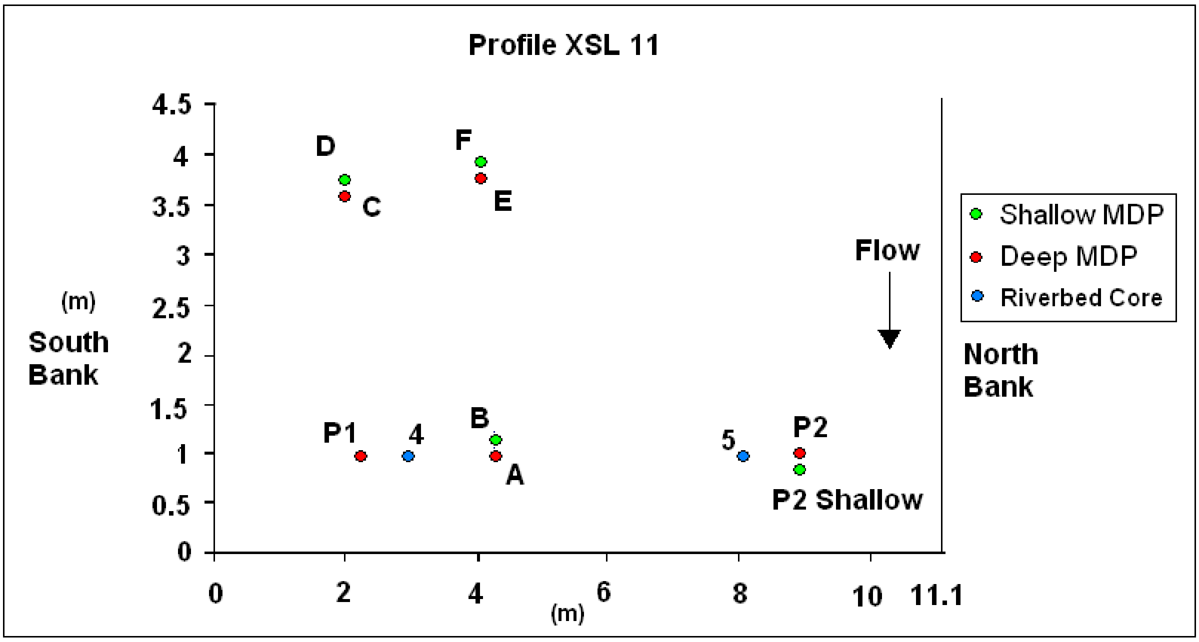

Figure 34: Location of MDP installations at XSL11 and riverbed core samples

5.4. Grain size Analysis Results

One of the most important factors that influence a stream's hydraulic conductivity is the streambed's sediment size distribution. This distribution influences inflow and out flow rates from the underlying aquifer and river channel respectfully. The sediment size distribution also controls the mixing potential of the hyporheic zone.

The investigation work along the stretch of the river Tame identified vertically stratified bed sediments in terms of particle size, in three layers, with each layer having its own distinct grain size distribution. The top two layers of the streambed are represented by alluvial deposits and display a coarsening upward sequence from sand and gravel through to cobbles at the river bed surface. However cobbles are not always present. The uppermost layer of cobbles is only as thick as the largest particle size contained within the deposit. Based on the narrowness of this first cobble layer and poor compaction, this layer is not considered to have noteworthy control on the hydraulic conductivity of the river bed.

The underlying sand and gravels are also of fluvial origin and are distinguishable both in the field by there contrasting colour, and in the lab by their distinctive physical characteristics. The upper sands display a medium to course grain size distinctive from the underlying red sands containing rare weathered sandstone fragments most likely of reworked Triassic origin.

A total of 31 riverbed samples were taken to depths of up to 0.35m from Profiles 1 to 12 across reach. The sediments typically displayed a layered sequence comprising a 5 - 10 cm section of coarse gravel occasionally containing cobbles <30cm in diameter overlying mixed sand and gravel.

Clasts were primarily rounded to sub-rounded quartzite derived from reworked alluvial deposits and the overlying glacial drift. Less than 5% of the clasts were of anthropogenic origin (fragments of brick, slag, coal and discarded metal artefacts). Based on the d50 value

the finest sample was classified as a fine sand and >60% of the samples were classified as gravel.

In general, particles < 10 microns (medium silt) were found to be rare, with all samples containing less than 3% fines. This is not uncommon as transport of finer particles are expected under river flow velocities commonly >0.5 ms^{-1} and > 1ms^{-1} during flood events (Ellis, 2002).

From the 31 No. samples analysed in the lab, the fine to medium red sands passed through a sieve size of 0.5 mm, but the alluvial yellow sand displayed particle sizes in excess of 0.5 mm. This vertical stratification in particle size makes it difficult to characterize the grain size distribution of river bed as a bulk sample. It is recognised that the bulk sampling procedure employed will mix the different populations, resulting in inaccurate sample results.

Lab analysis displayed a clear separation of the two individual strata observed. Grain-size distribution curves were constructed to allow estimates of hydraulic conductivity values, for the individual layers of strata. The lowest range of hydraulic conductivity determined were from the uppermost layer (yellow sand) which displayed a more moderately well sorted particle distribution, compared to that of the finer grained, well sorted lowermost layer (red sand)

To obtain accurate results for the pavement size distribution, a surface oriented sampling technique has been employed over a 25 m and a 12 m section of the study area by means of grid and aerial sampling. The findings were used to support the results obtained from sediment and core analysis. (See Figure 35 and Figure 36)

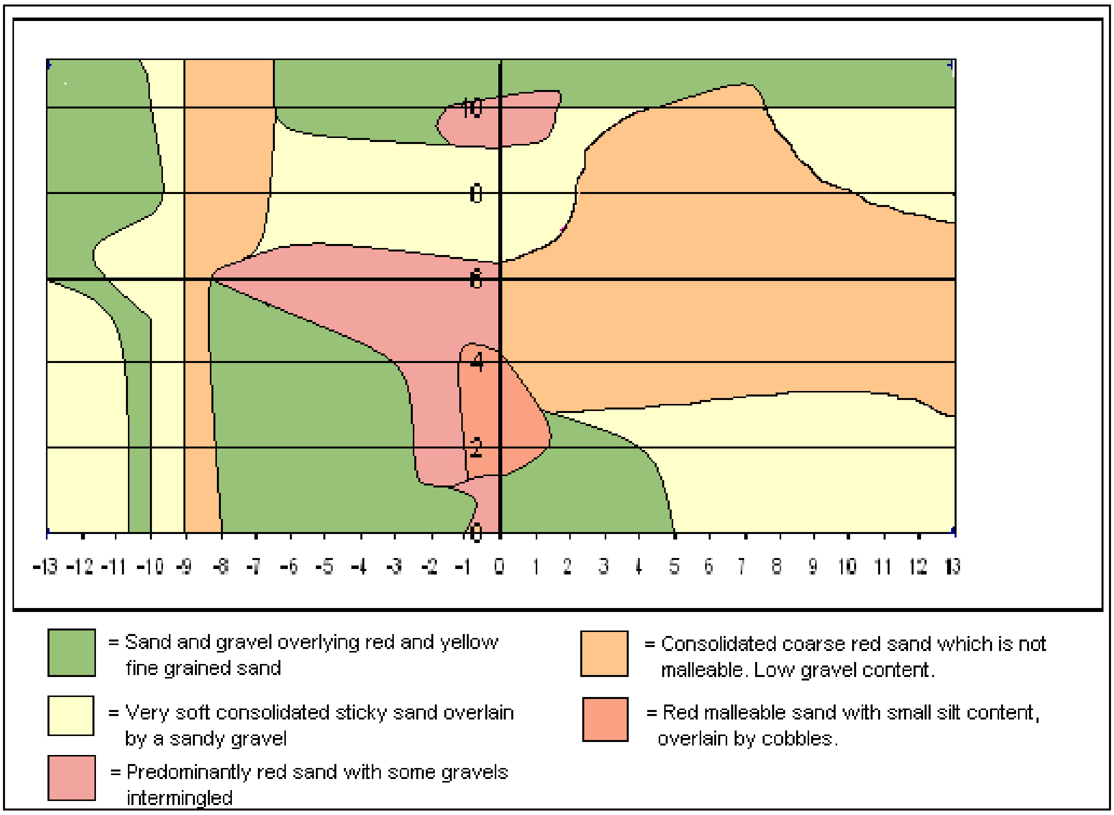

Figure 35: Detailed Aerial survey of 25 meter river section at XSL11

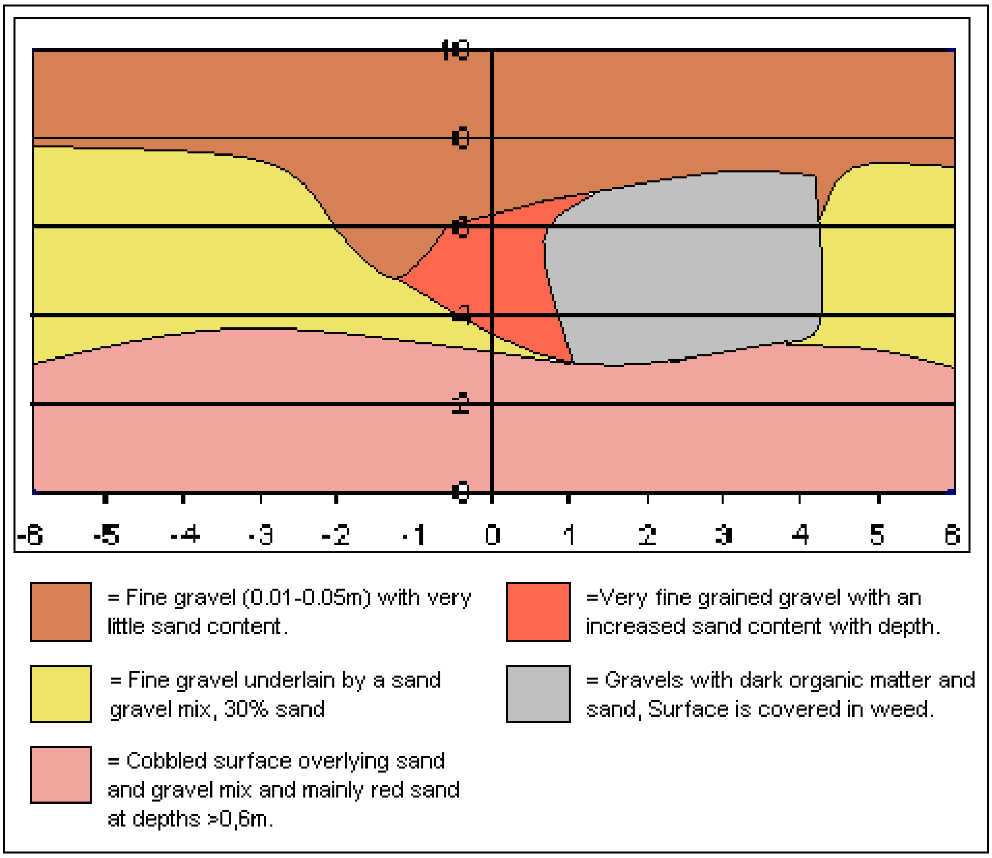

Figure 36: Detailed Aerial mapping of 12 m section at XSL9

Conductivity values were calculated using the Hazen (1911) and Shepherd (1989) methods. Both methods were found unsuitable for the gravel, giving unreasonably high values of conductivity. Estimates of hydraulic conductivity for both methods are displayed below (Table 8), along side previous results from other work.

	River bed conductivity			
	Method used	**Sieve Analysis**		
	Shepherd K	**m/d**	**Hazen K**	**m/d**
Lydon 2006	Average	35.8	**Average**	62
Red Sand	Range	27.7 - 49.6	**Range**	38 - 98
	Shepherd K	m/d	**Hazen K**	**m/d**
Lydon 2006	Average	26.3	**Average**	34.8
Yellow sand	Range	24.8 - 27.7	**Range**	20 - 49.8
	Shepherd K	m/d	**Hazen K**	**m/d**
Moylett 2001	Average	31	**Average**	41
Bulk sample	**Range**	11.0 - 91.0	**Range**	22 - 94
	River Bank conductivity			
			Hazen K	**m/d**
Caudell 2000			**Average**	6

Table 8: Results for hydraulic conductivity (K) using sieve analysis compared to results from previous work by Ellis, (2001) and Moylett, (2001).

The conductivity values derived from the particle size analyses are on average an order of magnitude larger than those derived from the field falling head tests. One reason for this is that the sieve analyses provide a bulk permeability for a mixed sample over a large vertical range (.>200 mm). Sieve analysis does not account for stratification of the bulk material and the degree of packing of the grains will affect the results.

Flood defence investigations by the Environment Agency (Caudell, 2000) gave mean conductivity of 6 md^{-1} from Hazen analyses of 6 particle size distributions and a mean of 4md^{-1} from Hvorslev analyses of rising head tests for these riverbank deposits.

5.5. Porosity

In total 10 No samples were used to gain an estimate of the riverbed porosity value.

The results ranged from 32.99 percent to 38.56 percent, with an average porosity value of 35.93 percent and a geometric mean value of 35.86 percent.

The degree of packing is also thought to affect these values as disturbed samples are used to obtain these results.

5.6. Stream Channel Morphology

Cross-sections of the riverbed can be used in conjunction with sediment cores to derive a good understanding of the riverbed thickness and geomorphology.

9 No. Environment Agency cross-sections conducted as part of the Flood Defence works were available after Ellis (2003). These were used in conjunction with 12 No. riverbed cross-sections collected during the field work. The absolute depth of each installation was calculated. The cross-sections were then used in estimating river cross-sectional discharge fluxes. The results of the average flux calculations are presented in Table 9.

Both positive and negative fluxes reflect gaining and loosing sections of river.

Profile	Average Flux to river m/d	AVERAGE Flux per unit stretch of river(m2/d)	AVERAGE Flux per 25m stretch of river (m3/d)
8	-0.237946713	-2.403261799	-120.1630899
9	1.610765198	18.04057022	902.0285111
10	-0.382826045	-3.598564821	-179.9282411
11	0.161082606	1.771908662	88.59543308
13	0.245096525	2.573513514	128.6756757
		Sum	819.2082888

Table 9: Table showing Average estimated flux through the riverbed

The complexities of the hyporheic sediment are apparent from the large variations in flux measurements. The method of calculation is very sensitive to large values of hydraulic conductivity. The conductivity measurements are all taken at discrete points in the river bed

and do not always reflects the permeability of the area the value is applied to. The unit measurement of flux is multiplied by the total river width at the point of sampling and subsequently multiplied by a stretch of river, in this case 25m.

