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The Vowchurch Gravel Aquifer Pipeline Crossing - Hydrogeological Evaluation of Impact

Groundwater Management Programme

Commissioned Report CR/06/186N

Report prepared for STATS Limited on behalf of Murphy Pipelines Limited within Murphy Report Series No. 34885/014 (Vowchurch Aquifer study)

BRITISH GEOLOGICAL SURVEY

GROUNDWATER MANAGEMENT PROGRAMME

COMMISSIONED REPORT CR/06/186N

The Vowchurch Gravel Aquifer Pipeline Crossing - Hydrogeological Evaluation of Impact

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The Brecon to Tirley gas pipeline includes a crossing of the River Dore, its flood plain and associated shallow gravel aquifer at Vowchurch in Herefordshire. The aquifer is used as a public supply resource and concern has been raised regarding the likely impact of the pipeline on the aquifer and the integrity of the public supply boreholes. A detailed assessment has been undertaken to identify the likely impact on the water table and hence the integrity of the abstraction borehole performance, which:

1. investigates the hydrogeological impact of the pipeline during construction, and
2. investigates the post-construction long-term effect of trenching and pipe laying.

A detailed evaluation of the geological framework, in which the gravel aquifer is a part, showed that the aquifer could receive direct rainfall recharge through the overlying granular till and fine-grained overbank deposits. A conceptual groundwater flow model was created which identified that secondary recharge was taking place from the River Dore in the upper part of the aquifer with loss back to the river at the lower end of the aquifer.

All available data on the aquifer, surface water, meteorological data and land use information were gathered together and used to assess and quantify the recharge processes in the catchment. Numerical modelling of the original pumping test data at Vowchurch yielded formation constants for the aquifer. These data were then incorporated into a numerical groundwater flow model using the object oriented ZOOM suite of modelling software to replicate the available historical data for the aquifer. The best fit model has been used to run a variety of ‘what if’ scenarios.

The effect of drawing the water table down with a well point system during construction, to maintain a dry 2m deep trench, will cause a likely drop in the groundwater head at the Vowchurch pumping station of up to 0.5 m. This head reduction will be apparent from about one day after the well point system is switched on until the pipe is laid and the trench backfilled some two to three days later. The trench will only need to be dewatered in the 200 m section nearest to and west of the River Dore. Should any turbidity be created in the aquifer by trenching it is likely to be lost as the groundwater travel time from the trench area to the pumping station is at least 100 days.

The impact of the completed pipeline on groundwater flow and groundwater heads is minimal. The damming effect of the pipeline on the water table is very small, as is the effect of varying the permeability of the backfill material. Although the relationship with the River Dore will be very slightly modified, that with the Slough Brook remains unchanged as it is perched in the vicinity of the pipeline.

The river crossing is through overbank silts and will neither penetrate the till nor the gravel aquifer. It is not expected to impact the aquifer. Trenchless crossings of either the River Dore or the Slough Brook would involve a risk of penetrating the aquifer and, therefore, pose a risk of contaminating it. The trench option for both crossings thus posses the least risk to the aquifer.

1 Introduction

The Brecon to Tirley gas pipeline is being built by Murphy Pipelines Limited for National Grid. It will cross the River Dore in the Golden Valley at Vowchurch, Herefordshire, through alluvial valley fill deposits which include a gravel aquifer utilised for public supply. The public supply boreholes are situated some 2 km down stream from the crossing. The possible impact, both during the construction and post-construction phases, that the pipeline may have on the public supply abstraction regime was raised at an initial meeting between the borehole operators, Welsh Water, and the environmental consultants to Murphy Pipelines Limited, STATS Limited. The outcome was that STATS Limited commissioned the British Geological Survey to investigate the aquifer, specifically:

1. To investigate the probable impact of construction, through trenching and associated dewatering, both of the pipeline crossing the alluvial plain to the side of the river Dore, and beneath the River Dore to the bedrock flank adjacent to the river on its eastern side.

2. To investigate the post-construction, long-term effect of trenching and pipe laying, specifically the possible dam effect of the pipeline to groundwater flow and the possible increase in permeability of the trench backfill with a hydraulic connection to the river, increasing the local recharge potential.

The work comprised three phases:

1. *Data Gathering*: Identify and gather all available data describing the aquifer.

2. *Conceptual Model*: Develop a 3-D model of the geological framework of the valley fill superficial deposits. Develop a conceptual flow and water balance model for the aquifer to identify all the likely input and output components of flow from this part of the catchment.

3. *Groundwater Modelling*: Construct a groundwater model of the alluvial aquifer in the vicinity of and between the pipeline and the abstraction boreholes. Validate the model against available historical data and identify data sets to which the model is most and least sensitive. Run ‘what if’ scenarios.

2 The Golden Valley at Vowchurch

North of Turnastone the flood plain of the River Dore is about 600 m wide. Below Vowchurch the flood plain tapers to only 300 m between the valley side immediately to the north-east of the river, and a mass transport deposit on the south-west flank of the valley (Figure 1). There is a major east-west fault in the bedrock along which a valley has been eroded west of Vowchurch. Bedrock is likely to be deeply weathered along the line of this fault.

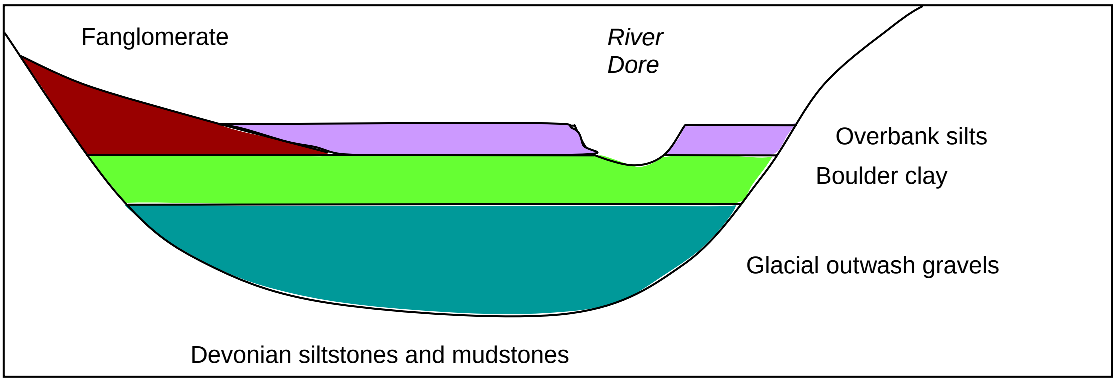


Figure 1. Schematic section across the Dore valley through Chanstone Court Farm (for locations see Figure 3)

The main geological units are:

Overbank silts: a red silty-sandy clay soil formed as over-bank flood deposits on the flood plain of the River Dore. It is free draining and there is no evidence of marshy or boggy ground. It is moderately permeable. The overbank flood silts are of recent origin and mainly occur along and beneath the present course of the Dore River.

Fanglomerate: comprises sub-angular sandstone cobbles in a red, gritty silt and clay grade matrix. The deposit is weakly permeable so that flow in the Slough Brook, which has been diverted along the valley side to Chanstone Court Farm, is maintained with little if any loss as it traverses the fanglomerate.

Boulder clay/till: The boulder clay is a silt-bound gravel which is poorly to moderately permeable and provides a cover to the main gravel aquifer. However, the River Dore and its main tributaries are in hydraulic contact with the more permeable lower gravels in some reaches. Although the fine-grained matrix of the till remains stiff after application of water, no clay is present. The till has a low permeability – Hazen analysis of the grain size analyses indicates that the permeability is 100 times lower than the fluvio-glacial gravel below it.

The gravel aquifer: comprises a high energy rounded sandstone cobble to gravel and sand grade material. It is highly permeable. This is a fluvio-glacial deposit which was dumped by high energy glacial outwash activity as a basal lag, coarse-grained, alluvial sediment. The fluvio-glacial gravel is generally found below the till and above weathered bedrock along the deeper parts of the incised valley. The deposit is mainly composed of rounded to sub-rounded gravels and pebbles with a silt and sand grade matrix.

Devonian Bedrock: comprises the mudstones and siltstones of the *Raglan Mudstone Formation*. Along the valley sides the uppermost member of the Raglan Mudstone Formation, the *Bishop's Frome Limestone Member*, is exposed. A line of springs discharges from the base of the limestone over the weakly permeable mudstones and siltstones. Higher up the valley sides, and above the Bishop's Frome Limestone Member, are the basal units of the *St Maughans Formation*, comprising mudstone and siltstone with sub-ordinate multicoloured sandstone horizons. The Devonian bedrock is essentially weakly permeable apart from the limestone units and occasional sandstone horizons in the mudstones and siltstones. The divide between glacial lag deposit and bedrock is characterised by a shallow zone of soft frost-shattered weathered bedrock. The weathered bedrock appears to be disaggregated in the deeper parts while frost-shattered along the valley sides.

Permeability values determined by Vivash (1983) for the main gravel aquifer and the overlying material are:

Upper silty sandy soil	0.3 – 5.0 m d^{-1}
Reworked till gravels	2.0 – 10 m d^{-1}
Main sand and gravel aquifer	50 – 90 m d^{-1}