The scale at which the measurement is being applied to, is not representative of the scale the measurement was taken at, large errors may result.

To refine this method stream gauging was conducted, to compare with flux estimates calculated from the piezometer measurements.

Although stream discharge measurements were carried out at 3 No. profiles (XSL 9, 10 and 11) along the study reach, the results proved inclusive. The count values obtained from the flow gauge was severely hampered by vegetation attaching itself to the impellor. Although areas with limited vegetation were chosen for the technique, floating algae was a constant problem during the data collection. Thankfully the piezometers have enough flexibility to allow the algae to slide past; unfortunately only tiny strands of vegetation were required to clog the impellor, decommissioning the gauge for several minutes until the measurements could be repeated.

It is advised that this method be tested during the winter months when vegetation has regressed. Alternatively some studies have used stream weir gauging where possible. Gauging larger rivers such as the Tame may prove problematic.

5.7. Falling Head Test Analysis

Falling Head tests were conducted in 27 of the 37 No. piezometers installed during the fieldwork. Access to ten piezometers installed between profiles 14 and 18 was prohibited shortly following their completion; as a result only estimates of permeability could be obtained from the 27 No. piezometers between profiles 7 and 13. The results of 2 No. falling head tests in the riverbed piezometers failed or proved inconclusive when tested. The tests

were recorded as having failed when no percolation of water occurred through the open section. Several reasons may account for this type of failure. It is likely to have been caused by either damage to the screened section during the installation procedure or as a result of encountering low permeability strata.

The 25 No. falling head tests that proved successful were analysed using the Hvorslev method (Hvorslev 1951). The method covers a large number of conditions. The procedure is based on the assumption that the effect of soil compressibility is negligible.

Values obtained for the bed conductivity ranged between 0.398 $md^{-1}$and 3.88 md^{-1} for the deep installations (>0.5 m) and gave an average of 1.62 md^{-1} and a geometric mean of 1.45 md^{-1}. (Appendix 9)

Shallow piezometers installed at 0.30 m yielded a range of between 1.8 and 7.9 md^{-1}, an average of 5.9 md^{-1} and a geometric mean of 5.33 md^{-1}.

These findings were consistent with that of Ellis (2003) during his extensive research

Table 10 below displays the results for the in-situ falling head tests, compared to estimates from previous work.

River bed conductivity Falling Head		
	(Hvorslev)	**m/d**
Lydon 2006	Average	1.68
Deep Piezometers	Range	0.39 - 3.88
	Geometric mean	1.44
	(Hvorslev)	**m/d**
Lydon 2006	Average	5.9
Shallow Piezometers	Range	1.84 - 7.86
	Geometric mean	5.33
	(Hvorslev)	**m/d**
Caudell 2000	Average	4

Table 10 : Estimates of hydraulic conductivity based on falling head tests conducted in the river bed, compared to results from previous work.

The conductivity estimates are very site specific but showed good repeatability, a large sample density was employed to determine average conductivity adequately. The spatial variation in conductivities along the study reach may reflect the changes in the sediment type observed at some locations, for example, high values at XSL 8 are believed be associated with coarse gravels. This section was typically loosing throughout the study period and was found to contain a large proportion of coarse gravel. The river bed is at a relatively high elevation at this point, this is reflected by a shallow depth of water and high flow velocity when compared with the rest of the river. The following schematic has been constructed following stage, bed elevation and ground water measurements taken on various days during the field work period. (Figure 37)

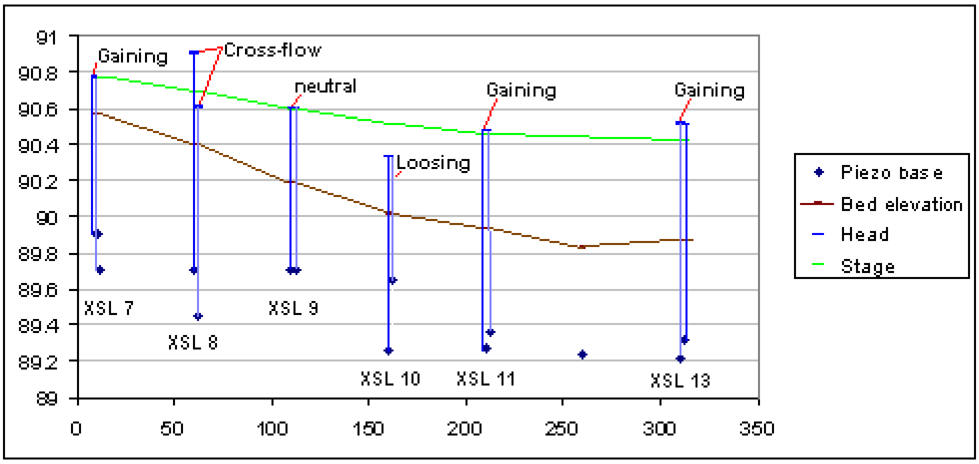

Figure 37: A conceptual understanding of the losing and gaining sections of the River Tame following field measurements

A complete selection of schematics is available in Appendix 1, which display the variations in head and stage with time. It is interesting to note at XSL8 both gaining and loosing portions were observed on several occasions that would suggest some cross flow is occurring at this point. Initial concerns were that the piezometers were not installed to sufficient depth but a review of the data shows that the deeper piezometer at XSL 8 was always the monitoring point which displayed loosing conditions.

XSL10 occasionally displayed loosing conditions, but did reach neutral and occasionally gaining conditions. Piezometers installed along XSL11 equally displayed a variety of conditions, but on average exhibited gaining conditions. (Location XSL11 is the position advised for the installation of the Dipole abstraction well)

5.8. Sediment sampling /Coring

The core procedure employed during this study was found to be unproductive.

Cores were attempted at profiles XSL 11, 10 and 8 as shown in Figure 38 to Figure 40.

The recovery of core was only successful for 2 out of 5 cores taken.

Cores numbered 1, 2 and 3 failed to recover a sediment sample. Bed sediments in these areas consisted predominantly of gravel and occasional cobble.

In each case the steel pipe was driven until further penetration was not possible. Typical depths reached ranged from 0.30 to 0.50 m.

The sample was removed from the core tool by pushing the plunger into the steel tube forcing the plastic liner out. The sample tubes were labelled top and bottom and taped at either end securing the sample until logging was conducted in the laboratory.

Sample descriptions along with photos found in Appendix 7.

In general the sample descriptions coincided with the sample descriptions collected for the sieve analysis.

The sampling procedure used could be improved on by manufacturing small rings that would rest securely on the driving end of the tube to minimise damage caused to the inner plastic liner.

Although the samples were not damaged during the driving procedure the liners always became crushed at the top due to the lack of support of the liner at the open end. However the method was simple and easily repeatable.

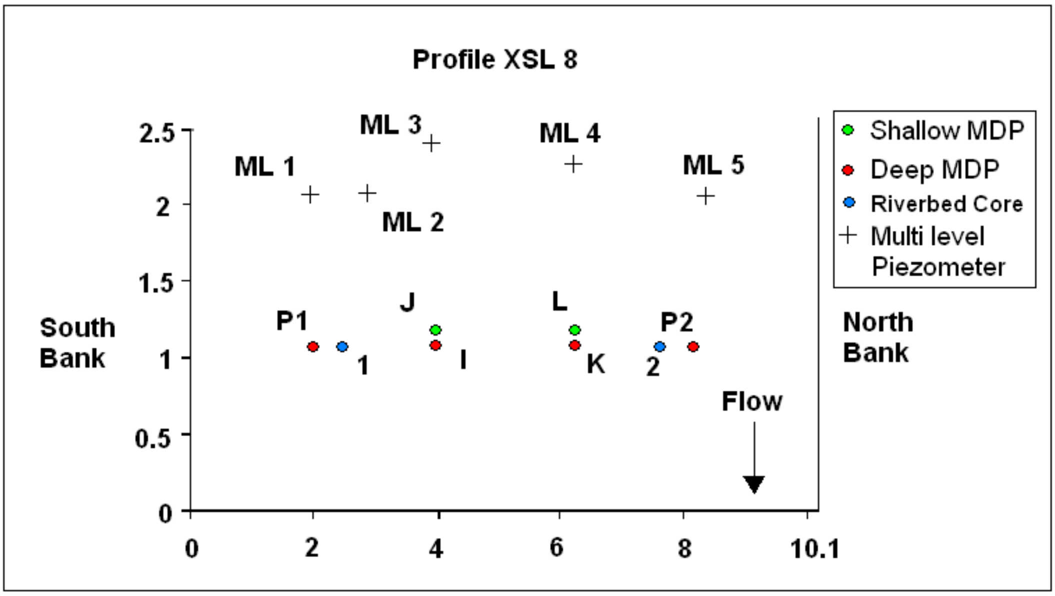

Figure 38: Location of riverbed core samples at XSL8

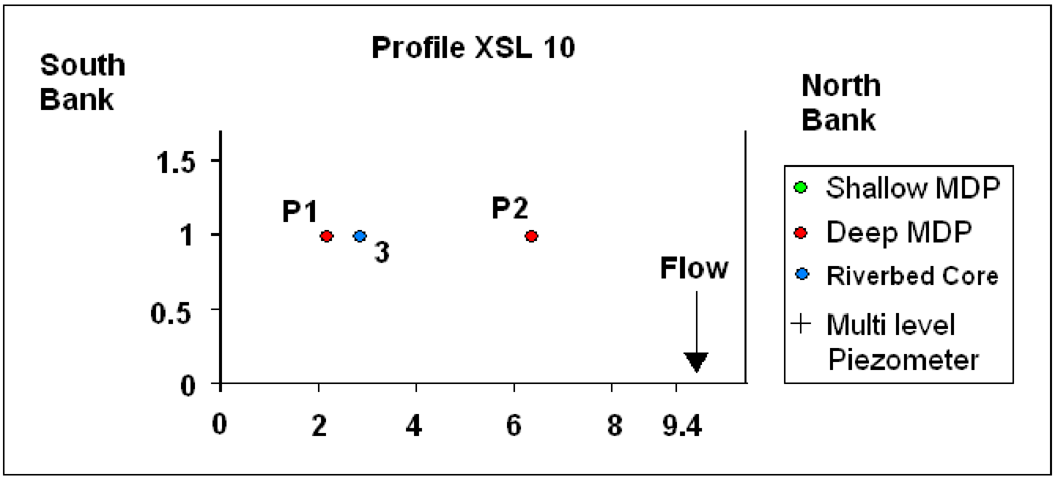

Figure 39: Location of riverbed core samples at XSL 10

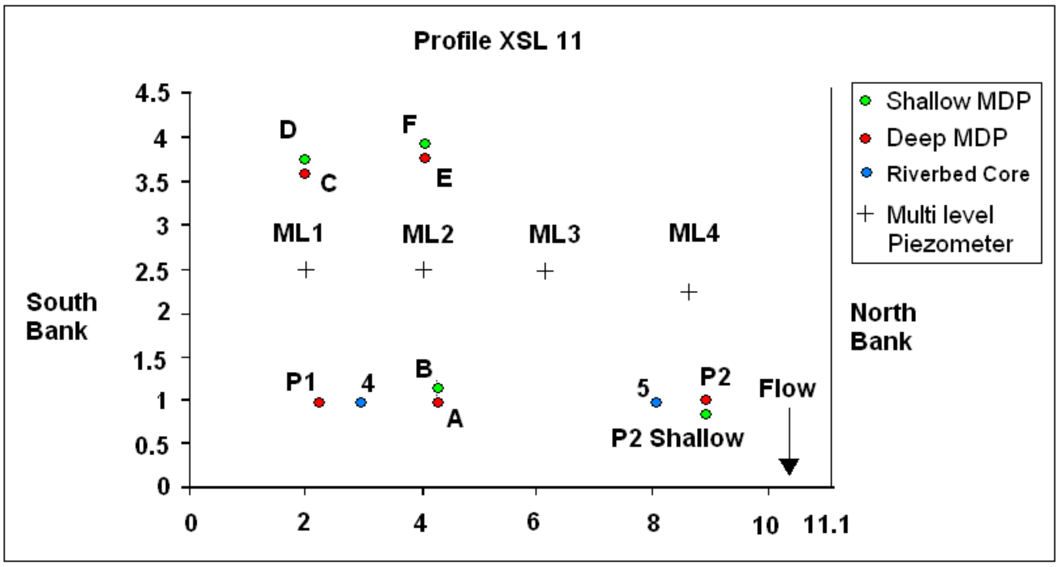

Figure 40: Location of riverbed core samples at XSL 11

5.9. Conclusion

Estimates of conductivity for the discrete layers of bed-sediments, using sieve analysis technique were not consistent with the estimates of conductivities derived using falling head tests in the riverbed piezometers. The Hazen and Shepherd values of >20 md^{-1} may be more representative of the horizontal conductivity of the riverbed.

The geometric mean value for hydraulic conductivity however (1.44 md^{-1}) from falling head analysis is considered to provide a good estimate for the combined vertical and horizontal riverbed hydraulic conductivity.

Overall no major abstractors were found to be pumping sufficient quantities of groundwater to influence the future Dipole experiment. The EA and Severn Trent Water borehole logs obtained by Ellis (2003) were useful in deciding which drilling techniques would be suitable for the proposed works. Stratigraphic correlation and analysis permitted average thicknesses and expected depth to each of the four strata to be determined. The following table is a summary of the average strata thickness (assuming horizontal Stratigraphic layers)

Elevation mAOD	Stratigraphic unit
GL – 91.56	Made Ground
91.56 – 89.72	Sand and Gravel
89.72 – 87.97	Weathered Sandstone
87.97 – 18	Sandstone

Figure 41: Summary of average strata thickness

A complete list operational boreholes along the study site are included in Appendix 5.

Dipping records for all of the boreholes in the area provided average groundwater levels, for use in calibrating the numerical model. The installed MDP's were useful in assigning groundwater head values directly beneath the modelled River Tame. Stage and riverbed elevations also provided a large dataset to model a detailed river boundary. The complete dataset used to construct the river boundary is included in Appendix 10.