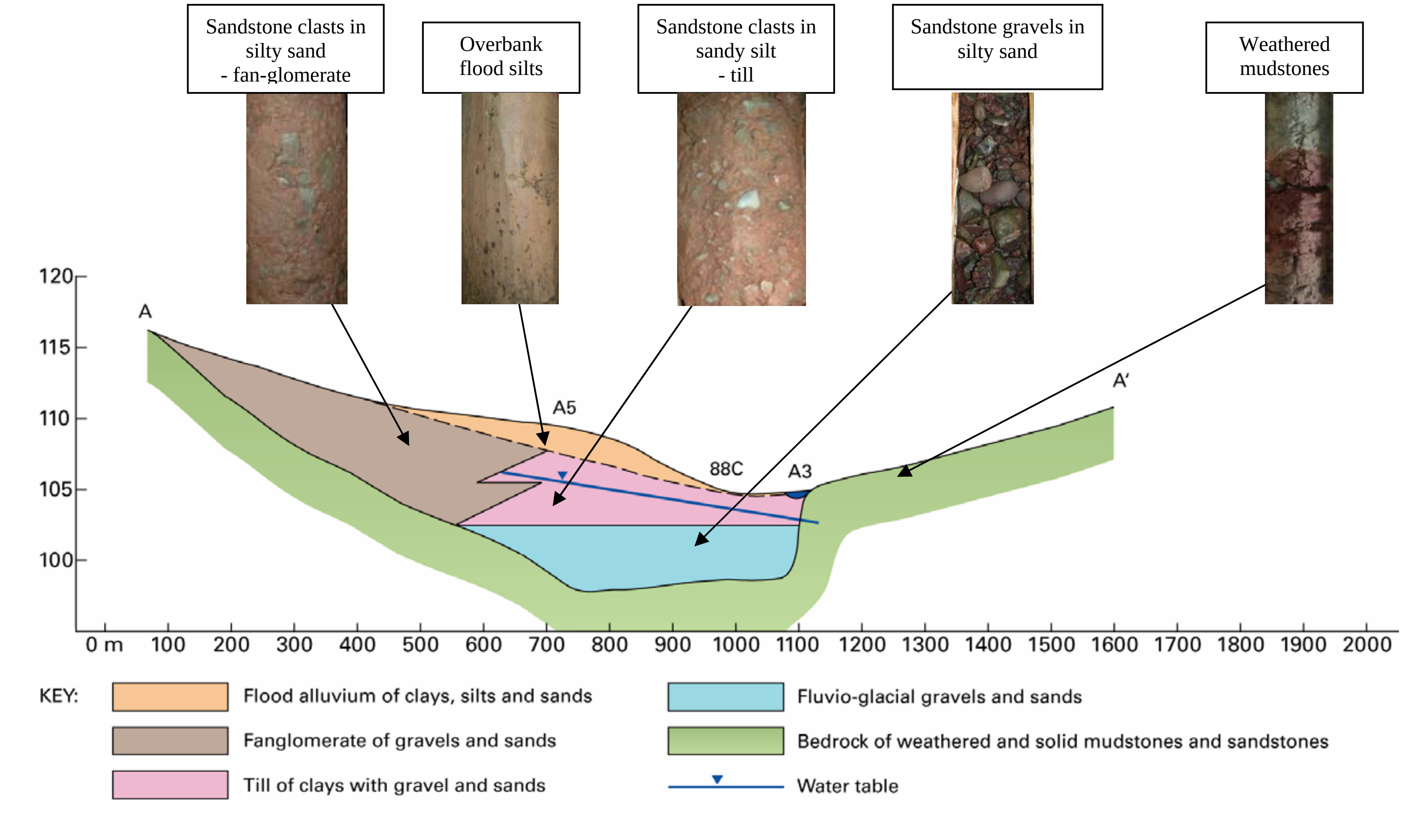
3 3-D Geological Model

The distribution of the superficial sediments at surface is given in the 1: 50 000 scale geological map, Talgarth Sheet 214. The 3-D spatial distribution of the gravel deposits is derived from the published map and from more than 20 boreholes reported by Vivash (1983) and ESI (2006) along with the interpreted data from a series of vertical electrical resistivity soundings. The 3-D model also draws on the feasible configuration of deposits according to their depositional character.

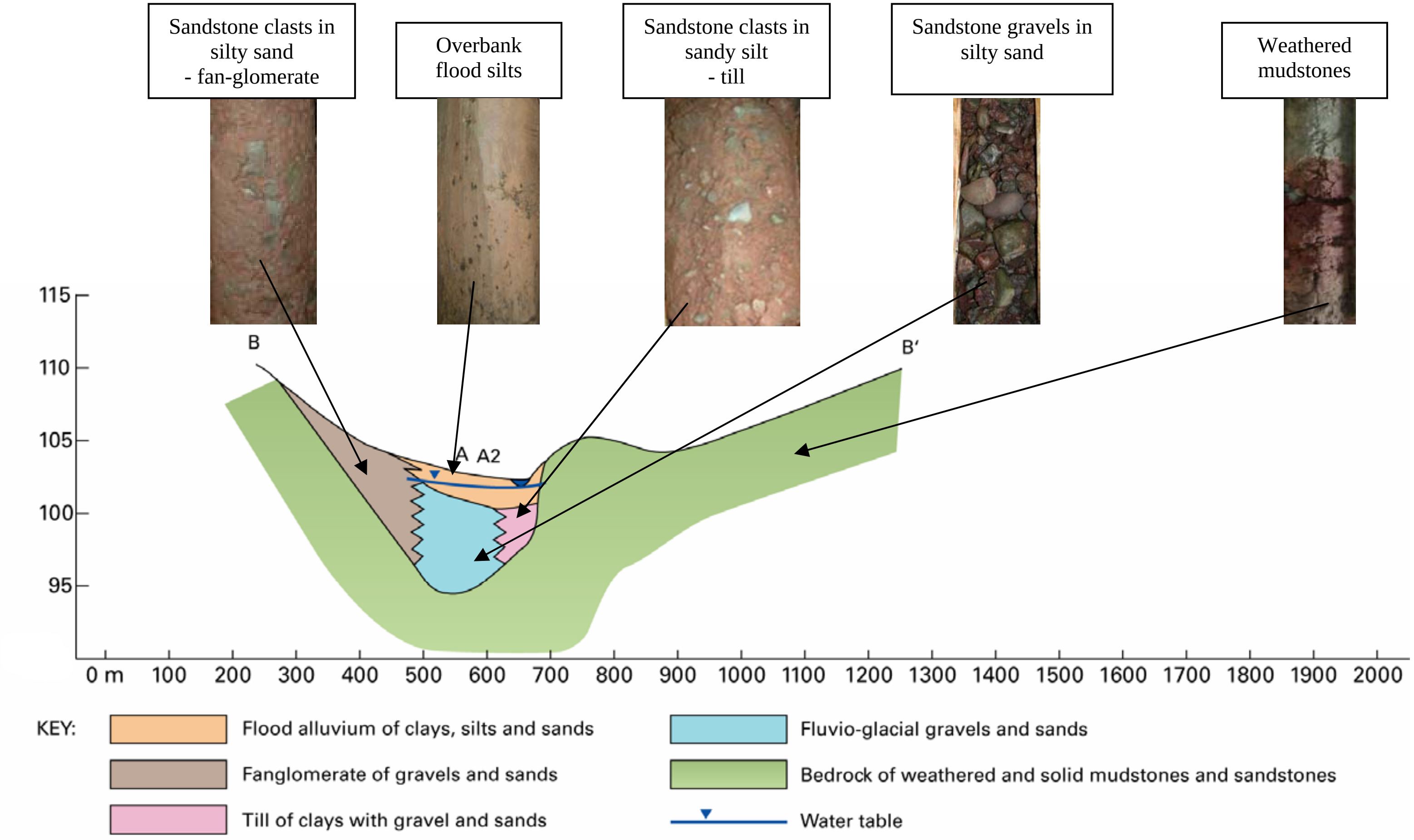
The distribution of the lithologies identified from the borehole cores are shown on the geological cross sections, Figure 2a-g, and their locations on the map Figure 3. Section C – C', follows the line of the pipeline trench. Note that Bh 88C is adjacent to Bh89.

At the location of the proposed pipeline crossing, the river Dore is cut into clays, silts and sands of the flood bank alluvium which in turn rests on till comprising clay-bound gravel and sand. The Slough Brook is cut into the till material and is locally perched at and downstream from the proposed pipeline crossing for a distance of about 1 km. Trenching of these materials will not require any dewatering at the Slough Brook, but will require some drainage at the Dore crossing. However, the important issue with trenching is that it will not invade the gravels forming the main alluvial aquifer and thus does not pose a hazard to the abstraction boreholes downstream. Trenchless crossings of either the River Dore or the Slough Brook would involve a risk of penetrating the aquifer and, therefore, pose a risk of contaminating it. The trench option for both crossings thus posses the least risk to the aquifer.

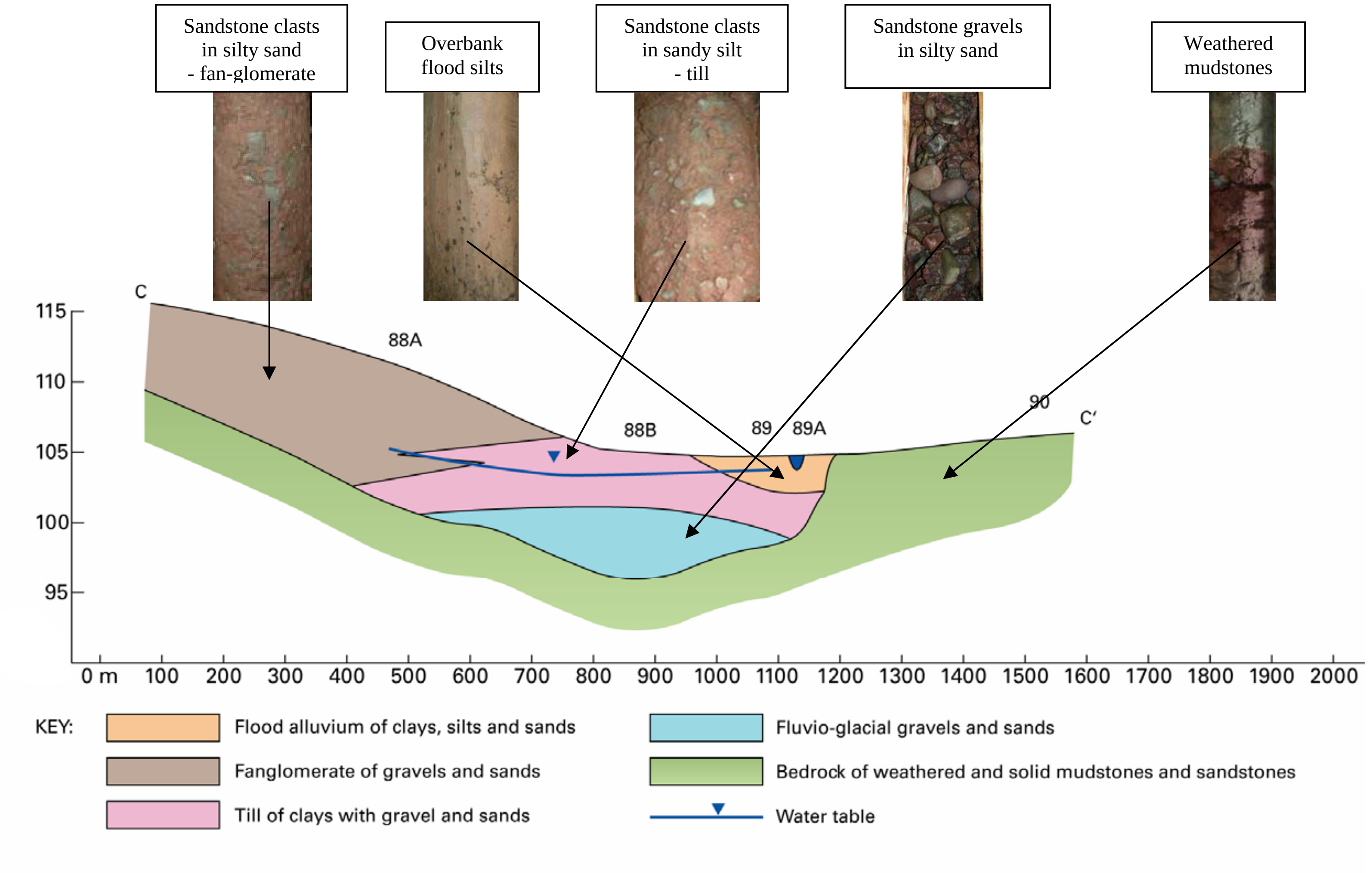
Figure 2. (a) –(g) Geological cross sections – see Figure 3 for locations