Chapter 6 Modelling

Modflow is a three-dimensional finite-difference groundwater flow model created by Mcdonald and Harbaugh (1986). Groundwater flow within an aquifer is simulated in MODFLOW using a block-centered finite-difference approach.

MODFLOW is used here to analyse the effects of a proposed abstraction injection experiment adjacent to the River Tame. The package was an obvious choice as it has the added benefit of being able to simulate flows from external stresses such as flow to wells and flow through riverbeds.

Many groundwater wells that pump water from aquifers located close to streams and rivers are in direct hydraulic connection with the underlying ground-water system. Pumping from these wells reduces stream-flow by capturing ground water that would otherwise discharge to the streams and, in some cases, by drawing water out of the streams and into the adjoining aquifer. Presented herein is the development of the steady state model and its use as a predictive tool to discover the optimal design setup.

5.10. The basic conceptual model

Field and archive data allowed a conceptual model of the groundwater flow system to be established, describing the aquifer geometry, regional groundwater flow direction, its interaction with the River Tame, the local boundary conditions and estimates of the hydraulic parameters.

Some simplifications were made during the conceptualisation of the system. These simplifications can be justified on the grounds that although they simplify the model, they allow the production of acceptably accurate results that will be the foundation for the design of the Dipole well system.

5.11. Aquifer Geometry

Ancient borehole records provided good estimates of the local unconfined aquifer thickness. Industrial groundwater demand has been the main driving force behind the installation of many fully penetrating wells within the Triassic unconfined aquifer.

11 No. borehole records from deep boreholes drilled within a few hundred meters the closest being approximately 450 m from the proposed site; suggest an average aquifer thickness of 76 meters (Butler and Lee, 1943). The effective base of the sandstone (the Hopwas Breccia) aquifer is considered impermeable for this model. Four distinct lithological units were recognised during this study. Each unit is represented within the model. Additional layers have been added to facilitate refinement of the model in areas where targets and well screens are positioned. All of the layers are horizontal which is consistent with borehole records (only shallow gradients exist between strata). Table 11 shows the distribution of the model layers and the stratigraphical unit each layer represents.

Layer	Top mAOD	Base mAOD	Stratigraphical unit
1	100	91.65	Glacio-Fluvial Deposits
2	91.65	89.72	River Terrace deposits (first terrace)
3	89.72	88.605	Weathered Kidderminster Fm/ Wildmoor Fm
4	88.605	87.97	Kidderminster Fm/ Wildmoor Fm
5	87.97	83	Kidderminster Fm/ Wildmoor Fm
6	83	78	Kidderminster Fm/ Wildmoor Fm
7	78	68	Kidderminster Fm/ Wildmoor Fm
8	68	18	Kidderminster Fm/ Wildmoor Fm
Total model thickness (m)	82		Hopwas Breccia – effective aquifer base

Table 11: Model Layers and the stratigraphical units they represent

5.12. Boundary conditions

Local boundary conditions have been assigned based on the availability of data.

For this model it is important that the boundary conditions are placed at great enough distances, as to not influence the Dipole well system. Positioning a well in close proximity to a constant head boundary will result in inaccurate results, as the well will draw water from an infinite source.

Constant head boundaries were assigned based on Severn Trent and Environment Agency groundwater monitoring data for local boreholes (After Ellis, 2003).

The boundaries were set at distances great enough to ensure they had very little impact on the experimental well setup, while also generating the required hydraulic gradients. The NW-SE boundary runs parallel to the River Tame at a distance of 220m. The constant head boundary reaches a maximum of 93.80 mAOD to the North West and recedes at a shallow gradient of 0.0005 to the south east extremity of the model where it reaches a value of 93.40 mAOD.

A constant head boundary also runs parallel to the River Tame along the base of the model in the same orientation as the Northern boundary. It reaches a maximum head of 92.70 mAOD along the central portion and declines towards the Tame to the North West to a head value of 89.42. The South Eastern margin holds a constant head of 92.26 mAOD. The western and eastern most boundaries are represented by no-flow boundaries as they dissect the River Tame. For a visual representation of these model bounds, consult Figure 42.

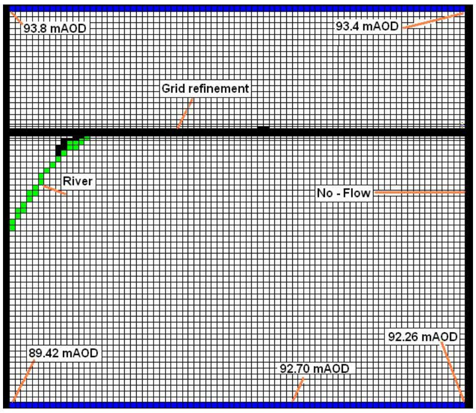

Figure 42: Model Boundary Conditions

5.13. River Tame

The River Tame traverses the central portion of the model area and is hydraulically connected to the aquifer. The river partially penetrates an unconfined aquifer and acts as the primary sink for the local groundwater system, with the majority of the river displays gaining conditions. A comprehensive database of the rivers physical characteristics was complied during field data collection. Absolute bed elevation values were assigned to 1 m x 10 m length cells across the channel following cross-section measurements collected in the field. Parameters were extrapolated between the points of measurement. (Every 50 m)

Stage measurements were taken over a one day period along with groundwater head measurements from MDP's in the riverbed. This provided a snapshot of the aquifer-river system that could be used to calibrate the model. The groundwater head measurements from the MDP's were taken just beneath the River Tame. A separate refinement layer (Layer 3) of thickness 1.11m was used to allow insertion of the model targets directly below the riverbed. An initial bulk conductance value of 0.3m/d was assigned to the riverbed sediments based on the lowermost values yielded from falling head tests conducted along the river reach.

A riverbed thickness of 0.60m was assigned to the entire river. Although it is recognised that the riverbed thickness varies across and along the river reach, this value was considered sensible following analysis of sediment cores and hand taken samples.

Extremity	Stage	Bed Elevation	Bed Thickness	River Width
North-Western	91.21	90.70	0.6	10
South-Eastern	90.08	89.15	0.6	11.5

Table 12: A basic idea of the model parameters used in constructing the River boundary condition

For a full summary of the parameters used in constructing the River boundary consult Appendix 10.

Modflow’s river package simulates the interaction with the groundwater system where flow between the two is dependent upon the head differential between the groundwater and the river stage and a stream bed conductance term as follows:

$$C = K (L_R \times W / b)$$

Where:

C is the riverbed conductance (m2/d)

K is the hydraulic conductivity of the sediments (m/d)

L_R is the length of the river reach (m)

W is the river width (m)

b is the thickness of bed sediments (m)

If the groundwater head value drops below the specified bed elevation, the flow from the river to the groundwater system is considered independent of groundwater levels and will remain constant at a rate determined by the head difference.

Modflow assumes that the area immediately below the river is saturated and permeable. In addition the riverbanks are assumed vertical and impermeable, only allowing flow to occur through the river base.

Targets of measured head within the riverbed were placed within the lowermost bounds of layer 3 which computed head values immediately below the streambed.

5.14. Aquifer Hydraulic Parameters

Initial estimates of hydraulic conductivity values for the hydrostratigraphic units were assigned following consultation of recent geological maps, memoirs, publications & shallow borehole records. A database of hydraulic properties was generated for the model calibration. The initial hydraulic properties used to run the steady state model are presented in

Stratigraphy	Lithology	KH/KV Ratio	KH (median) m/d	Sy	θ
Glacio-Fluvial Deposits	Sand and Gravel	1.54	20	0.24	0.33
River Terrace deposits (first terrace)	Sand and Gravel	4.26	20	0.24	0.33
Wildmoor Fm	Sandstone	1.88	1	0.13	0.27
Weathered Wildmoor Fm	Sandstone	1.88	1	0.13	0.27
Kidderminster Fm	Sandstone	1.81	3.5	0.19	0.29

Table 13: Literature hydraulic parameters for the modelled stratigraphy after Thomas, 2001,Allen et al., 1997, Krusman and Ridder, 1994 and Freeze and Cherry, 1997

The model contains two principal aquifer formations. Although the formations are gradational along their contact, sufficient data was available to allow them to be modelled as separate units (Wildmoor Formation & Kidderminster Formation).See Figure 43.

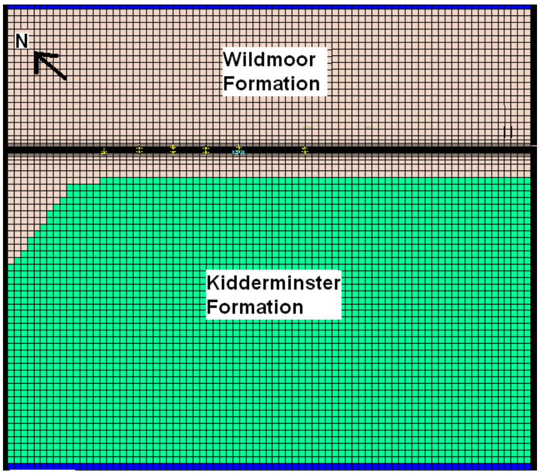

Figure 43: Boundary between the two principal sandstone aquifer formations

The first layer is represented by the Glacio-fluvial deposits which are not considered confining. However due to recent redevelopment of much of the area, frequent clay and made ground has been added to the upper layer, resulting in a complex inhomogeneous unit.

Initial hydraulic parameters were entered as bulk parameters, shown under KH median in Table 13. The addition of anisotropic hydraulic conductivity values were added later using the ratios in Table 13.

Although practically redundant within this unconfined aquifer, a nominal Storativity value of 0.0001 was assigned to all the model layers.

Contoured water levels indicate the dominant flow direction to be towards the River Tame, with other minor flows from north to south along this study reach. As shown in Figure 44.

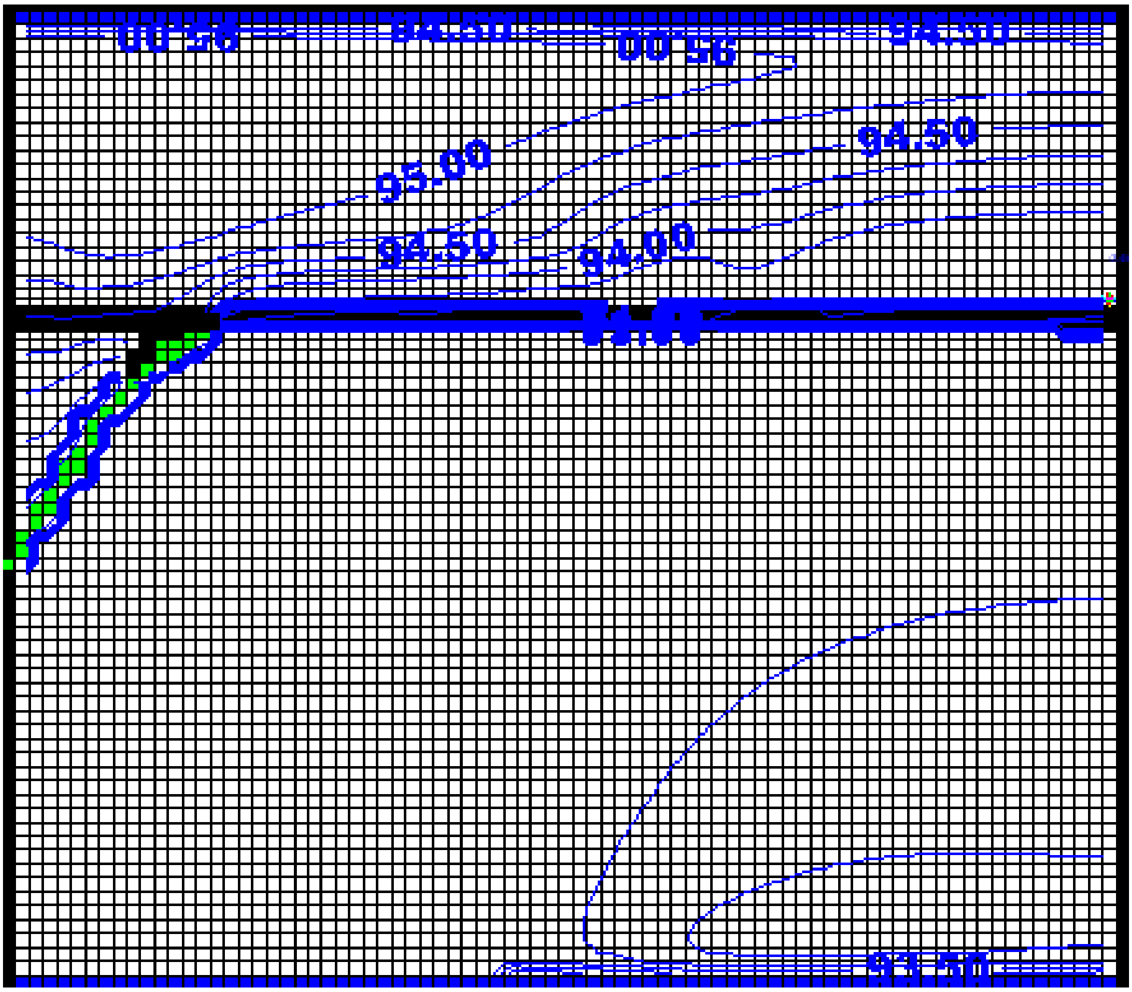

Figure 44: Contoured heads measured during study

5.15. Spatial Descritization

The study site is 1.15 by 1.04 km which contains the river Tame.

The model area was divided into a grid 115 columns and 104 rows of cells.

Each cell represents 10 meters in the x-direction, 10 meters in the y-direction.

The model contains 10,920 total cells with 6474 active cells. The total model surface area is 1.2 km2. The layer elevations are based on geological, hydrogeological and hydrological information collected during the field work and on literature review. The groundwater flow model includes heterogeneous values of permeability, porosity, and specific yield in the glacial outwash deposits resting on top of the unconfined aquifer under the River Tame.

5.16. Recharge

The estimated average regional recharge to the unconfined aquifer system ranges between 0.36 mm/d (Knipe et al., 1993) and 0. 45 mm/d (Greswell, 1992).