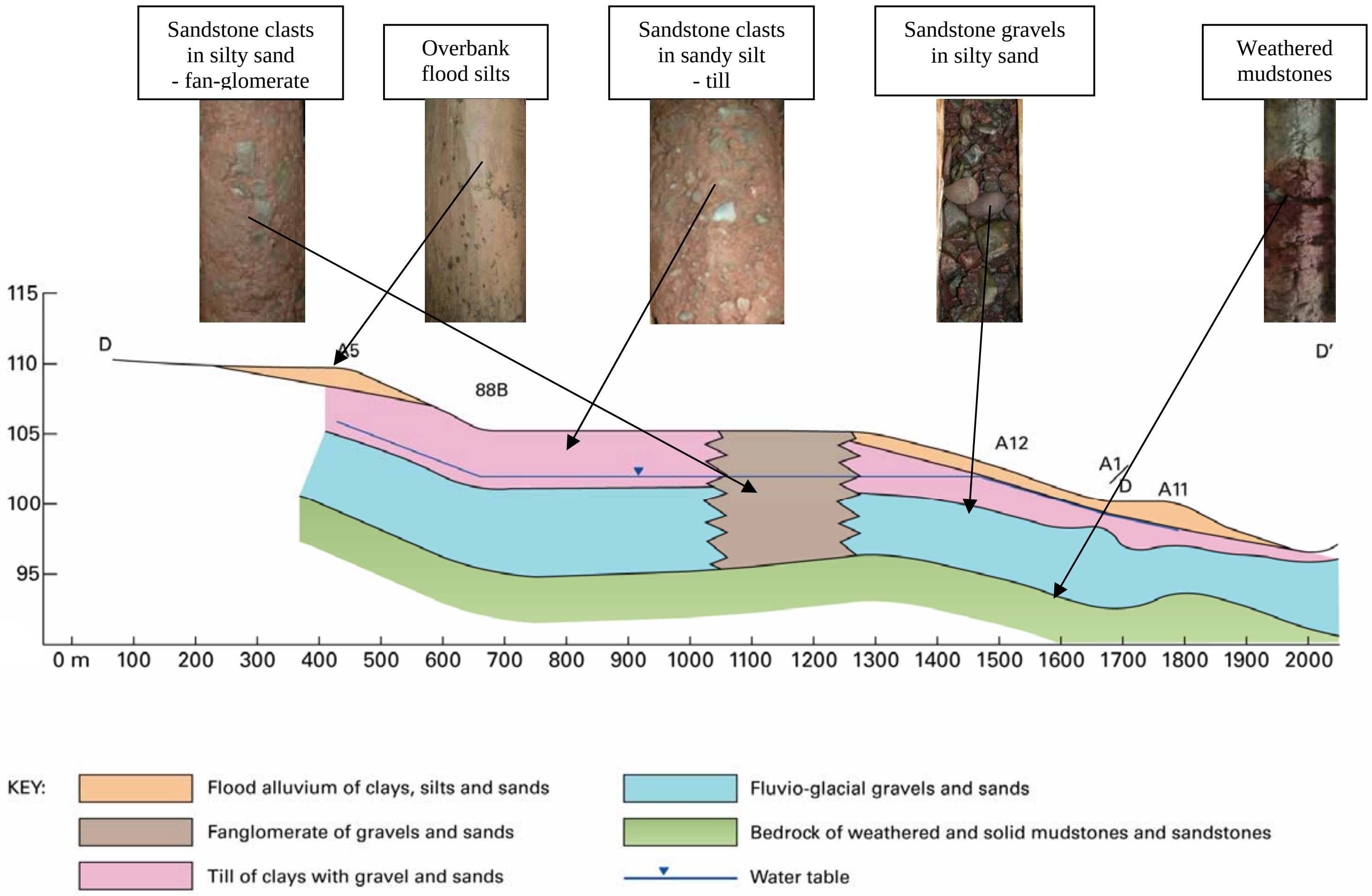
Fluvio-glacial and alluvial deposits of the Vowchurch area – section A-A'



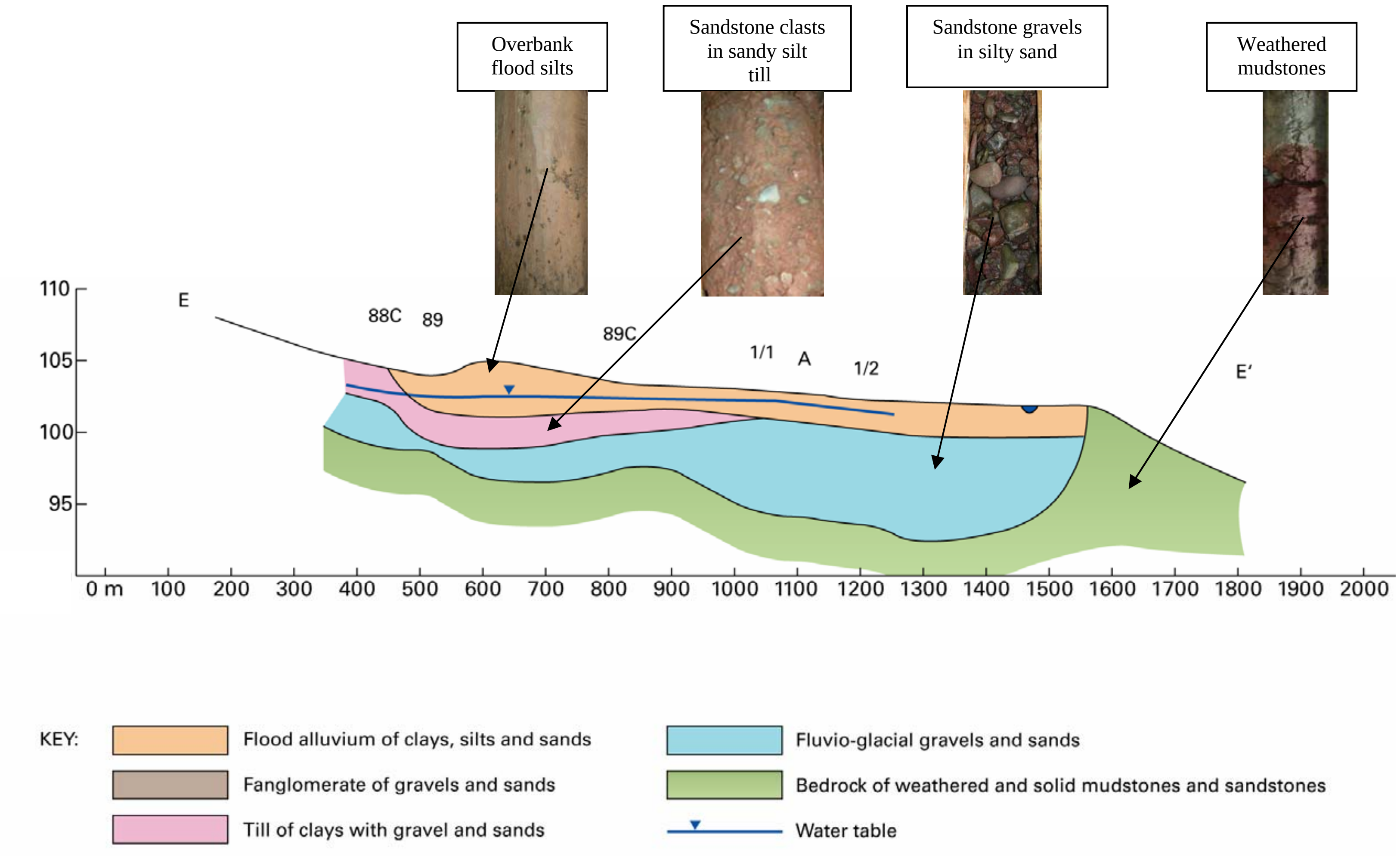
Fluvio-glacial and alluvial deposits of the Vowchurch area – section B-B’



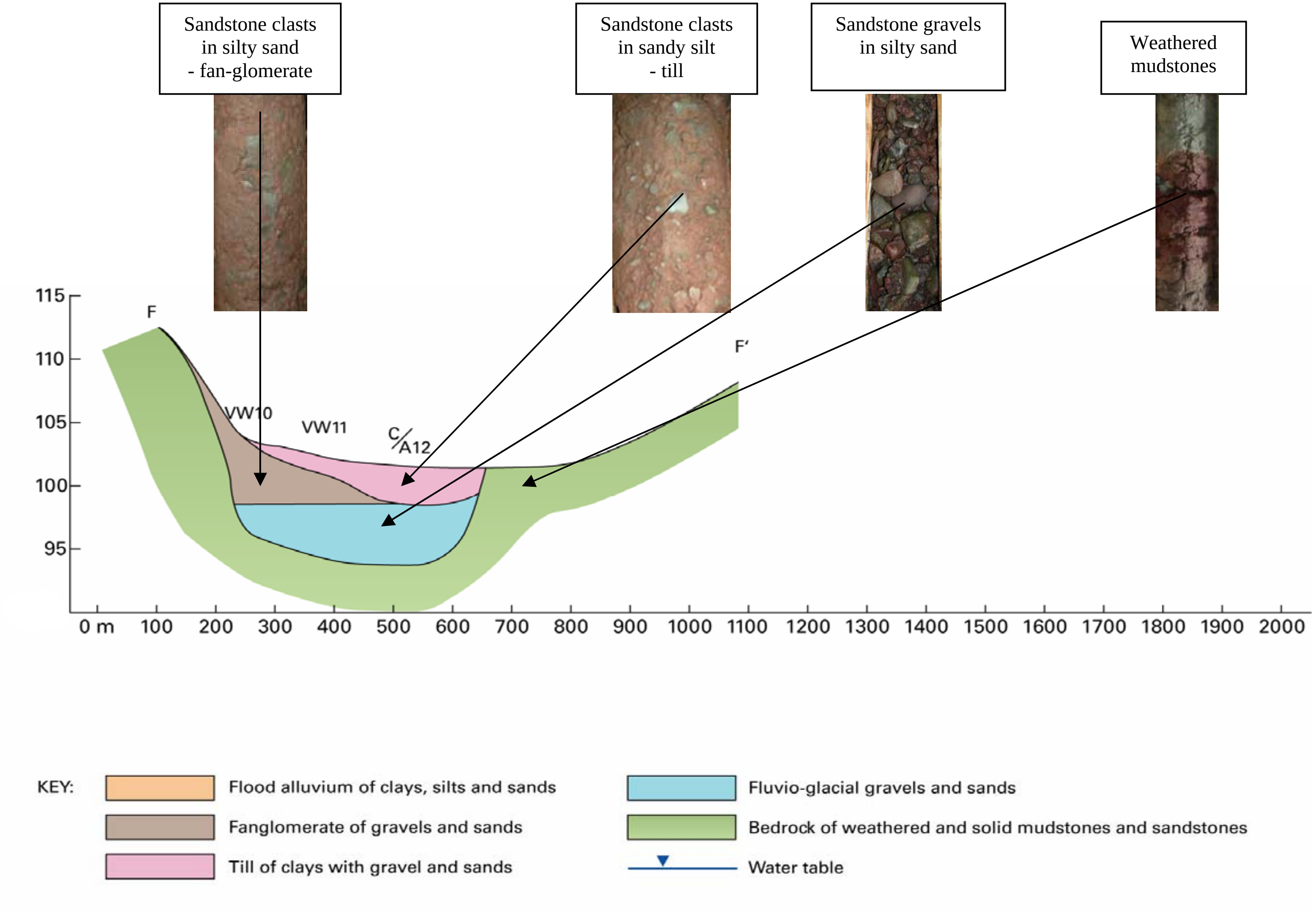
Fluvio-glacial and alluvial deposits of the Vowchurch area – section C-C’



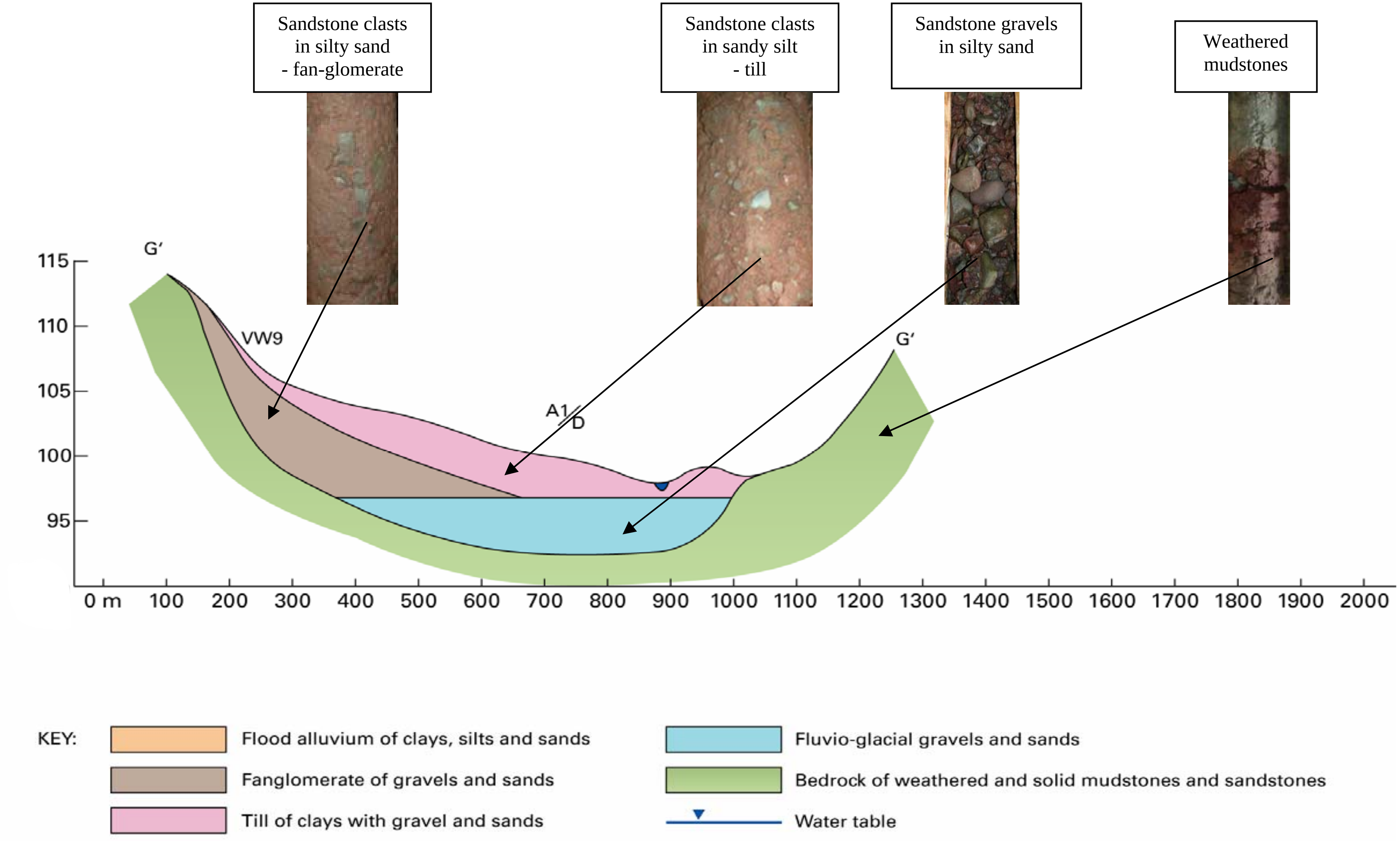
Fluvio-glacial and alluvial deposits of the Vowchurch area – section D-D’



Fluvio-glacial and alluvial deposits of the Vowchurch area – section E-E'

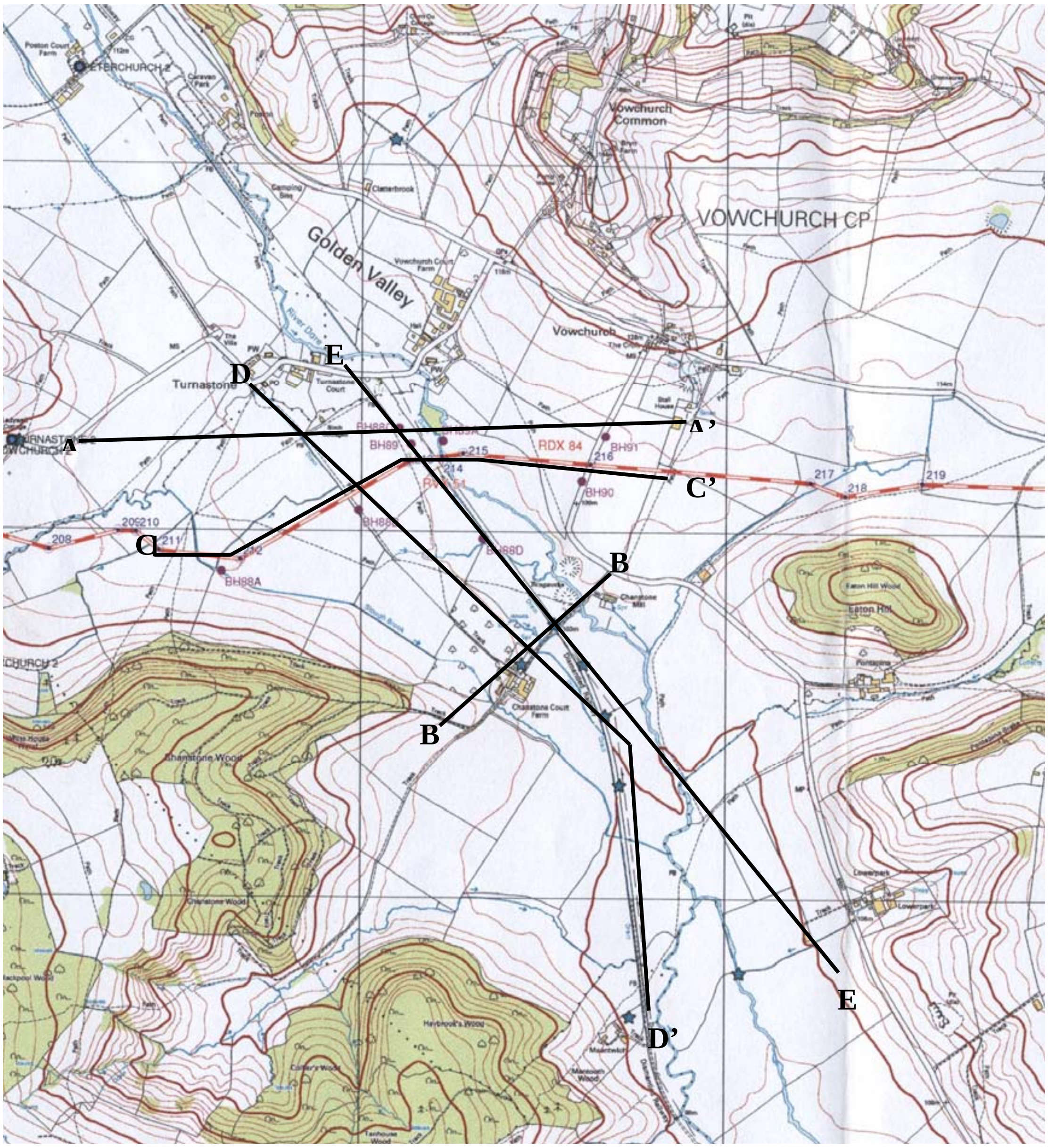


Fluvio-glacial and alluvial deposits of the Vowchurch area – section F-F'



Fluvio-glacial and alluvial deposits of the Vowchurch area – section G-G'

Figure 3. Section locations, note C-C' is the proposed route of the pipeline; B-B' passes through the Vowchurch pumping station



4 Conceptual Groundwater Flow Model

The key considerations in the Vowchurch reach of the Golden Valley are:

- The River Dore is largely underlain by low permeability silt deposits that inhibit infiltration, although it is in direct hydraulic contact with the gravel aquifer in some reaches.
- The flood plain is underlain by 2 – 4 m of till that has low to moderate permeability.
- The gravel aquifer is ‘dirty’ containing a sand and silt grade matrix and pebbles and cobbles. It is of variable, generally high permeability with the vertical permeability less than the horizontal permeability.
- The deepest parts of the Dore Valley are located along a central buried valley, whilst the current course of the Dore hugs the eastern flank of the deposits.
- The Slough Brook is perched on the fanglomerate, and only loses water in the lower reaches towards Chanstone Court.

Recharge to the gravel aquifer may occur from direct rainfall recharge through the overbank deposits and the till, by secondary recharge from river and stream losses, or overland flow down the valley sides. The gravel is poorly to moderately sorted and contains material down to silt grade. The permeability varies areally and with depth, but transmissivity is greatest along the buried valley which continues downstream through the public water supply wellfield. The observed water table reflects the increased transmissivity along the central portion of the gravel, the buried valley, with leakage from the River Dore on the east and ingress from overland flow in the west. Upstream loss of water from the River Dore to the gravel flows down the buried valley where it is partly abstracted at the Vowchurch boreholes, the remainder flowing past the wellfield to rejoin the river before the gravel pinches out downstream. There is a close relationship between groundwater levels and river stage, borehole 88C being nearest to the river and least affected by rainfall recharge (Figure 4). Groundwater flowpaths reflect the transport of river water through the gravel aquifer.