5.17. Abstractions

No significant abstractions were found to influence the model area. Only intermittent minor abstraction rates were found to be pumping to the south west (0.48 ML/d). No deep abstractions were represented during the modelling.

5.18. Model assumptions

The following is a list of the model assumptions:

- The River Tame only permits water though the base of the channel.
- Under normal conditions the aquifer supply's water to the River Tame.
- Inputs to the system including discharge sewers are ignored.
- No abstractions currently influence the model area.

5.19. Steady-State calibration

Steady-state model calibration was initiated using the initial parameter values obtained from studies in the area and further estimates from the slug tests conducted in the area.

All parameters entered into the groundwater model were given units of meters and days.

The model was calibrated using 22 targets. The targets represented wells where water-level measurements were available and head values collected from the mini drive point piezometers installed in a tin layer immediately below the riverbed.

The water-level data was collected at the end of the study period on the 14^{th} of August, 2006, to try and achieve a single snap shot of the aquifer system. Calibration of the model was approached with a deliberate reluctance to add complexity to the model unless clearly warranted.

Deep monitoring wells		
Borehole reference	Layer installed	Value
27	5	91.17
26	5	91.16
45	5	92.10
44	5	92.06
AC	5	92.058
21	5	92.60
23	5	91.075
24	5	91.26
25	5	91.36
AB	5	91.70

Shallow MDP's		
MDP reference	Layer installed	Value
XSL7 P1	3	90.766
XSL7 P2	3	90.76
XSL8 P1	3	90.902
XSL8 P2	3	90.606
XSL9 P1	3	90.594
XSL9 P2	3	90.599
XSL10 P1	3	90.621
XSL10 P2	3	90.526
XSL11 P1	3	90.456
XSL11 P2	3	90.48
XSL13 P1	3	90.517
XSL13 P2	3	90.51

Table 14: A complete list of targets used to calibrate the steady state model. (Note values are in mAOD)

Best estimates of hydraulic conductivities were initially assigned to each model cell.

The model was calibrated through trial and error parameter adjustment. The most sensitive parameters were identified through an initial sensitivity analysis, and these parameters were adjusted until the model approached calibration.

Accurate results could not be obtained initially for the targets placed beneath the river bed because of relatively high groundwater levels adjacent to the River Tame. These high levels produced very steep head gradients. These steep gradients were estimated by hand to be on average 0.025. The steep gradients are believed to be a result of radial convergence at the river. The constant head boundary conditions were lowered by 0.50 m to try and achieve shallower gradients (0.01) and to reach calibration of the targets immediately below the river. Although this will result in a deviation from the natural system, the changes were taken to

avoid the need to place unrealistic low conductivity zones around the river. In doing this a closer replication of the observed head values below the river bed was achieved, with the main required impact area (XSL11 P1 installed below the river) having a residual of 0.01m. These heads would then be altered through pumping adjacent to the River Tame.

Despite the accuracy of the model surrounding the river, there are significant discrepancies between measured and computed steady state heads for boreholes at greater distances from the river. Residual differences reach a maximum of 0.7 m for BH 23, approximately 450m down stream from the proposed drilling site. The failure of the model to accurately represent observed water levels in these areas was not resolved by changing hydraulic conductivities. In order to replicate the steep gradients along the river, a zone of low conductivity could be used, effectively reducing the drainage from the system at that point. These changes were considered unrealistic and would have affected the modelled pumping scenarios.

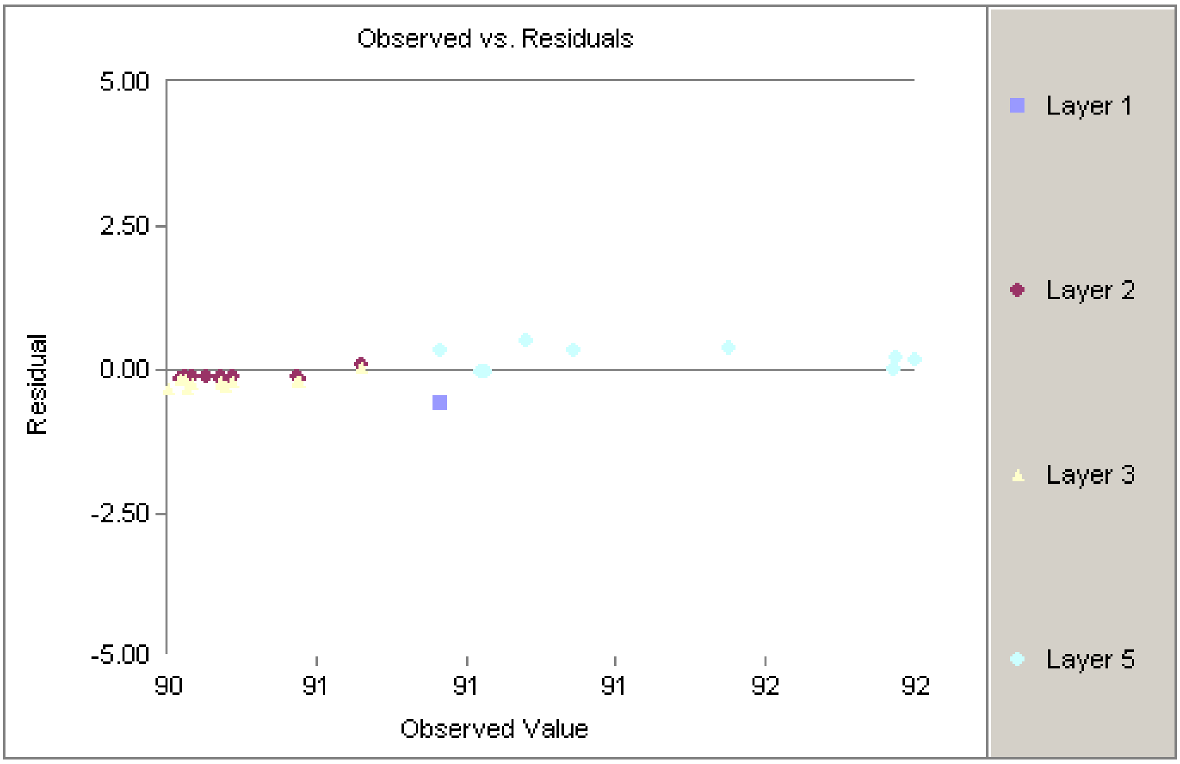

Figure 45: Observed Vs. Computed Target Head Values prior to calibration

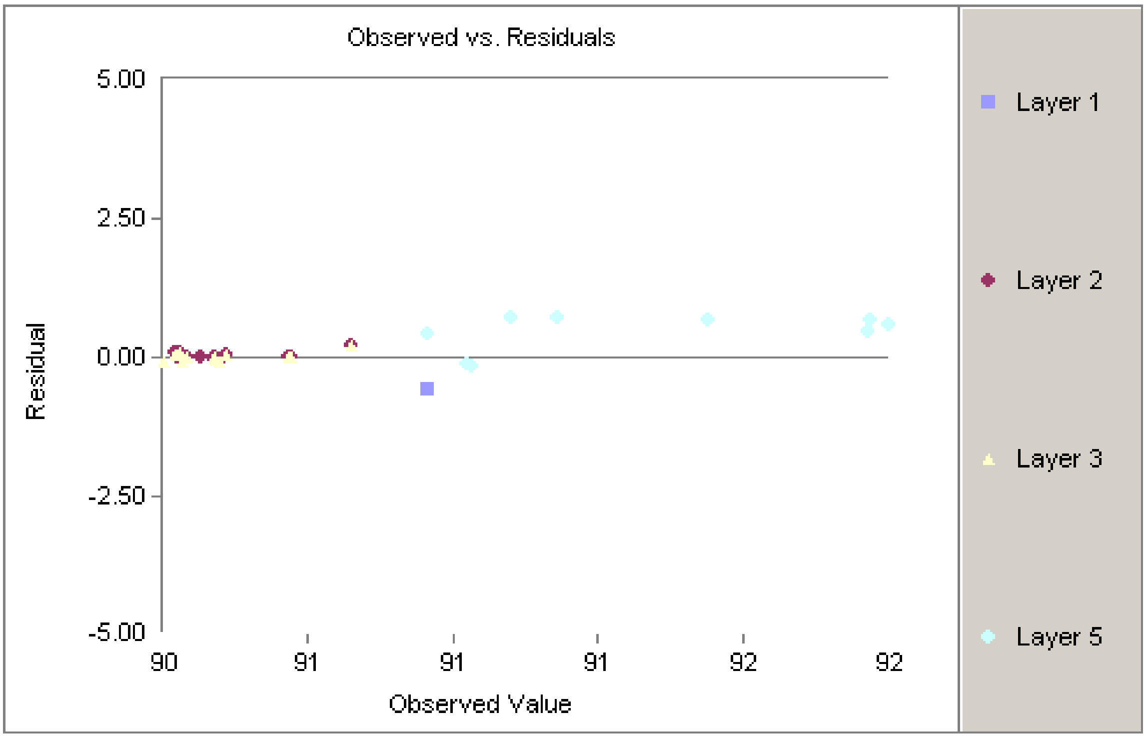

Figure 46: Observed Vs. Computed Target Head Values after to calibration

The graphs above show how following calibration the computed target values in layer 3 (below the riverbed) are within a few millimetres of the observed head values.

However in order to achieve the calibration of the targets along the river the targets at greater distances suffered and deviated slightly further from the observed values.

A mass balance of the two pumping scenarios revealed that between 18 and 37.5 percent of the water drawn to the well was derived from the river. The overall mass balance contained errors less than 0.01 percent.

5.20. Conceptual Understanding of flow reversal

For a rectangular channel of ideal geometry, with a permeable base and impermeable channel sides, a "flow reversal" can be generated using an appropriate drawdown to generate a downward flux.

Suppose your river system is gaining, the drawdown required to generate a flow reversal will initially entail generating sufficient drawdown to produce equilibrium conditions. Under these equilibrium conditions the aquifer head equals the channel stage, at any point along the radius of impact. (See Figure 47)

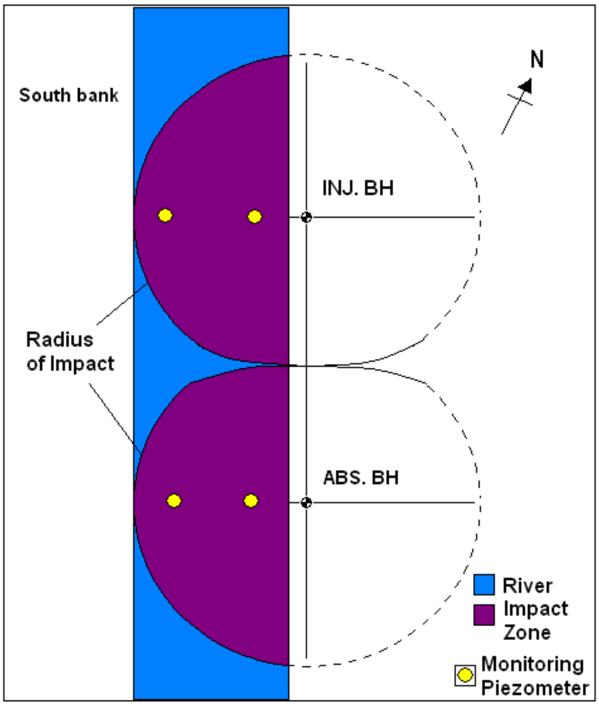

Figure 47: Impact Zone generated by the Dipole well setup

The area between the radius of impact and the abstraction well can be termed the zone of impact (ZI). Within the ZI, flow paths to the well will be shorter in distance and the downward gradient generated by the abstraction well is greater in this area compared to the area along the radius of impact (RI).

Any additional drawdown along the RI will generate a downward gradient encouraging the channel to become influent from its base to the underlying aquifer.

5.21. Well Positioning

By placing the wells as close to the river as reasonably possible, the River Tame will be more responsive to changes in head gradients, as it would be if the wells were placed further away from the river.

A comprehensive site assessment, confirmed that a well could be located comfortably 1-2 m from the north river bank. The diameter of the river is 11 m and 9.4 m at XSL 11 and XSL10 respectively. By positioning the wells at 30 and 50 meter separations the impacted stretch of river will meet the original criteria stated at the beginning of this chapter.

Both gaining and loosing sections of river will be generated over a short distance. The gradients between the wells will increase at the point of inflection between the wells. This is represented by the constricted equipotential lines show in Figure 48.

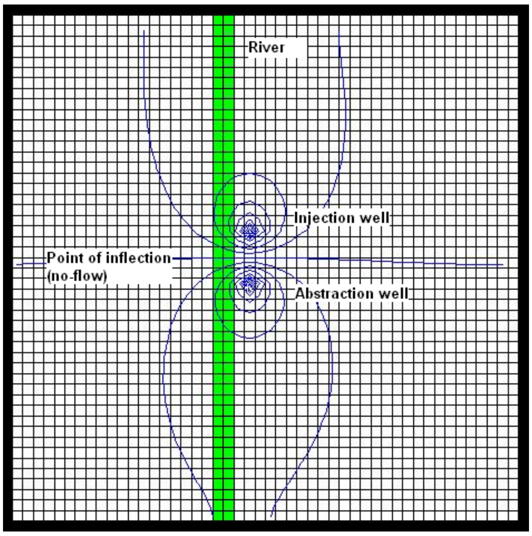

Figure 48: Increased gradients along the shortest distance between the abstraction and injection well

Travel times for flow lines within the ZI to the well will be greater than those along the RI or outside the radius of impact. Contaminant tracer tests are proposed during the future Dipole experiment.

5.22. The models use as a predictive tool

Following steady state calibration the flow model was used to simulate various Dipole setups at different well separations, screen depths and lengths as well as distance from the River. The specified design criteria in chapter one are repeated here briefly for the readers' convenience.

- Re-injection of the abstracted groundwater during the pumping tests is favoured over direct discharge to the river Tame, therefore the design requires a loop system or 'Dipole' setup to minimise the impact on the overall experiment.
- Optimal well spacing will be imperative to maximise the area of river affected. The abstraction – injection well setup must impact a large enough section of river approximately 30 - 40 m to allow adequate assessment of the hyporheic zone and its potential to attenuate various chemicals during the testing period.
- For the initial assessment of the well setup a mixing zone of half a meter must be generated within the bed sediments, over pumping periods of varying test lengths.
- A third well is required at an alternative separation to the first two wells. This well will lend itself as optional pumping-injection arrangement.
- The pumping rate must be minimised while also maximising the area of river that will be impacted.
- The well screen must be positioned at an optimal depth to facilitate the preceding requirements.
- A reliable groundwater model should be used to validate the design.