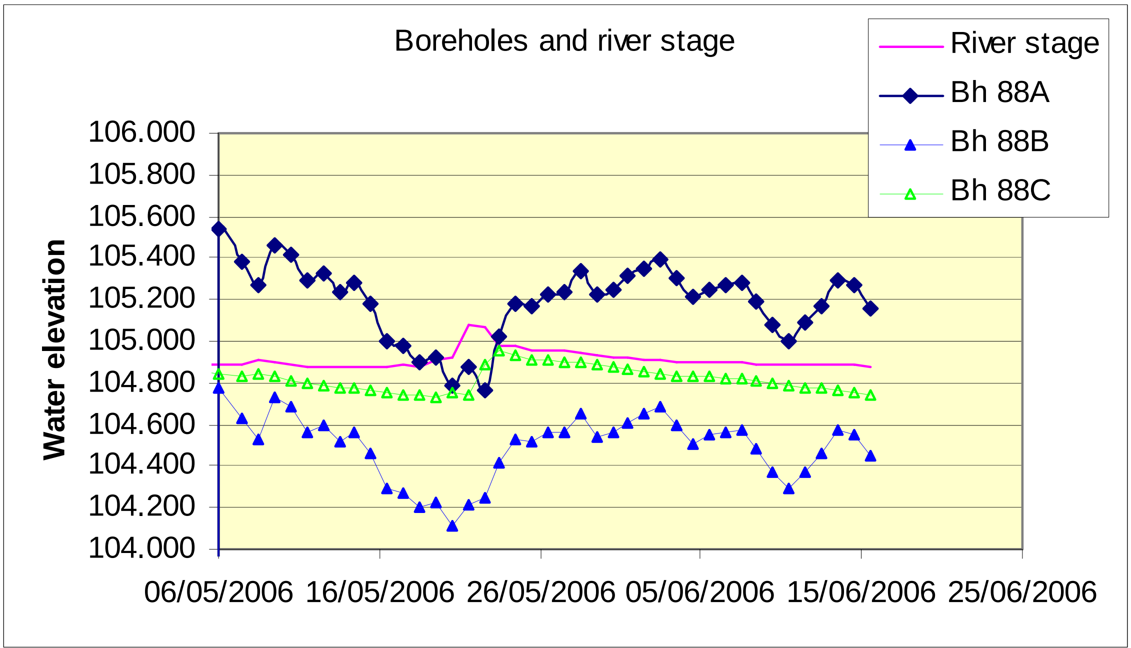


Figure 4. Groundwater levels and river stage (m above OD)

5 Groundwater Modelling

The ZOOMQ3D groundwater flow model and the ZOOPT particle-tracking model (Jackson and Spink, 2004) were used to investigate the impact of the pipeline on the groundwater system and on the public supply boreholes in particular. ZOOMQ3D allows the refinement of the model grid to provide more detail for the areas of most interest, i.e. the abstraction boreholes and the pipeline.

The structure of the model is shown in Figure 5. The area is discretised using a base grid of 100 m square cells. This grid is refined around the pumped boreholes by reducing the cell size to 25 m and along the path of the pipeline to 20 m. While the refinement around the pumped boreholes improves the positioning of these boreholes in the model and the particle tracking procedure in the areas adjacent to them, the refinement along the pipeline improves scaling and allows a more realistic investigation of the system.

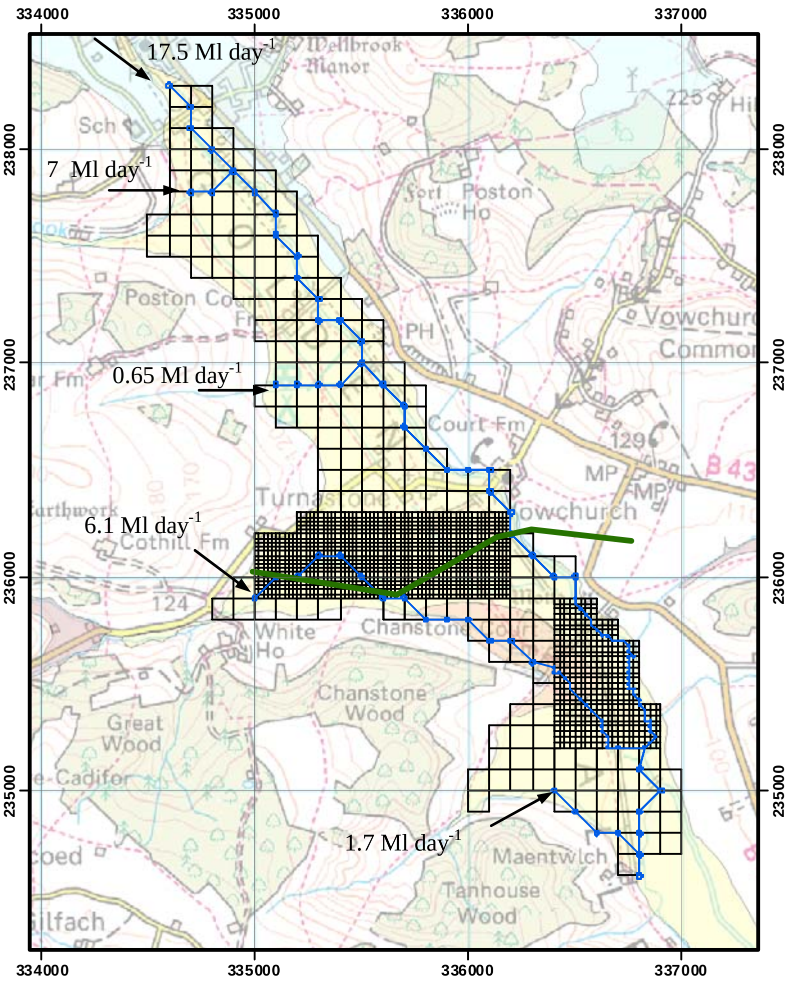


Figure 5. Groundwater model setting.

The boundaries of the model are based on the extent of the gravel aquifer as shown in the 1: 50 000 geological map sheet. The gravel aquifer thins to the north-western and south-eastern ends of the model grid. The adjacent and underlying Devonian siltstones and mudstones are modelled as impermeable. The groundwater heads at the upstream and downstream ends of the aquifer are controlled by the river stage.

The gravel aquifer is represented in the model by either one numerical layer or two numerical layers depending on the representation of the pipeline in the model. In both cases the total saturated thickness of the aquifer is set to 6 m and it is assumed that this thickness remains constant during the whole simulation period. In the numerical model, the riverbed elevations at the nodes representing the River Dore and the Slough Brook are set to values provided by Celtic Water Management on behalf of Welsh Water.

Since the model is to be calibrated against the available historical groundwater level data, the long-term impact of the pipeline on the groundwater flow system can be studied under steady state conditions. It is assumed that rainfall recharge and the water gained via the bed of the river (which includes overland flow from the greater catchment area) are the only sources of water entering the system and that groundwater leaves the system through the abstraction boreholes and downstream back into the River Dore. A numerical recharge model ZOODRM (Mansour and Hughes, 2004) was used to estimate the recharge values distributed over the aquifer. In addition, constant rate and step pumping tests have been undertaken at some of the abstraction boreholes. The results from these tests are analysed using a numerical radial flow model to estimate the values of the hydraulic parameters of the aquifer.

5.1 DETERMINATION OF THE DISTRIBUTED RECHARGE VALUES

The distributed recharge model ZOODRM (Mansour and Hughes, 2004) has been used to estimate distributed recharge. ZOODRM uses rainfall, potential evaporation and runoff to determine the excess rainfall and then applies the Penman-Grindley soil moisture deficit method (SMD) to calculate the evapotranspiration and so estimate recharge. In the SMD method the actual evapotranspiration depends on the excess rainfall, the soil moisture and the crop characteristics represented by the crop root constant (C) and the crop wilting point (D). The difference between the actual evaporation and the excess rainfall either goes to replenish the soil and reduce its moisture deficit or is considered as recharge if the soil moisture deficit is zero.

The recharge process in the alluvium is direct recharge from rainfall infiltrating the ground enhanced by the amount of water originating at and running from the hills in the larger catchment. The run-off (water from the rainfall), run-on (water from run-off to the adjacent node) processes are, therefore, important for the estimation of recharge values. Because of the limited knowledge about the runoff and the absorbed run-off, the evaluation of the runoff and run-on coefficients is associated with a high level of uncertainty. To overcome this problem, several runs with different coefficient values were undertaken to study the possible recharge rates. The comparison of the simulated and available river flows helps refine the values of these coefficients.

While the groundwater model has the same shape as the aquifer, the recharge model area needs to be larger to include the runoff and run-on processes. ZOODRM deals with this through two levels of numerical grids. The upper level grid is used to deal with the surface recharge processes such as rainfall recharge and surface water routing while the lower level grid is used to deal with subsurface water flows, such as routing to springs, and can be also used to pass recharge water to the smaller groundwater model area. The southern limits of the recharge model catchment area are close to Moorhampton Bridge where a gauging station is installed. The other edges of the recharge model catchment area are defined by the catchment watershed and the upstream end of the River Dore and its tributaries as shown in Figure 6.

The catchment area is discretised using a grid with 100 m square cells. Two refining grids have also been included (Figure 6). The locations of these grids are selected so they match the refining grids used in the groundwater model.

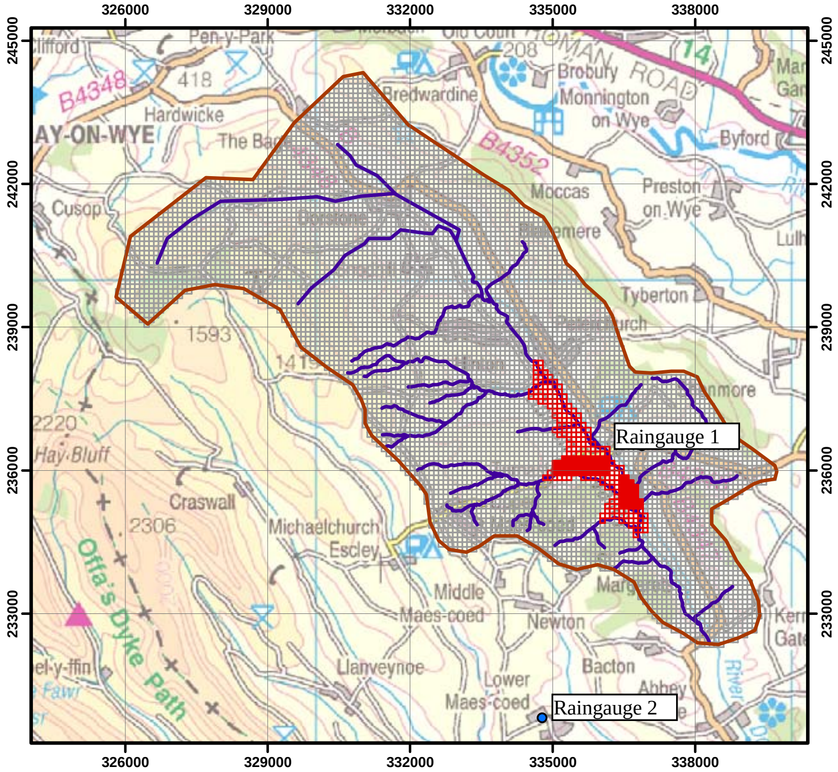


Figure 6. Recharge model area.

There are two raingauge stations in the vicinity. Raingauge 1 is located inside the model area, and Raingauge 2 is to the south of the model area (Figure 6). Raingauge 1 covers the period from 1999 to 2004 and is used as the primary station for rainfall data. The average rainfall value calculated at this station is 2.24 mm day^{-1}. Raingauge 2 covers the period from 1981 to 1997 and is used as a substitute to the first station if longer simulations are undertaken. The average rainfall value calculated at the second rain station is 2.42 mm day^{-1}.

The potential evaporation data have been obtained from the Meteorological Office's MORECS data set. The data are available on a monthly basis and cover the period from 1961 to 2005. The average daily potential evaporation rates of these data are 1.52 mm day^{-1}.

There are no clear surface features that reflect the nature of the underlying rocks. However, the boundary between superficial deposits and bedrock can be distinguished by associating the flatter topographical nature to the former and the hills to the latter. The soil cover, geological and topographical characteristics influencing the runoff and run-on coefficient values, are based on the geological map sheet and are grouped into three sets, two representing the bedrock outcrops, the first for flat lying bedrock and the second for bedrock with steep topography, and the third representing the superficial deposits. Runoff and run-on coefficient values are designated to each set based on the topographical characteristics.

The catchment includes a variety of land use. Only the woodland and built up areas can be clearly distinguished from aerial photos. The root constant C and wilting point D of the crops over these areas are specified to known values derived from Lerner et al. (1990) (both C and D values of 25 mm for the populated areas and C and D values of 203 and 254 mm respectively for the forested areas). These values are kept the same in all simulations. The land use activity is assumed to be the same over the rest of the studied area and the C and D values are changed as shown Table 1.

Several runs have been undertaken to estimate recharge values. The Long Term Average (LTA) recharge values calculated over the period from 1982 to 2004 are used to investigate the effects of each parameter. Table 1 summarises the values set for each parameters in each of six different runs and shows the estimated runoff component of the flow derived for the Moorhampton Bridge river gauging station and the average recharge value in mm day^{-1} estimated over the groundwater model area. Run 6 investigates the effect of using monthly C and D input (Table 2). Figures 7 to 9 show the distributed recharge output from runs 1, 3 and 6 respectively.

Table 1. Parameter values set for each recharge run

Simulation number	Runoff	Run-on (m^{-1})	C (mm)	D (mm)	River Flow (Ml day^{-1})	Recharge over groundwater model area (Ml day^{-1})	Recharge over groundwater model area (mm day^{-1})
Run 1	0.5	0.0005	13	51	62.27	1.61	0.49
Run 2	0.5	0.005	76	127	62.27	1.15	0.35
Run 3	0.5, 0.7 and 0.2	0.001, 0.0005 and 0.0015	13	51	50.24	3.18	0.96
Run 4	0.5, 0.7 and 0.2	0.0005, 0.0001 and 0.001	13	51	61.23	2.97	0.9
Run 5	As Run 3	As Run 3	76	127	50.24	2.65	0.8
Run 6	As Run 3	As Run 3	See Table 2	See Table 2	50.24	2.9	0.88

Table 2. Monthly values of C and D (mm) used in Run 6 (after Lerner et al., 1990)

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
C	25	56	56	76	97	140	140	140	140	25	25	25
D	25	25	102	127	152	203	203	203	203	25	25	25

The results presented in Table 1 show that the change of the crop type results in a difference in the estimated LTA recharge values in the order of 12.5 to 40 % (LTA recharge value drops from 0.49 mm day^{-1} in Run 1 to 0.35 mm day^{-1} in Run 2 and from 0.9 mm day^{-1} in Run 4 to 0.8 mm day^{-1} in Run 5). The change of the runoff and run-on coefficient values on the other hand result in a change as high as 96 % in the estimated LTA recharge values (LTA recharge value increase from 0.49 mm day^{-1} in Run 1 to 0.96 mm day^{-1} in Run 3). While the run-on coefficient value may not have significant influence over the estimated recharge values it has significant impact of the river flow values as can be seen by comparing the results of Runs 4 and 5.

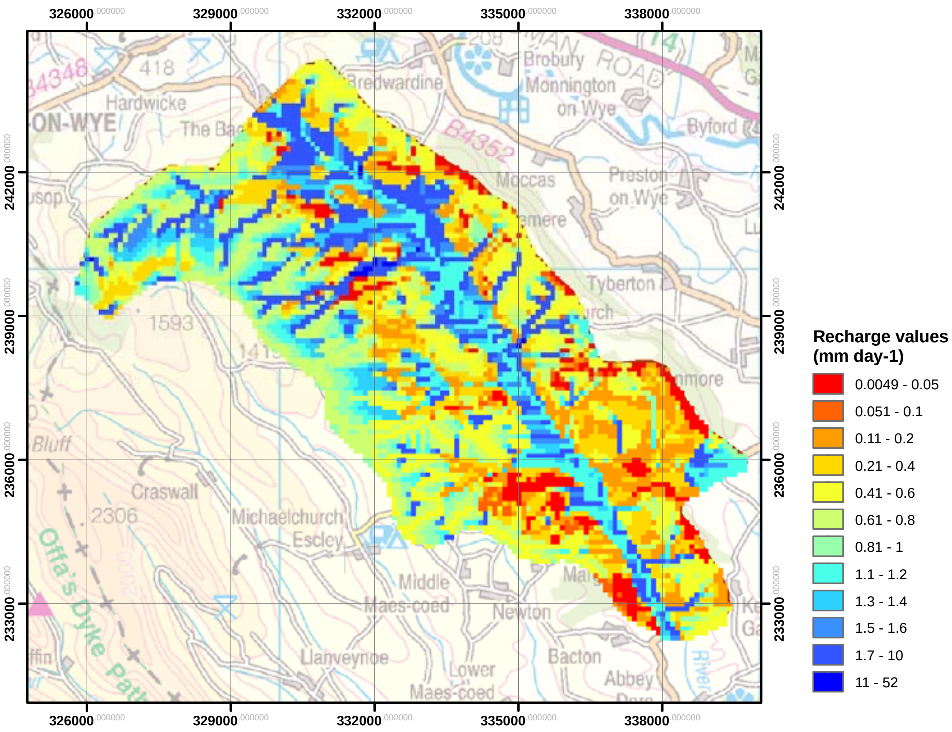


Figure 7. Run 1 simulated LTA recharge values.

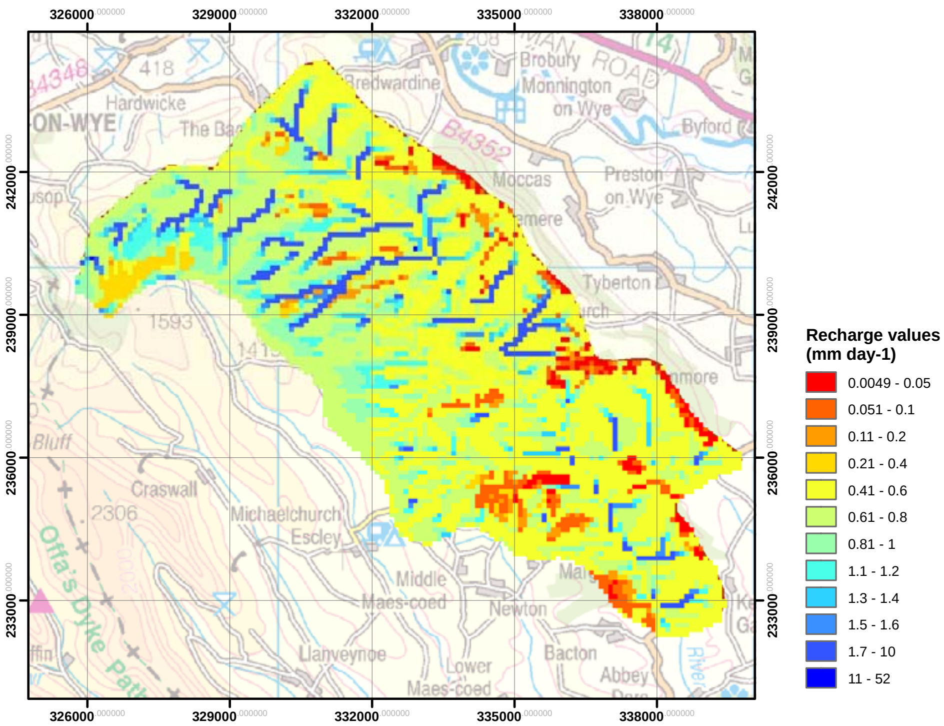


Figure 8. Run 3 simulated LTA recharge values.

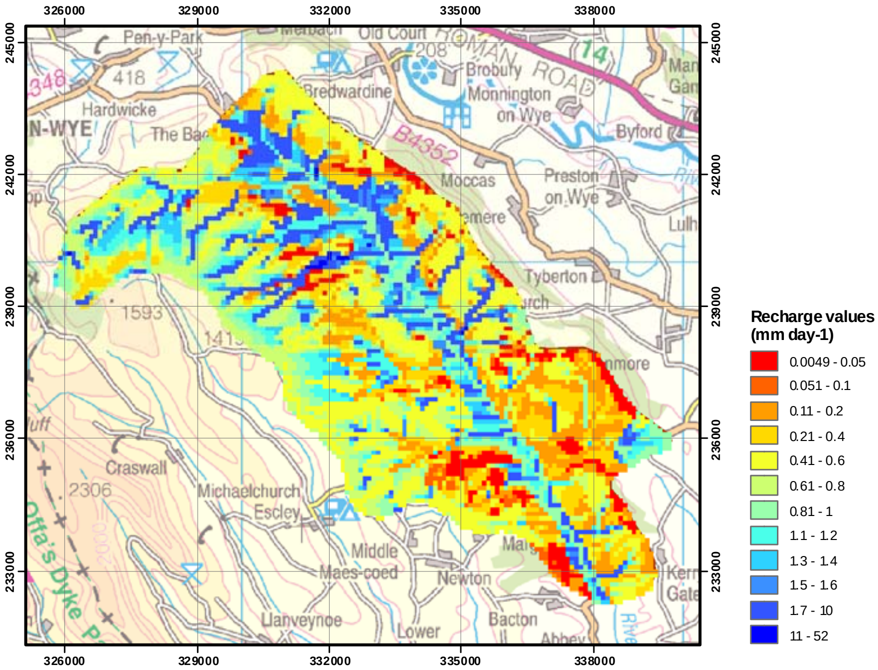


Figure 9. Run 6 simulated LTA recharge values.