The complex conditions generated when both injection wells and discharge wells are operating in such close proximity to a river, that is hydraulically connected with the underlying aquifer system, are difficult to analysis analytically as several variables are

dependant on one another. For this reasons modelling provides a less complex means of assessing the likely development of such a pumping setup.

Some of the most sensitive variables were tailored to reduce the number of undefined values, during the course of the modelling. The discharge rate is constrained significantly by the power supply available on site. A 230V mains power supply is expected from a nearby street lamp. This will limit the size of the pump that can be used at this site.

A discharge rate of between 1 to 4.5 L/s was deemed adequate to generate the small flow reversals to generate a mixing zone of 0.5 to 1 m, over short test periods (30days).

Two discharge wells, one abstraction and the other an injection well were positioned 50 m apart and 1.5 m from the River Tame. Both wells were initially assigned the same discharge rate, (- 86.4 l/s and 86.4 l/s respectfully). Initially the well screens were both positioned at 70mAOD and contained a screen length of 5 m.

In order to estimate the likely drawdown required to generate the flow reversals over varying test lengths, a series of simple mathematical solutions were used:

(Note the following calculations are made using stage and groundwater levels taken at XSL11 P1 on the 14/08/2006. XSL11 P1 is the monitoring point, approximately 10 m from the abstraction well, where the flow reversals are intended to be generated.)

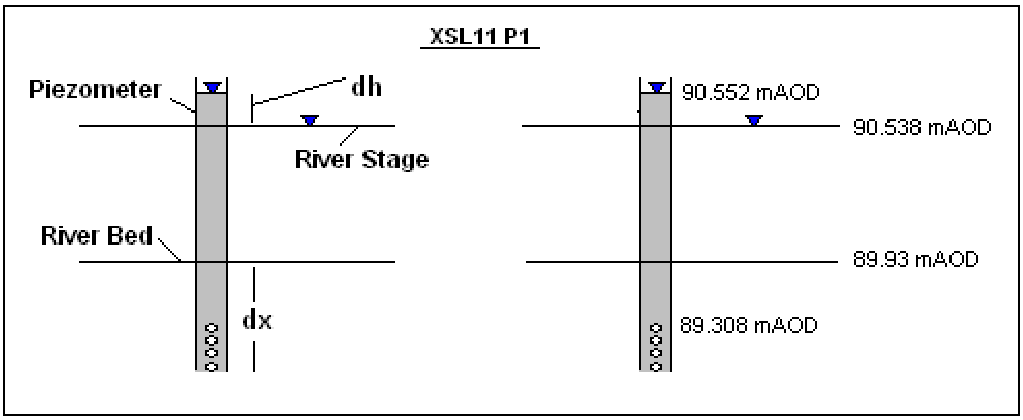

Figure 49: Measurements used to calculate the required drawdown

To determine the change in head required to generate flow reversals at a single point along the River Tame, we must first calculate the downward flux that will permit 0.5m penetration of the riverbed sediments over a test period of one month (30days).

$$V = d / t$$

V corresponds to the required velocity (md-1)

d is the penetration distance (m)

t is time duration of test (d)

Example: $V = 0.50 / 30$

$V = 0.016$ md-1

From this a new downward flux can be estimated:

$$q_{NEW} = V \times n_e$$

q_{NEW} is the downward flux required to generate a flow reversal

V corresponds to the required velocity (md-1)

n_e is the effective porosity of the bed-sediment (33% from laboratory analysis)

$$q_{NEW} = (0.016 \times 0.33) = -0.00275 \text{ md-1}$$

The flux is negative because it corresponds to the downward flow. To solve for the new aquifer head $H_{A\ new}$ and hence the required drawdown, we rearrange Darcy's law as follows:

$$q_{NEW} = \frac{K(H_{A\ new} - Hs)}{dx} \longrightarrow \frac{q_{NEW}(dx)}{K} + Hs = H_{A\ new}$$

Where:

q_{NEW} is the flux through the bed-sediments (md-1)

K is the hydraulic conductivity units (md-1)

$H_{A\ new}$ refers to the new head required in the aquifer (m)

Hs refers to the stage measurement (m)

dx represents the installation depth of the piezometer (m)

$$H_{A\ new} = [- 0.00275\ (0.655) / 1.329] + 90.538$$

$$H_{A\ new} = 90.5366\ \text{mAOD}$$

The required drawdown (Sw) therefore is given by H_A - $H_{A\ new.}$

$$Sw = 0.0154\ m$$

An excel spread sheet was set up to calculate the required drawdown for tests of varying lengths and depths of penetration (See Appendix 11).

A new target was assigned to XSL11 P1 (Piezometer target in river bed) based on the drawdown calculated above. The Dipole setup was then run using this drawdown as a target value to be achieved during pumping.

Initially the pumping rate was fixed at 1 L/d and the well positioned at the closest realistic distance of 1.5 meters from the river. The screen length and position were the first two parameters to be analysed. The top of the screen was kept below the river bed elevation along the impacted section of river at all times.

Results found that the model was not very sensitive to the well screen depth or elevation for the pumping rates used. Changes of one meter in elevation often only resulted in changes of a few centimetres of drawdown at the monitoring point XSL11 P1.

Similar results were obtained when altering the screen length; significant changes in drawdown were expected by shortening the screen, however only minor changes in drawdown occurred.

Even though the numerical model was set up to simulate the effects of pumping from a partially penetrating abstraction well by dividing the sandstone into layers, the modelled drawdown, was relatively smaller than it would when the wells were positioned at a few tens of meters from the river.

The problem was investigated by temporarily moving the abstraction well to a different location further away from the river to determine if this was affecting the results. Drawdown of around 2 m for a pumping rate of 1 L/s was observed when the well was moved just 10, 15 and 30 m from its actual location. This suggests that the river was acting as a recharge boundary and because the modelled river has a fixed stage elevation it can effectively act as an infinite source to the abstraction well, thereby suppressing drawdown in the aquifer.

The wells positioned directly adjacent to the river (1.5 m) containing a screen depth of 76 – 80 mAOD (roughly nine meters below the river bed) produced a drawdown close to the required values at the 1 L/d pumping rate (50 m separation of wells) Final adjustments to the pumping rate produced the exact required drawdown at XSL11 P1 along the south bank of the river. The model was largely sensitive to changes in discharge rate above all other parameters. A final discharge/injection rate of 160m3/d or 1.86 L/s generated the required flow reversals for a one month test period

Overall the drawdown is sensitive to the following parameters:

Screen length

Screen depth

Pumping rate

This process was repeated for a second alternative Dipole separation. A 30 m well separation scenario was chosen as it would still generate the hydraulically controlled conditions beneath the River Tame yet it would offer an alternative separation should scenario 1 be unsuccessful. The optimal well positioning and discharge rates are presented in Table 15. The injection well was assigned identical parameters to the abstraction well throughout the experiment.

* Elevations in mAOD

Parameters in meters	Scenario 1		Scenario 1	
Well Reference	**ABS 1**	**INJ 1**	**ABS 1(i)**	**INJ 2**
Well Separation	30	30	50	50
Screen Length	4	4	4	4
Top of Screen Elevation	80	80	80	80
Screen Diameter	0.1524	0.1524	0.1524	0.1524
Well Distance from North bank edge	1.5	1.5	1.5	1.5
Distance from Pumping Well to XSL11 P1	10.27	31.71	10.27	51.04
Sw at XSL11 P1 (30day test)	0.0154	0.0154	0.0154	0.0154
Sw at Discharge well				
Ground Elevation	92.377	92.701	92.377	92.915
Discharge rate (L/d)	-345.2	345.2	-160	160

Table 15: Optimal Dipole parameters based on modeling results

5.23. Transient modelling

The main reason for constructing the transient model was to estimate the time required for the Dipole setup to reach steady. Emphasis was placed on computing head values within the first hour of pumping and less emphasis days after the onset of pumping. Stress periods were assigned accordingly.

No of stress periods	Period length	No. of time steps	Time step multiplier
1	1	24	1
2	0.04166666	60	1
3	0.16666666	16	1
4	0.79166666	38	1
5	178	178	1

Table 16: Stress peroid setup for the transient model runs

The final optimal well designs developed during the steady state model runs, were tested to see how long it took for the pumping system to achieve steady state.
After observing how the river acted as a recharge boundary to the abstraction well, it was anticipated that seepage from the river, would result in delaying the accomplishment of steady state conditions.

The transient model consisted of 5 stress periods, each with different period lengths.
Stress period 1 was used to simulate one day of no pumping. Period two simulates the onset of pumping by the Dipole setup; the period length duration is one hour and has time steps each minute of that hour to capture the early time response to pumping. Stress periods 3 and 4 are used to represent the subsequent 4 and 19 hours of pumping, each containing time steps every 30 minutes. The final stress period was used to assess the see how long the abstraction well would take to reach steady state. 178 days is probably over compensating but the time steps are only one a day.
Results
The change in storage was calculated to be 0.00122 for the 30 meter separation scenario.
Monitoring wells were positioned in layer 3, directly above each well location, to monitor the drawdown and cone of impression with time created by the abstraction and injection setup.
This allowed modelled hydrographs of the drawdown to be generated. Steady state drawdown was achieved within 26 days of pumping for both of the Dipole well setups. The drawdown curves display a leaky type response, illustrate by the time required to reach steady state. This is believed to be as a result of water being drawn from the river. The monitoring point along the radius of impact XSL11 P1 was also monitored with time. The river bed piezometer also showed a leaky type response.

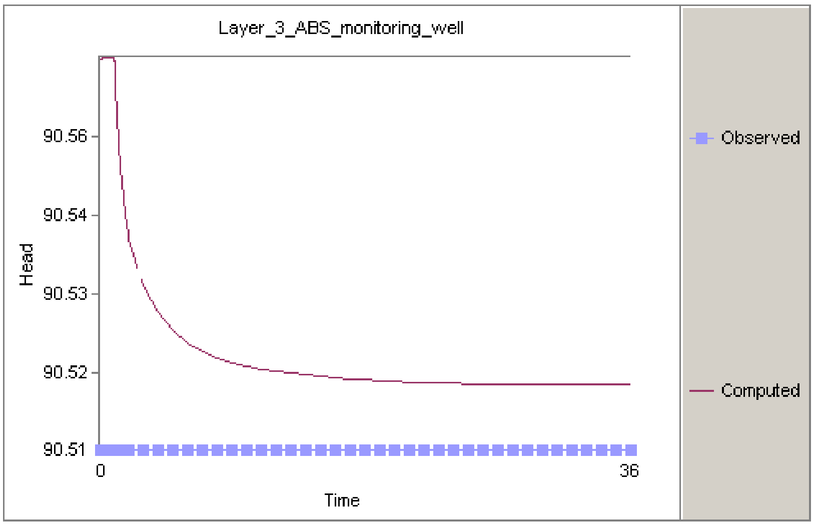

Figure 50: Layer 3, Abstraction well drawdown takes 26 days to reach steady state. (30 m separation scenario)

Similar results were observed at the designated monitoring point XSL11 P1, where achievement of steady state was slow, suggesting seepage from the river.

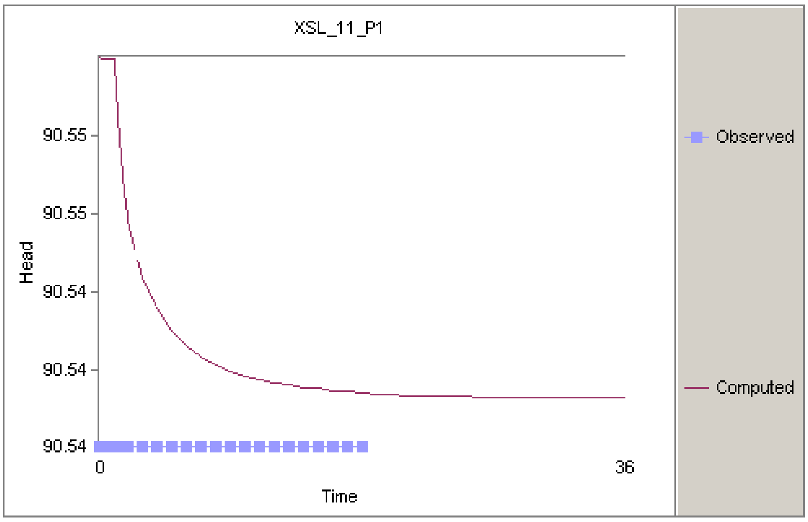

Figure 51: Layer 3, XSL11 P1 response to drawdown takes 26 days to reach steady state. (30 m separation scenario.)

Similar results were obtained for the 50 m separation scenario. 26 days were required for the pumping wells to reach steady state. The hydrographs were similar in shape and not worth presenting here. For a complete set of hydrographs see the 'Modelling File' on the accompanying cd.

5.24. Sensitivity Analyses

An indirect sensitivity analysis vas conducted during the model calibration stage.

Hydraulic parameters were altered individually, until they produced residual head differences of 0.50 m.

Hydraulic conductivity for each zone was analyzed. The analysis was not quantified as the calibrated model statistics were skewed by the larger deviations in computed versus observed head values in areas away from the river.

Some of the zones for hydraulic conductivity were highly sensitive relative to other zones. The analysis found that the model was largely sensitive to changes in conductivity to the layers 3 – 8 which represented the Sandstone portions of the aquifer. Changes to layer 1 which represented the made ground was insensitive. This is largely due to the fact that some portions of the constant head boundary were below the bottom elevation of this layer, further more no recharge was added to the steady state simulations.

Layer 2 represents the sand and gravel layer. Changes of one order of magnitude produced changes of 0.5 from the calibrated head values.

Changes to the River bed conductance resulted in large changes to the proportion of water drawn from the river to the wells. Increasing the bed conductance value by one order of magnitude from the original conductance of 0.3 increased the percentage of river derived water from 18 to 39 percent during the 50 meter separation scenario and 37.5 to 43 during the 30 meter scenario.

Reducing the bed conductance by one order of magnitude from the calibrated value had the opposite effect; only 11 percent of the pumped water was draw from the river.

This has highlighted the importance of obtaining a good estimate for the river bed permeability.

6. Chapter 7

6.1. Introduction

The design of any well should ensure that the particular needs of that well are met.

The wells proposed for the Dipole setup is designed to lower the water table enough to produce flow reversals within the hyporheic zone of the River Tame.

The wells are purely for research purposes, therefore minimising costs particularly operational costs is an important requirement.

Several parameters can affect the initial and operating costs of a well. It is the aim of this report to present a well design that both meets the basic requirements and minimises cost.

The main objectives of this chapter are as follows:

- Choose casing diameter, material type and method of joining
- Casing completion method
- Size and type of surface casing
- Type of grouting materials
- Protection for surface casing
- Estimate well depth, diameter
- Screen length, diameter and material
- Gravel pack, method of sizing, and emplacement method
- Screen length, slot size
- Head losses &Pump selection
- Drilling method & Well development
- Illustrations of each well
- Site Diagrams

6.2. Casing Diameter

For this experimental project the casing must accommodate the pumping equipment, allow sealing of the well and provide support in areas of unconsolidated aquifer material, preventing sediment from entering the well.

Casing diameter is largely controlled by the pump diameter.

A 3” submersible pump is chosen for this design and will be discussed later in the chapter. A 6” diameter casing was chosen as it is offers more flexibility to the project in the unlikely event that the discharges proposed do not fulfil the requirements and a larger diameter pump is required.

The surface casing is designed to provide support from the unconsolidated Glacio-Fluvial Deposits and River Terrace deposits, and will allow a surface seal to be completed.

Depths of expected strata and casing installation depth will be given in table xyz and on the individual well illustrations.

The casing length (L) was calculated using the following formula:

$$L = H + Sw + x + y + z$$

Where:

H is the depth to the lowest expected water level (m)

Sw is the maximum expected drawdown (m)

X is the manufacturer’s minimum submergence of the pump (m)

Y is the length of casing required above ground (m)

Z is an extra allowance for very low water levels or poor well performance (m)

GL = 92.377 mAOD

H = 2.377 m

Sw = 8 m

X = 1.5 m

Y = 0 m

Z = 0 m

L = 11.877 m

The base of the designed casing at XSL11 (Abstraction well) is positioned at 80.50 mAOD, 0.50 m above the proposed screen depth (Figure 56), it is therefore sensible to complete the installation of the casing as far as the top of the well screen at 80 mAOD.

Likewise for the injection well positions the casing should be installed to 80 mAOD.

Driscoll (1986) advises that a casing diameter one preferably two pipe sizes greater than the pump diameter, should be used to maintain the up-hole velocity at 1.5m/s or less.

Up-hole velocities of greater than 1.5m/s can result in excess head losses.

$$V = Q / (\pi * rw^2)$$

Where

V is velocity in m/s

Q is the max discharge rate in m^3/s

rw is the radius of the rising main

For a rising main diameter of 3” (0.0762m), and a maximum discharge rate of 4.5 l/s, the estimated up-hole velocity is 0.987m/s and satisfies the guidelines suggested in Driscoll (1986).

6.3. Casing Material

Stainless steel threaded casing is advised from the ground surface to the top of the well screen as it is widely available and moderately priced. The casing will provide sufficient support from the surrounding unconsolidated strata and limit the impact of contaminant spills at the surface reaching the well.

6.4. Casing completion method

The casing should be installed in conjunction with the well screen. Full details of the installation procedure are described under the well screen section below.

For expected depths of installations see appendix 14.

6.5. Grouting Material

The annular space between the casing and well bore is one of the principal avenues through which water and contaminants may gain access to a well. Grouting prevents the entrance of unwelcome water and contaminants. Therefore, the annular space shall be filled with a neat cement grout, a mixture of bentonite and neat cement or bentonite clay grout.

The grouting material used shall meet the appropriate specification listed below:

- The cement grout shall consist of cement and water with not more than 23 litres of water per typical bag (25kg) of cement.
- Bentonite powder may be used in conjunction with the above cement to form a grout mixture to form a flexible seal within the borehole. The final strength of the grout is dependant upon the mix ratio with cement and water. The bags are typically supplied in 25kg sealed waterproof bags.
- Grout should be installed by means of a grout pump or tremie pipe from the bottom of the annular space upward in one operation until the annular space is filled

6.6. Protection for surface casing

Grouting should be conducted as far as ground level and an apron should be constructed to produce a one foot radius around the casing at least six inches thick.

A small opening should be cut into the ground adjacent to the well apron or immediately beneath the flow meter to house any flow logging equipment. (See Figure 56)

6.7. Well diameter and depth

The well diameter is one of the least sensitive parameters of the well design and is largely controlled by the required pump size, which in turn controls the casing size. An 8" diameter hole is required to allow the installation of the surface casing (6"). The diameter of the well is not expected to significantly alter the drawdown away from the well.

The abstraction rates required to produce the required flow reversals are low, and installing a fully penetrating well would be un-economical. On this basis a partially penetrating wells are preferred. The well length has been determined using Modflow to simulate various well depths. A sensitivity analysis was conducted via a trial and error approach to find the optimal well depth for a narrow pumping range, of between 1 and 4 l/s, constrained by the available power supply.

The depths for the three discharge wells range between 16 and 17.5 m below ground level. More accurate depth estimates for the three proposed discharge wells are shown in the individual well table (Appendix 14).

6.8. Screen Length, diameter and material

The screen plays a critical role in the performance of the well since it provides the filtering of the water entering the well. In this section, the type of screen, aperture size, diameter, length, entrance velocity, and material of the screen are described, along with the installation method. The well screen should protect the pump from loose material that may become free from the sandstone aquifer, particularly if the formation is poorly cemented. For this experiment high flow rates are not expected therefore the risk of borehole wall collapse is reduced.

An estimation of aperture (slot) size is made based on the results of a sieve analysis preformed on weathered sandstone samples hand excavated and cored from the riverbed deposits. This is only a rough guide; the calculation should be repeated following retrieval of samples from the production interval of the well.

Numerous screen lengths were modelled to estimate the optimal screen length.

Deeper wells screened over the lowermost section tended to reduce draw down whereas shallower wells resulted in excess drawdown.

Long screen lengths were tested to reduce flow convergence and increase the efficiency of the well. However shorter screen lengths were favoured to produce the required drawdown to generate the flow reversals. A screen length was chosen based on the modelling results. The small pump rates (1-4 l/s) produced the required drawdown for screen lengths of 4 m.

Only minor variations in pumping rate were required to produce the same effects for the varying well separations.

Prior to installation of the well screen and casing the borehole should be checked to ensure it has been drilled to the specified depth. The well screen should be welded to the casing and lowered into the borehole. A plug should be fitted to the base of the well screen to seal the well. Each length of casing can be connected together as they are lowered into the well. The lengths should be measured and added to ensure the screen is placed at the designed depth.

Following the installation of screen and casing, a surface seal should be constructed to minimise infiltration of water or any other liquids down along the spacing between the borehole wall and the surface casing. Immediately after installation of the well screen and riser, the total inside depth should be checked to determine the exact inside depth of the well and to see if any damage has occurred during installation.

6.9. Rising Main Diameter and Material

The Rising main material has been selected to minimise construction cost and because of its availability and ease to work with.

PVC plastic material is advised for the dipole setup as it provides sufficient flexibility during the construction process, at a location where varying ground and river bed elevations will require customised pipe lengths to be cut on site. PVC requires less effort and expertise to cut required pipe lengths and has excellent chemical resistance to weak alkalies, alcohols, aliphatic hydrocarbons and oils.

6.10. Well separation & distance from the river

The well separation was constrained by the limitations posed by the early specific design criteria. A 30m section of river (minimum) is the desired impact length between the two wells. This distance proved a sensible distance over which to conduct the experimental work, allowing tracer tests to be conducted within the time limits imposed by the SWITCH project. An alternative separation to the first Dipole setup was set at 50m to allow the flexibility to work over a larger impact zone, in the event that the first setup proved unsuccessful. This separation allowed the tests to be run within the imposed time and discharge limitations. The proposed separations are illustrated later in Figure 59 to Figure 61 and have been referenced to noticeable structures on site.

6.11. Drilling Technique

Many factors control the drilling technique used for this particular project

- Drilling should cause as little disturbance of the subsurface as possible to minimise disruption of the adjacent riverbed sediments.
- The use of air or water flush should be limited particularly in the upper unconsolidated material, as even the lowest air pressures used in rotary drilling can cause disruption to adjacent river sediments and risk jeopardising the overall project.
- The drilling rig used must be capable of comfortably setting up within 1 to 2 meters of the river bank edge.
- The technique employed must be capable of penetrating sand and gravel layers containing occasional cobbles effectively.
- The rig must be able to setup on small slopes.

Boreholes drilled within a 500-m radius of the proposed site have used either cable percussion or rotary drilling techniques (Environment Agency, 2000; Severn Trent Water, 1993; Taylor et al., 2003); however the ideal drilling solution for this particular experiment would require minimal impact to the bed sediments along the section of river of interest. Mini rigs typically with extendable masts are available within the UK, for use in areas of restricted access. The proposed site will require a tracked mini rig to set up within 1-2 meters of the North River bank along the Tame.

14 No rigs were considered for this project, however the final choice is readily available within the Midlands area would provide an ideal drilling solution. The proposed rig is the Beretta T25 tracked lightweight rig. This machine weighs approximately 1700kgs and is easy to transport to site.The rig is self propelled, tracked and is capable of conducting core boring, auger drilling, drag bit and down the hole hammer drilling (DTH). The Rotary drill head has

a modular structure and by superimposing 1-2 or 3 hydraulic motors, it is possible to select various rpm and torque.

The Beretta also incorporates head side movement and a single hydraulic clamping vise which allows alternation between drilling techniques.

The rig also has the following additional features:

- Capable of conducting SPT's
- Hydraulic winch
- A double hydraulic vise allows clamping of casing or drill bits
- Lubricator for DTH drilling is available
- Contains a water pump
- The mast length can be adjusted according to head room.

The rig is primarily chosen for its ability to drill into both consolidated and unconsolidated sediments. Casing should follow the drill bit as closely as possible and air pressure should be minimised to limit the amount of air pressure being focused on the borehole wall. Once the unconsolidated strata have been cased off, air pressure may be increased to remove cuttings from the borehole while drilling into the more consolidated Sandstone aquifer.

Provided the casing is kept as close to the base of the hole at all times this technique should have minimal little impact on the integrity of the riverbed sediments during the drilling process. The 14 No Mini drilling rigs highlighted during this review have been obtained from numerous geotechnical and drilling websites. The complete list of rigs and a brief summary of their capabilities are presented in Appendix 12.

The Beretta requires a separate diesel/hydraulic power generators to conduct the drilling. The generators can be left at great distances from the drilling rigs provided adequate hosing is requested to span the required distances. Hosing is usually 2" in diameter and come in lengths of 10m. The hosing is typically supplied by the generator provider.

6.12. Hydrogeological requirements for the well design

Borehole logs provided information about the underlying site stratigraphy, the expected depths to and thickness of each formation.

The closest reference borehole to the proposed site at XSL 11 is BH AB (grid: 993260.00, Easting: 474971.00) which is approximately 150 m south of the site.

The proposed site is expected to contain between 1 to 2 meters of made ground, consisting of sandy, gravely clay, underlain by unconsolidated sands and gravels (expected thickness 2 to 2.5 meters), resting on-top of weathered Triassic sandstone (Wildmoor formation thickness: 1 to 1.5 m).

The principal aquifer unit The Wildmoor Formation has an expected thickness of 76m.

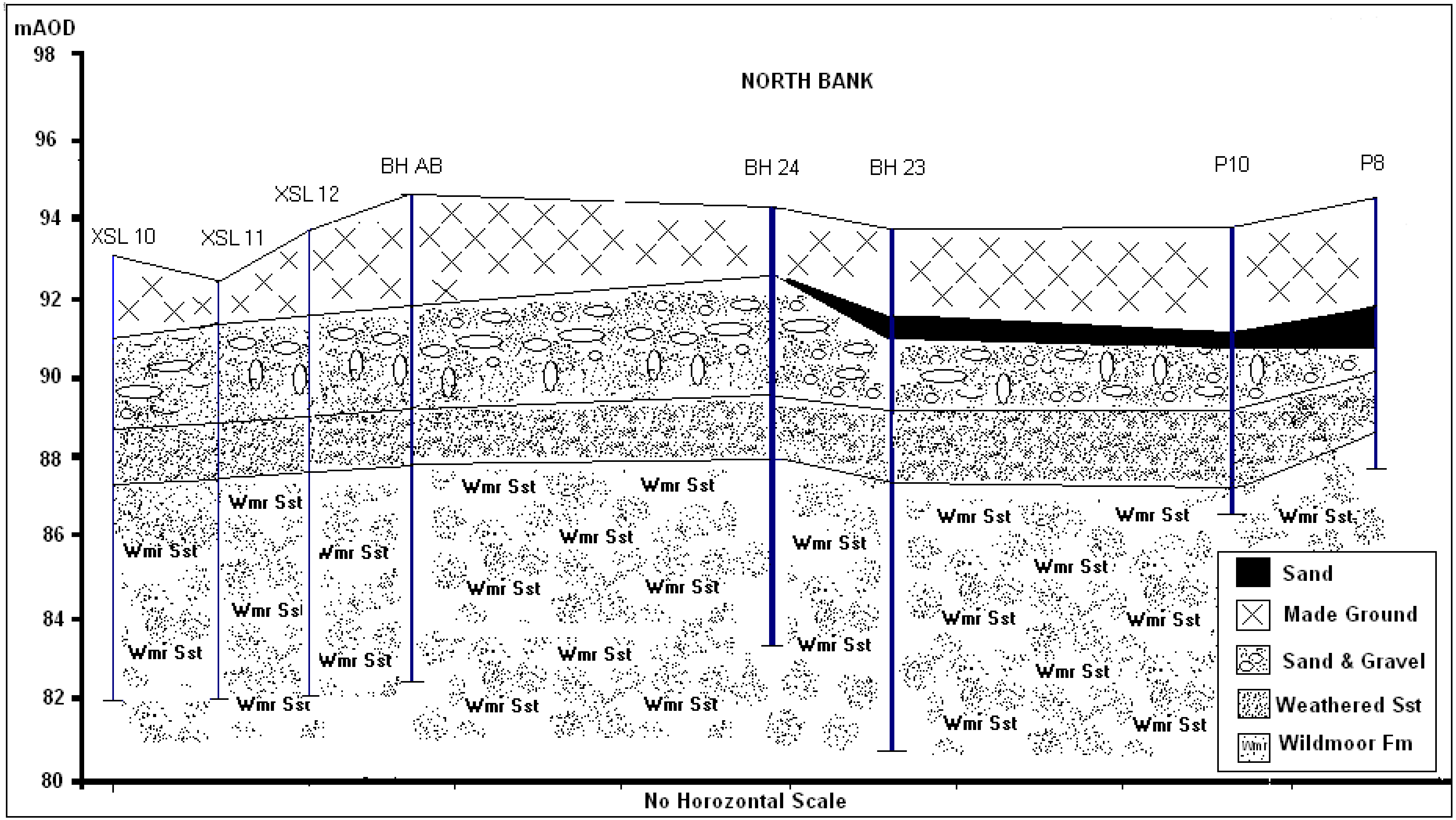

Figure 48 : Expected geology at XSL 11 and XSL 12 from stratigraphic correlation of Severn Trent Water and Environment Agency borehole logs

6.13. Gravel Pack

An initial estimate has been conducted to assess the likely requirement of an artificial gravel pack, using guidelines suggested by Driscoll (1986):

Uniformity Coefficient < 3 and D_{10} < 0.25mm

Likewise for a natural gravel pack the uniformity coefficient should be > 3 and have a D_{10} > 0.25 mm. The Uniformity coefficient is defined as: D_{10} / $D_{60,}$ where D refers to the percentage passing the sieve. A large uniformity coefficient represents ill-sorted sediment.