The runoff and run-on coefficient values can be refined by matching the estimated and observed river flows. However, river flow data (rather than river stage data) are not available for the local gauging stations and resort has to be made of flow data from adjacent catchments. The hydrometric register and statistics of hydrological data between 1996 and 2000 (CEH/BGS, 2003) indicate that the nearest gauging station is located on the River Monnow, of which the River Dore is a tributary. The size of the catchment area above this gauging station is 357.4 km^2 and the statistics show that the average river flow is approximately 510 Ml day^{-1} with a baseflow index of 0.5, i.e. an average runoff component within the river flow of 255 Ml day^{-1}. Assuming that the Dore catchment possesses the same characteristics as the area related to the gauging station on the River Monnow, and down scaling to just the area of the Vowchurch model (71.21 km^2), the average runoff component of river flow at Moorhampton Bridge is calculated to be 50.8 Ml day^{-1}. This suggests that the LTA recharge results of Run 3, 5 or 6 are all reasonable estimates, with run 5 providing the overall best fit to the observed and monitored response of the aquifer.

5.2 ANALYSIS OF PUMPING TEST DATA

The Welsh Water Authority report on the production boreholes at Vowchurch (Vivash, 1983), includes field results for a pumping tests carried out on the then newly completed abstraction boreholes. The durations of the abstraction and recovery phases are not explicitly given but have been estimated from the available plots. The data have been reanalysed using the finite-difference radial flow model developed jointly by the University of Birmingham and BGS (Mansour, 2003) in order to illustrate the hydraulic characteristics of the aquifer and to estimate appropriate values for horizontal and vertical hydraulic conductivities, specific storage and specific yield.

5.2.1 Borehole A - Step Test

Table 3 shows the abstraction rates and the durations of both the abstraction and recovery phases of the step test at Borehole A. The borehole is 11.5 m deep (Figure 10).

Table 3. Abstraction rates and durations of the abstraction and recovery phases of Borehole A Step test.

	Abstraction phase		**Recovery phase**
	Abstraction rate (m^3 day^{-1})	**Duration (days)**	**Duration (days)**
Phase 1	573	0.0417	0.0208
Phase 2	742	0.0417	0.0208
Phase 3	1036	0.0347	0.0417
Phase 4	1396	0.0417	0.104
Phase 5	1898	0.0625	0.104

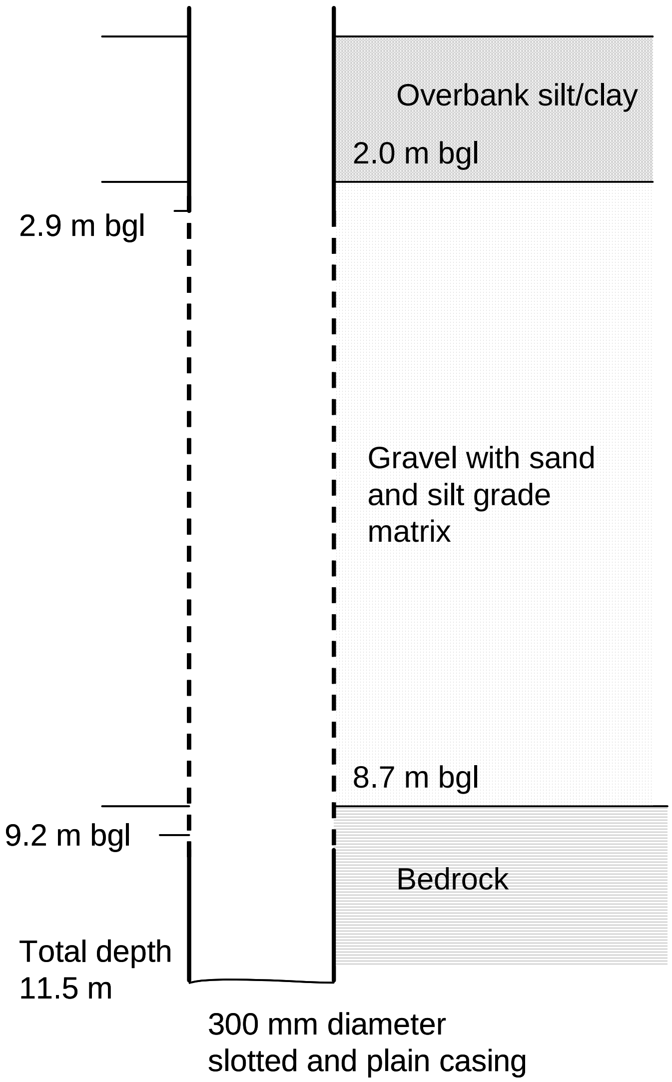


Figure 10. Schematic log of Vowchurch Borehole A.

The numerical model comprises one finite-difference layer in one vertical plane. This simple form of model is acceptable if the aquifer is assumed to be radially symmetric. The vertical movement of the groundwater flow observed in unconfined aquifers is represented by the inclusion of an additional finite difference layer that represents the water table. The only flow process missing in this setting is the seepage face.

The abstraction well radius is set to 0.15 m in the numerical model. The outer boundary of the model is taken to be impermeable and located remotely at a distance of 10 km from the abstraction well. This is sufficiently far from the abstraction borehole, given the abstraction rates and pumping durations stated in Table 3, to have no effect on the results. The aquifer thickness is set to 6.0 m.

The first attempt to simulate the time drawdown curve observed at Borehole A assumed unconfined conditions, and after several numerical modelling runs an acceptable match with the time drawdown curves was achieved (Figure 11). The horizontal hydraulic conductivity value used to produce the numerical curves was 195.0 m day^{-1}, i.e. a transmissivity of 1170 m^2 day^{-1} with an aquifer saturated thickness of 6.0 m. This value is within the limits reported by Vivash (1983). The vertical hydraulic conductivity was set at 5 m day^{-1} in this simulation. The specific storage and specific yield values were set to large values of 0.005 m^{-1} and 0.3. However, the specific storage is unreasonably large and yields almost no numerical response at the observation wells.

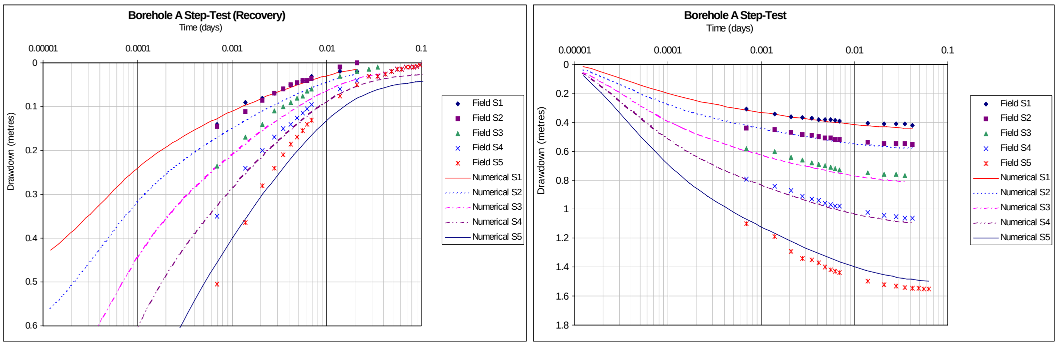


Figure 11. Simulated and observed time-drawdown curves at abstraction Borehole A assuming unconfined aquifer system.

An attempt to match the numerical and field results was also undertaken using a confined aquifer system. In this case the river is the only source of water that can be included for the numerical time-drawdown curve to replicate the behaviour of the field curve. The river is represented in the radial flow model by imposing fixed head conditions at the grid nodes located along the river. To improve the representation of the groundwater flow movement in the model the aquifer is represented by two finite difference layers. This allows the fixed head conditions to be imposed at some points of the upper numerical layer while maintaining the possibility that the abstraction may propagate below the river through the lower layer. The horizontal circumferential direction is also included in the model to represent the river. The model is used as a full three-dimensional R-θ-Z model.

The effect of abstraction propagates over large distances in relatively short periods in a confined aquifer system. Therefore, it is most likely that the impermeable boundary conditions caused by the bedrock cropping out at the edges of the valleys will impact the observed time-drawdown curves. These boundaries will increase the rate of drawdown with time and are included numerically by eliminating all nodes that are located beyond two parallel lines set at a certain distance to the left and right of the abstraction borehole

(Figure 12). Although the aim is to set the distance between these lines to be equal to the valley width it is not possible to achieve this in a radial flow model since the spacing of the more distant nodes may be greater than the valley width.

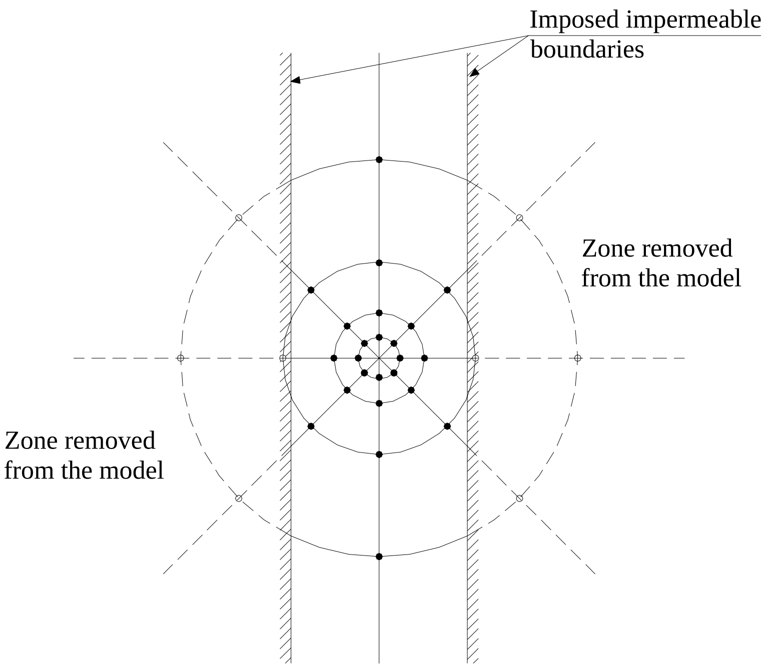


Figure 12. Imposed impermeable boundaries in the radial flow model.

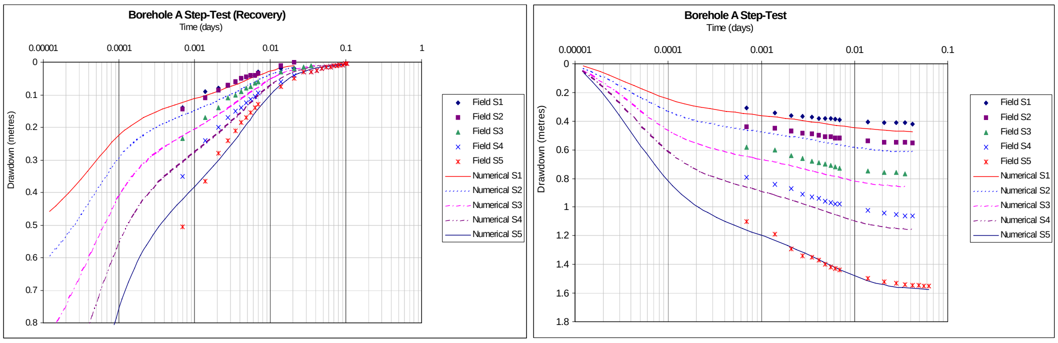


Figure 13. Simulated and observed time-drawdown curves at abstraction Borehole A assuming confined aquifer system.