Riverbed samples 2, 3 and 5, were chosen from the detailed map area (XSL 11). All of the samples consisted of fine to medium red-brown weathered Sandstone. These samples are thought to closely resemble the grain-size of the underlying sandstone.

Uniformity coefficients for the 3 No. samples ranged from 1.90 to 1.93 and the D_{10} fraction ranged from 0.21 to 0.29 mm with an average D_{10} of 0.24 mm.

Based on these findings it is predicted that the formation is likely to require an artificial gravel pack. However this procedure should be repeated following completion of the three boreholes, to be conclusive. The predicted gravel pack will serve as a filter, prevent movement and increase the effective hydraulic diameter of the well.

A common consensus seems to be that a gravel pack performs well if its uniformity coefficient is similar to that of the surrounding aquifer material.

6.14. The Gravel-pack specification:

The process for designing the artificial gravel pack has been given by Driscoll (1986).

The D_{70} of the finest sediment must be multiplied by a factor of 5. This will give you the D_{70} of the new artificial gravel pack. A new distribution curve with a uniformity coefficient < or = 2.5 was constructed as shown in Figure 53.

The D_{10} of the new distribution curve has yielded a value of 0.8mm.

This is now the recommended slot size for the well screen.

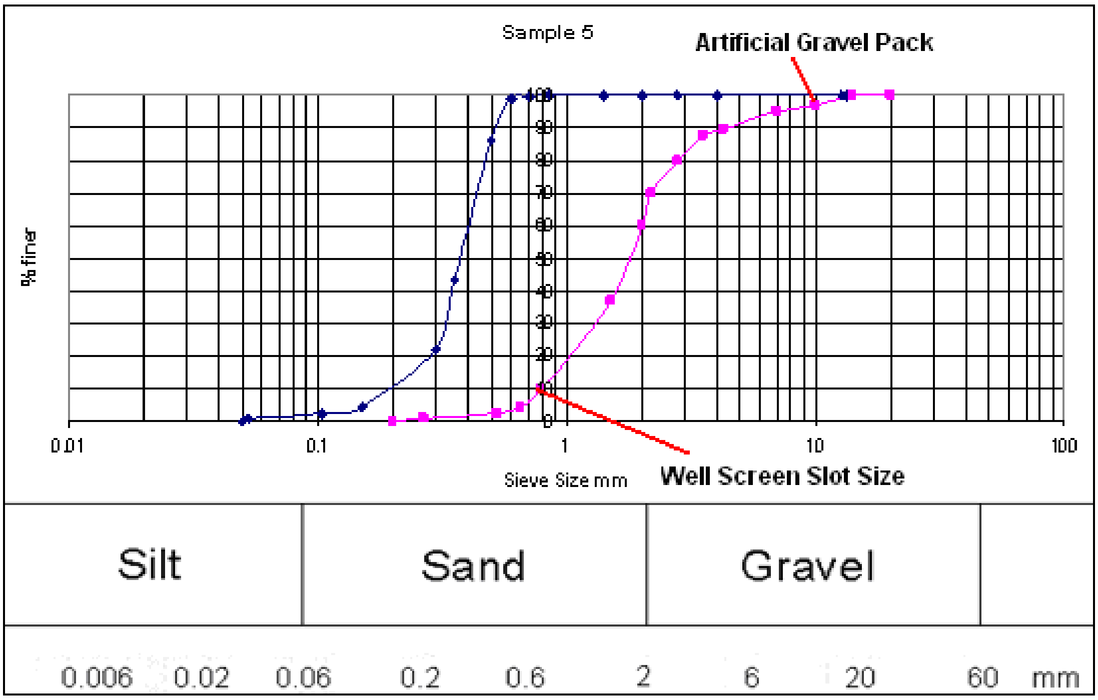

Figure 53: Artificial gravel pack design following riverbed hand and core sample analysis.

A minimum thickness of 75mm but no greater than 120mm is required around well screen.

The artificial pack should be poured from the top of the borehole using a tremie pipe as shown in Figure 54 .

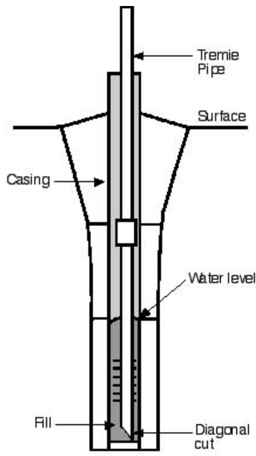

Figure 54: Artificial gravel pack being poured using a tremie pipe

The gravel material should be clean and well rounded with a maximum of 10% flat surfaces and should be a minimum of 95% siliceous in content (this avoids dissolution in low pH water), (Driscoll, 1986).

Immediately prior to installation, the gravel pack should be treated with chlorine to disinfect the aggregate and reduce the likely growth of bacteria.

6.15. Injection well issues

Injection wells differ from production wells in several ways. Two of the more important differences are screen design and seal placement. Most references recommend a water velocity through the screen of one-half of that used in the production well. It appears that this guideline is primarily related to the allowance for plugging of the injection screen with fine particles carried into the well with the water.

These guidelines are typically met by increasing the borehole diameter or more commonly the length over which the borehole is screened to reduce flow velocities entering the aquifer. Additional head build up may occur due to clogging of the injection well, with as little as 0.004 mg/l of sand. (Driscoll, 1986)

If the production well is sand free, or if a surface strainer is used to minimize sand, the additional screen may not be necessary.

6.16. Sealing

Sealing of an injection well should be done in much the same way as a production well penetrating an artesian aquifer. (Rafferty 2001)

The reason for this is that in the course of the operation of the well, the pressure exerted on it is greater than the natural pressure of the aquifer it penetrates. As a result, there is a tendency for water to migrate up around the casing toward the surface.

If the well is exposed to a positive pressure at the ground surface, the potential exists for water to leak out around the casing at the surface. To prevent this, injection wells should be sealed from just above the injection zone, continuously to the surface, with a minimum 2 in. (50 mm) annular (between the casing and the well bore wall) cement seal.

The injection stream should be introduced into the well using an injection tube terminating below the water surface. This prevents the injected water from surging down from the well head and generating air bubbles in the process. Bubbles driven out into the aquifer can act as an obstruction to water flow in much the same fashion as particulate matter.

The abstraction well should contain gravel pack of 5 meters as specified; a 1 m sand filter followed by 10 m of cement grout. The bore hole should be completed to the surface with 1.5 m cement seal.

6.17. Well development

After the structure of the well is installed the well must be developed in two stages. The initial development should be conducted by the driller on site using air-lift pumping to remove as much sand and fine material as possible. Termination of the well development will depend on the nature of the formation and the on site engineer should decide when enough development has been reached.

A second phase of development using jetting should be conducted to remove sand and fine material to the desired specification. It is recommended that a Rossum Sand Tester (capable of measuring sand content to 0.5ppm).

There are very few guidelines for the final sand content that is deemed acceptable for re-injection. The Californian Groundwater Association (CGA, 1994) recommends that

Limits of 15 ppm should be achieved in wells designed for irrigation proposes.

A limit of 10 ppm for wells supplying water to sprinkler irrigation systems or industrial cooling systems should be sought.

As little as 5 ppm for wells supplying water to homes, institutions, municipalities, and industries, whereas no more than 1 ppm for wells supplying water to be used in contact with or in the processing of food and beverages.

6.18. Disinfection

Disinfection is the process of cleaning and decontaminating the well of bacteria that may be present due to the drilling action.

Chlorine disinfection is the most common method where by a small volume of chlorine solution is added into the top of the well, followed by circulating the chlorine into the water, through pumping.

Holben & Gaber 2003 recommend 1 cup of chlorine for every 4 meters of a 6 inch well. The chlorine should be mixed with between 19 and 20 litres of water before being added to the well. For more detailed description consult Holben & Gaber (2003).

6.19. Entrance Velocity

The entrance velocity is calculated by dividing the well yield in meters per second by the total area of the screen openings in square meters. This will ensure:

The hydraulic losses in the screen opening will be negligible.

The rate of incrustation will be minimal,

The rate of corrosion will be minimal.

Driscoll (1986) recommends that the entrance velocity should not exceed 0.03m/sec. A excel spread sheet was constructed to assess the likely entrance velocities for the maximum pump discharge of 4.5 l/s at varying screen lengths, using

$$Q = VA$$

Where

Q is the wells anticipated yield (m^3/s)

V is the entrance velocity (m)

A is the screen open area (m^2)

The minimum screen length that will produce entrance velocity's that match recommendations of 0.03m/sec (Driscoll, 1986) is 0.55 feet or 0.17m. Larger screen lengths increased the overall open surface area available therefore decreasing the entrance velocity. (Note the analysis was conducted using a wrapping wire width around the screen of 0.156 inches and a discharge of 4.5 l/s.)

For the final design, the entrance velocity has been calculated to be 0.002 m/sec. This value is still considerably lower than the recommendation for injection wells of 0.015 m/s by Driscoll (1986) to allow for some additional head build up due to clogging.

6.20. Approach Velocity

It is important that within this design, the migration of fine particles from the aquifer to the well is avoided. It has been argued that particle movement is largely controlled by the approach velocity (Va).

Va is estimated using the following equation:

$$Va = Q/ (Ls*pi*D)$$

Huisman (1972) recommends that

$$Va < Sqrt\ K/ 30$$

Where:

Q is the estimated yield (m/s)

Ls is the well screen length (m)

D is the drilled diameter (m)

K is the average hydraulic conductivity of the screened formation (m^2/d)

For the maximum required discharge capacity of the well approach velocities were estimated using an excel spread sheet. Va was calculated to be 0.0018m/s and Huisman's recommendation was met using a range of hydraulic conductivity values.

Approach velocity was calculated to ensure turbulent flow and air entrapment in the gravel pack is avoided. It is widely accepted that to avoid turbulence in the gravel pack that the Reynolds number (Re) must be less than 4.

$$Re = (V_s * D_{50})/V$$

Where:

Va is the approach velocity (m/s)

V is the kinematic viscosity of water ($1.307x10^{-6}$) [-]

D_{50} refers to the median gravel pack grain size (m)

Ds refers to the screen diameter (m)

Vs is estimated using the following equation

$$Vs = Q/ (Ls*pi*Ds)$$

The Re calculated for the maximum pump discharge and median gravel pack grain size yielded a value of 2.43 using the equation above, and therefore satisfies this requirement. To run an injection well efficiently it is beneficial to have water free of sand. In the case of the Switch setup, design of clean wells both for pumping and injecting is fundamental to the operation and maintenance of the wells. Sand free water will maximise the recharge at minimum pressure build up.

6.21. Upflow losses

Up flow losses are head losses due to friction in the well.

The following equation is used to estimate the head loss:

$$h = 3.428\ Q^2 n^2 LsD^{-16/3}$$

Where

Q flow rate (m3s-1)

n is Mannings roughness coefficient (0.015 for wire wound screen)

Ls is the length of the well screen (m)

D is the well screen diameter (m)

For a six inch diameter, 4m long screen the pumping at maximum capacity of 4.5 l/s the upflow losses amount to a head of 0.0014 m.

Pump Selection

The design capacity of the pump must exceed the overall project requirements. However, this section is designed to ensure that the capacity of the pump does not exceed the capacity of the well. Pump manufacturers publish charts giving the pump discharge capacity for their particular pumps at various operating pressures.

Calculation of the desired flow rate and TDH permits selection of the pump based on performance curves. The pump curve typically shows a pump that will operate over a wide range of conditions. However, it will operate at peak efficiency only in a narrow range of flow rate and TDH.

Grundfos Ltd are selected as a potential pump supplier for this study. The following design will present a pumping solution that will meet the overall requirements of the Dipole project.

The total dynamic head (TDH) of the system must be calculated accurately from the physical pumping arrangement and is represented by the following equation:

$$TDH \quad = \quad H_S + H_D + H_F + \frac{V^2}{2g}$$

Where:

H_S = the suction lift; the vertical distance from the waterline at drawdown under full capacity, to the pump centreline.

H_D = the discharge head; the vertical distance from the pump centreline to the maximum elevation level in the system.

H_F = Friction head; loss of head across pipelines caused by during entrance and exit from the system as well as losses due to fittings and the smoothness of the pipe work. H_F is strongly flow dependent. These losses should be kept as low as possible.

$V^2/2g$ = velocity head; head necessary to accelerate the liquid and maintain flow in the system.

6.22. Suction Lift HS

A submersible pump is desirable for this project for a few reasons:

It helps minimise visual impact at the ground surface

They can deliver a reliable constant discharge

They are relatively inexpensive and widely available

They are easily installed and can be retracted when not in use

Surface noise is minimised

Suction lift is therefore not required as the pump is fully submersed.

6.23. Friction Head Loss (H_F)

The friction head of a pumping system is estimated by adding together two components.

The friction losses in the straight pipe work.

The friction losses caused by fittings i.e. bends, valves, tees, etc.

$$H_F = H_{FF} + H_{FS}$$

6.24. Friction Losses in the Fittings

Losses in fittings are caused by interaction of the fluid and the internal surface of fitting and the turbulence generated within.

The friction loss for fittings depends on a K factor which can be from many pump suppliers and fitting manufacturers. Pump specifications supplied by Grundfos pump manufacturers was used to select an appropriate pump for this project (Appendix 12).

Fitting loss tables for various materials and sizes were also provided which permitted the estimation of losses likely to be endured due to bends in the pipe work the Coefficients for K were cross-referenced with values obtained from and engineeringtoobox.com and are available for reference in Appendix 12.

The following equation is widely used:

$$H_{FF} = K \text{ x } v^2/2g$$

Where:

H_{FF} - Friction losses (m)

v - flow velocity (m/s)

g - Gravitational Constant (= 9.81 m/s^2)

K - specific loss coefficient for the fitting

6 No 90°, 3inch pipe fittings are required to complete this dipole design.

Literature values were used for the specific loss coefficient. A K value of 0.3 was used to represent each of the (6) regular 90°, flanged fittings.

Using the following equation to calculate the area of a circular pipe:

$$A = (Pi/ 4)/D^\wedge 2$$

The velocity of flow through the pipe was estimated from:

$$V = Q/A$$

Where

Q is the discharge (m^3s^{-1})

A is the pipe area (m^2)

D is the rising pipe main diameter (m)

V is the velocity (ms^{-1})

The total head loss due to friction caused by the pipe fittings was 6(0.015) = 0.09m.

An alternative method was used to calculate head loss. The Equivalent Pipe Length Method is based on laboratory head loss results and equivalent lengths are easily read from tables. (Appendix 12)

The following is the calculation of the head losses for 6 No 90°, 3inch PVC pipe fittings, based on the international voluntary standards organization schedule 80 (ASTM), where the equivalent pipe length is 7.9 inches.

(6 No.) 90 deg 3" Elbow = (6x7.9”) = 47.4” ~ 1.204m

The resultant head loss is significantly larger using this method. The latter estimation of H_{FF} is used to calculate the Total Dynamic Head required by the pump, as it is considered more sensible to over estimate the required head than under estimate.

6.25. Friction Losses in straight pipe lengths (H_{FS})

Losses in straight pipes are caused by interaction of the fluid and the internal surface of the pipe. The magnitude of the losses is dependant on the following factors:

Internal Pipe Diameter

Flow Velocity

Length of Pipe

Pipe material / Internal roughness

Nature of the flow

Properties of the fluid

The inner roughness of the pipe can create eddy currents. This increases the friction between the pipe wall and the fluid. The relative roughness of the inside of the pipe is used in determining the friction factors that are widely available from equipment suppliers.

Coefficients for Friction head losses can be obtained for various pipe diameter and materials (See Appendix 12)

Friction Head Loss (H_{FS}) is then given by:

$$HFS = f\frac{L}{D}\frac{V^2}{2g}$$

Where:

f - Darcy-Weisbach friction factor for straight pipes [-]

L - length of pipe (m)

d - pipe diameter (m)

V - flow velocity (m/s)

g- Gravitational Constant (= 9.81 m/s2)

The Darcy-Weisbach *f* is a complex function of the Reynolds Number and relative roughness.

The Reynolds number, Re is defined as,

$$Re = \frac{\rho V D}{\mu}$$

Where μ is the fluid absolute viscosity, ρ is the fluid density, V is the fluid velocity and D is the pipe diameter. Re is equal to 5.83×10^4.

The following equation can be used for hydraulically smooth pipes such as glass, copper and plastic tubing in turbulent flow situations

Blasius's equation for *f* is as follows:

$$f = \frac{0.3164}{Re^{0.25}}$$

Applicable for (4,000 < Re < 100,000).

Using the calculated Re above a friction factor *f* = 0.02036 was obtained.

Subsequently a head loss H_{FS} of 0.925m was estimated.

An equivalent length approach was also used to estimate H_{FS} using literature values, (see Appendix 12). A value of 1.5 feet of friction head is expected to accumulate over a 100 feet stretch of pipe work.

Calculation

Total Equivalent Length of 3" Pipe	=	69.484 + 1.204 = 70.688 m
Friction	=	0.015m for every 1m therefore
H_{FS}	=	1.06m over the entire length

6.26. Discharge head

The discharge head is a large factor in determining the total dynamic head required for the system.

The Total Discharge Head is a function of the Static lift plus Friction losses.

Discharge Static Head is given as follows: 14.915 m + 0.25 m = 15.165 m.

Total Discharge Head (TD) is the sum of the losses

15.165 m [Static] + (1.06m + 1.204m) [Friction] = 17.429 m

Total Dynamic Head TDH = [TD + TS + Sw*] = 17.429m + 0m + 0.06 m = 17.489 m.

*Sw corresponds to the modelled drawdown.

6.27. Power Consumption

The submersible pump power requirement is a function of flow, head, and efficiency. Properly selected the design hopes to maximise efficiency.

The power input to this system is constrained to a 230V power source. The pump selection therefore requires that this criteria is met.

A submersible pump from the Grundfos SP – 17 NE range has been selected based on its ability to meet the required well capacity and the head it can deliver.

The following is a summary of the requirements used to select the pump(See Table 17).

Requirements	Values
Max capacity requirement	4.5 litres per second
TDH required	17.489 meters
Power input 230V	230 V
Require good efficiency	65-80%

Table 17: Summary of the pump requirements

The Power input requirement can be calculated using the following equation:

$$P = g\ Q\ (TDH) / \eta_o$$

Where:

P is power in kW

g the earths gravitational constant (9.81m/s)

Q is the maximum desired capacity of the pump (m3/s)

TDH represents the lift required from the pump (m)

η_o is the overall efficiency of the pumping system

From the manufactures performance curves the required capacity and lift is achieved with an efficiency of 73%, at the cost of 1.057 kWh.

The estimated power consumption for a 180 day pumping test running 100% of the time is 4568.9 kWh.

6.28. Installation of the Submersible Pump

The submersible pump should be laid horizontally on the ground surface adjacent to the borehole so that the 3” PVC riser pipe work can be assembled.

An appropriate drop pipe adaptor should be attached to the pump discharge end followed by a 3" torque arrestor, to prevent the starting torque of the motor causing damage to the submersible pump. (Damage caused by rubbing against the inside walls of the well casing).
A steel safety rope should be attached to the connecter provided on the discharge side of the pump to avoid strain on the riser pipe or electrical cable during installation.
The pump's power cable can be placed inside an appropriate small diameter tube to avoid it from rubbing against the rising main; this tube can then be taped every 3 m to the riser.

A pipe joint compound could be used to attach the PVC pipe lengths (typically 3 m in length).
A foot clam or an A-frame could be set up to assist lowering the pump into the well.
The pump should be lowered to the required depth (78 mAOD) ensuring the manufacturer's minimum submergence criterion is met. Unfortunately the minimum submergence was not available at the time of this study but from consulting similar pumps the minimum submergence is typically 1 to 2 meters. The static and dynamic water levels for this design more than meet these requirements.
The top of the riser should be attached with a 3" (90^0) elbow allowing the discharge pipe centreline to run 0.25 m from the ground surface. The following accessories should be connected to the discharge pipe in series to allow adequate well monitoring.
A non- return valve; a combined sampling tap and pressure gauge; a flow meter; a 3" gate valve. For a visual representation see Figure 55.
The abstraction well and injection wells should be completed with an appropriate well seal that allows the rising main to exit the borehole yet retain a watertight seal. This will be imperative for the injection wells. Although water levels are not expected to exceed the surface elevation, it is important to maintain a good surface seal in the case that additional head build up occurs due to clogging caused by sand.

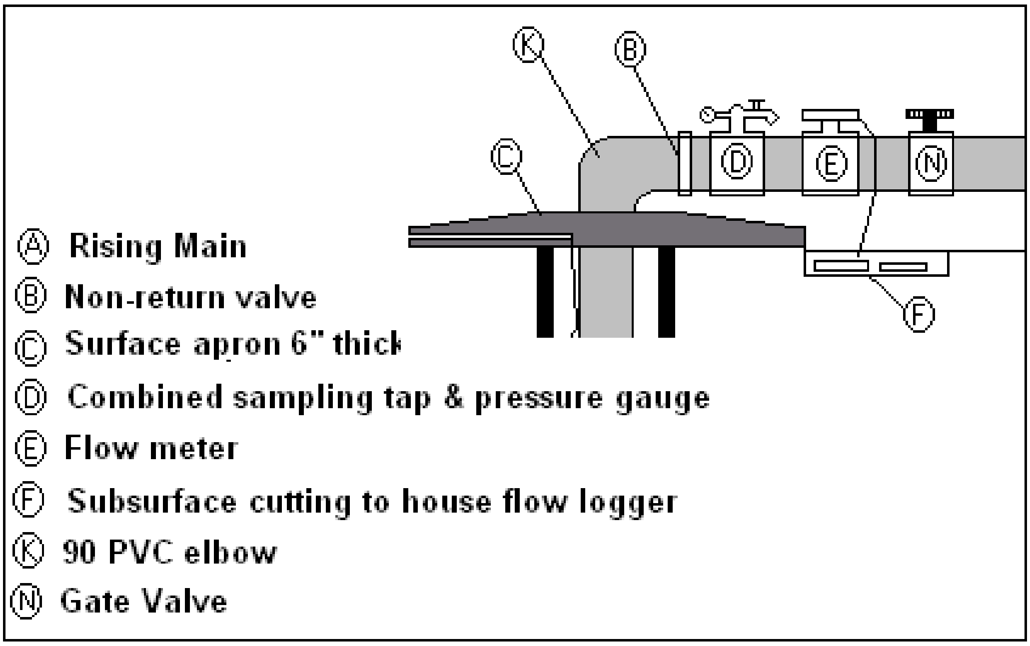

Figure 55: Pump monitoring equipment

6.29. Conclusion

The Dipole well setup was designed based on measurements taken in the field and the modelling results. Two Dipole arrangements have been produced from this study. The first scenario at a 30 m separation produces steep hydraulic gradients across the river bed in order to generate the flow reversals required to run tracer tests within the time limits imposed. The second scenario produced is a 50 m separation and the required drawdown is generated for smaller hydraulic gradients than the first. The following plan, elevation and end view drawings have been produced for the readers use.

Figure 56: Abstraction Borehole design specification adjacent to XSL11

Figure 57: Injection Well (A) design specification at 30m seperation to Abstraction Well

Figure 58: Injection Well (B) design specification at 50m seperation to Abstraction Well

Figure 59: Front Elevation of Dipole Experiment

Figure 60: Front Elevation of Dipole Experiment

Figure 61: Plan View of proposed Dipole site

Figure 62: Diagrammatic aid at locating TBM

7. Chapter 8

7.1. Conclusion

This work concludes with the presentation of a complete design for the Dipole well setup. Two scenarios have been developed, each containing a pair of partially penetrating wells. The wells are designed to alter the hydraulic conditions at a proposed location beneath the River Tame.

The design was optimised using a numerical model (Modflow). Although the modelling was successful in generating the flow reversals within the Hyporheic zone, the River Tame was found to act like a recharge boundary to the abstraction well, particularly when positioned a few meters from the north river bank. The amount of water drawn from the river is a function of the riverbed permeability. In reality the modelled drawdown close to a river will largely depend on this parameter. Values for the riverbed conductance have been entered into the model based on falling head tests conducted at discrete points along the river. It is important therefore that further work be conducted to evaluate exact aquifer and bed permeability parameters. (See recommendations for future work is presented below)

The study area has limited access points to the River Tame; the drilling location specified in this report has been based on the suitability of the location for a drilling rig to gain access and its proximity to a reliable power source. The best location for the wells is directly adjacent to profile XSL11 P1 recognisable in the field by wooden markers and the piezometer installations within the river.

A combination of borehole log and field data analysis of riverbed and bank elevations has permitted detailed cross sections to be constructed, showing the expected strata and depths at each of the proposed drilling sites. Based on this information an appropriate drilling technique and well design has been developed.

The proposed design meets all the requirements of the Dipole experiment and this work has been successful in highlighting the potential problems that may be encountered with such an experiment. It is hoped that most of the problems ranging from pumping sand to the drilling technique, have been address during the culmination this work.

7.2. Future Work

Following permission from the land owners the wells should be drilled using the specification presented in this report. Prior to the installation of a pumping system it is advised that a series of pump tests be conducted at each of the three boreholes.

The hydraulic tests should aim to tackle the following issues:

- Assess the true capacity of the wells using a step-drawdown test.
- Characterise the local site aquifer parameters.
- Try and gain a better estimate of the riverbed conductance.

The step-drawdown tests could be used to determine the likely drawdown for various pumping rates and times. The piezometers installed in the riverbed could be monitored during the test to see how soon after the onset of pumping that changes in the hyporheic zone are observed, and possibly estimate the discharge rates required produce the downward velocities calculated in chapter 7)

The amount of drawdown generated by the abstraction well will depend on the amount of water the aquifer will yield and on the amount of water that will be drawn from the river.

Without accurately being able to assess how much water is being drawn from the stream it is difficult to determine the effects of pumping and makes predicting drawdown for a partially penetrating well difficult.

A stream aquifer analysis test could be conducted to gain both estimates of the riverbed permeability and aquifer parameters from the drawdown. Christensen, (2000) has used streambed piezometers and analytical solutions to inversely estimate the bed permeability. (See Appendix 13) Although a large number of falling head tests were conducted during this project to estimate the bed permeability, a pumping test would provide a larger scale estimate, rather than the discrete values obtained during this work. The falling tests results could be compared to such a pumping test.

Finally it is recommend that following the completion of the boreholes that considerable effort is made during the development stage. This will ensure the long term success of the Dipole project.