Figure 13 shows a comparison between the simulated and observed time-drawdown curves at Borehole A using the hydraulic parameters given in Table 4. These parameters are close to those found when the aquifer is considered under unconfined conditions except that the value of specific storage is much lower at 0.00012 m^{-1}. It must be noted that while there is no water release from the water table in the confined condition, the river provides an alternative source of water causing the time drawdown curves to flatten out towards the end of the abstraction phase. It is concluded that the river has a pronounced impact on the time-drawdown results even with the relatively small value for the vertical hydraulic conductivity (5 m day^{-1}).

Table 4. Hydraulic parameter values of the gravel aquifer assuming that the system is under confined conditions – 2 layer analysis of Borehole A data

	Layer 1	Layer 2
Layer thickness	1.0	6.7
K_h (m day^{-1})	260.0	260.0
K_v (m day^{-1})	5.0	5.0
S_s (m^{-1})	0.00012	0.00012

The simulated and field time-drawdown curves at the observation borehole (Borehole A2) located at approximately 40 m away from the abstraction borehole are also compared. It is clear that these curves are in good agreement as shown in Figure 14. However, the geometry of the valley, i.e. the location of the impermeable boundaries, is as an important influence on drawdown. For example, the time-drawdown curves for Borehole A produced a transmissivity value that is higher than the transmissivity value required to match the curves at Borehole A2. The match produced in Figure 14 was produced by using the transmissivity value obtained from the analysis of the curves at Borehole A but moving the impermeable boundary nearer to Borehole A2. This impermeable boundary causes an increase of drawdown rate with time without causing a significant impact on the drawdown values at the abstraction borehole A.

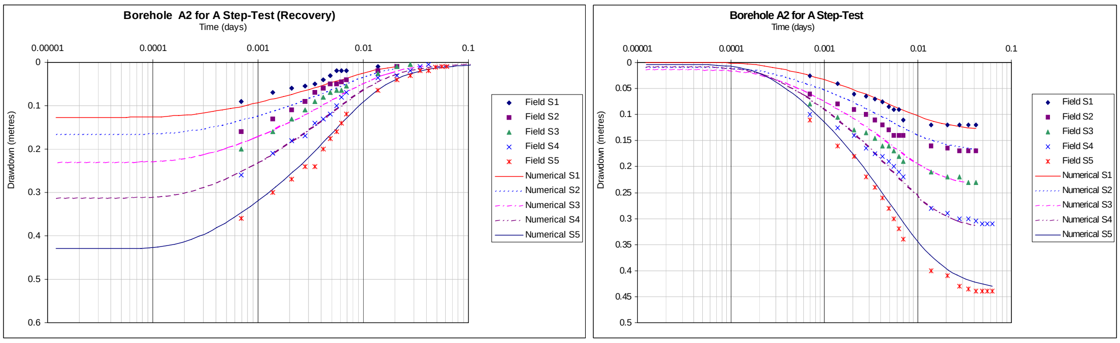


Figure 14. Simulated and observed time-drawdown curves at observation borehole A2 assuming confined aquifer system.

The transmissivity value calculated using the horizontal hydraulic conductivity value shown in Table 4and an aquifer-saturated thickness of 6.7 m is 1742 m^2 day^{-1} and is within the range of values given by Vivash (1983). The specific storage value for the confined aquifer system is compares acceptably with the value produced for an unconfined aquifer system. The good match between the simulated and observed time-drawdown curves at the observation well is a further check on the validity of this storage value and possibly also the validity of the assumed confined aquifer system.

5.2.2 Borehole C - Constant Rate Test

Borehole C is 9.0 m deep and also has a completion diameter of 0.3 m. Slotted casing has been placed from 3.4 to 7.5m bgl against the gravel aquifer. The upper part of the gravel consists of gravel with traces of fine sand and silty clay particles, and the lower part of medium to coarse gravel.

The constant rate test observed at this borehole includes time drawdown curves for the abstraction and one observation borehole, A12 located 4.2 m away from Borehole C (Vivash, 1983).

As with the conceptual model for the step test in Borehole A, the aquifer system is assumed to be confined, the river is represented by fixed head nodes and the valley boundaries are defined. In the numerical model the abstraction well radius is set to 0.15 m. The outer boundary is assumed to be impermeable and located at a distance of 10 km from the abstraction well. The valley edges are included in the model by eliminating the grid nodes that are located beyond a specified distance from the abstraction well. Three layers are included in the model. The first two represent the upper gravel unit and the third and lowest layer represents the lower gravel unit. The aim of representing the upper gravel unit by two numerical layers is to provide the model with grids of nodes which can be specified as fixed head nodes without compromising the possibility of groundwater flow propagating beyond these nodes.

Figure 15 and Figure 16 show the simulated and observed time-drawdown curves for abstraction Borehole C and observation Borehole A12 using the hydraulic parameters listed in Table 5 for the three layers. The values of the Layer 3 parameters are those obtained from the analysis of the Borehole A test. The parameter values of Layers 1 and 2 have been modified to improve the fit. However, it was very difficult to improve this fit especially as the locations of the impermeable boundaries representing the edges of the gravel aquifer have a direct influence on the drawdown values.

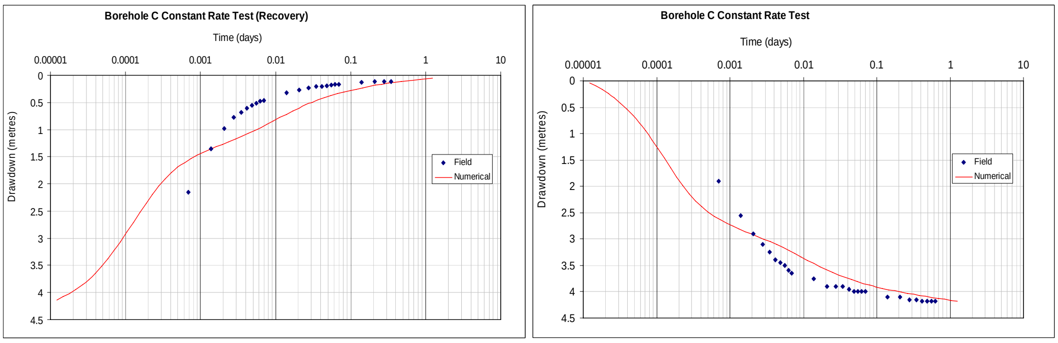


Figure 15. Simulated and observed time-drawdown curves at observation borehole C assuming confined aquifer system

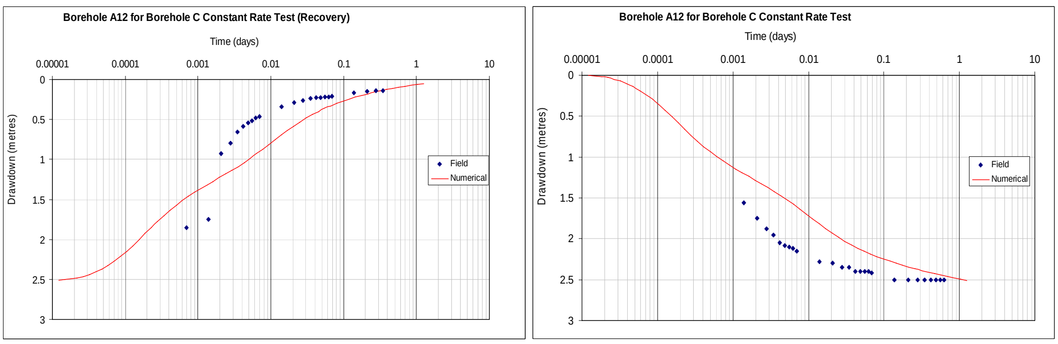


Figure 16. Simulated and observed time-drawdown curves at observation borehole A12 assuming confined aquifer system

Table 5. Hydraulic parameter values of the gravel aquifer assuming the aquifer is under confined conditions – 3 layer analysis of Borehole C test data

	Layer 1	**Layer 2**	**Layer 3**
Layer thickness	2.0	2.0	2.0
K_h (m day^{-1})	120.0	120.0	260.0
K_v (m day^{-1})	5.0	5.0	5.0
S_s (m^{-1})	0.00012	0.00012	0.00012

5.3 GROUNDWATER MODEL DEVELOPMENT

The recharge values obtained from the fifth simulation (Run 5) have been selected as the best fit to observed data and have been applied to the whole groundwater model area (see Section 5.1). Since the rivers are truncated by the edges of the groundwater model, river flow inputs must be specified at the upstream and downstream ends of the model. Gauging stations are, therefore, defined in the recharge model at the locations where the rivers are terminated in the groundwater model. The flows at these stations are monitored over the simulation time and average river flow values are calculated. These river flow values at the upper part of the River Dore and the Slough Brook in Run 5 are 17 Ml day^{-1} and 6.1 Ml day^{-1} respectively as shown in Figure 5. These river flows do not account for the springs originating from the limestone layers.

Pumping test analyses at abstraction boreholes A and C show that the horizontal hydraulic conductivity for the gravel is about 250 m day^{-1} although it can be as low as 120 m day^{-1} when sand and silty clay are present. In addition, it has been found that a vertical hydraulic conductivity value of 5 m day^{-1} is acceptable throughout the aquifer. A vertical hydraulic conductivity value is needed when a two-layer numerical model is constructed.

The numerical simulation used a horizontal hydraulic conductivity of 250 m day^{-1}. The transmissivity value calculated with an aquifer-saturated thickness of 6 m is 1500 m^2 day^{-1}. Figure 17 shows a plot of the simulated groundwater level contours against the average observed groundwater levels. They are in good agreement especially in the southern part of the groundwater model around the abstraction boreholes. The comparison between the observed and simulated groundwater levels in the northern part of the model area, however, indicates that the transmissivity value is too high for this area. Because there are few available observed groundwater levels, the derived groundwater heads are assumed to be acceptable and both the distributed recharge values and transmissivity values are used for the subsequent runs.

5.4 THE IMPACT OF THE PIPELINE ON THE GROUNDWATER FLOW

The trench in which the pipeline will be laid is to be approximately 2 m deep. The pipeline will cause a reduction in the transmissivity value for groundwater flowing perpendicular to the pipe down the valley towards the wellfield. The impact of the pipeline on the groundwater flow system is investigated with two separate approaches.

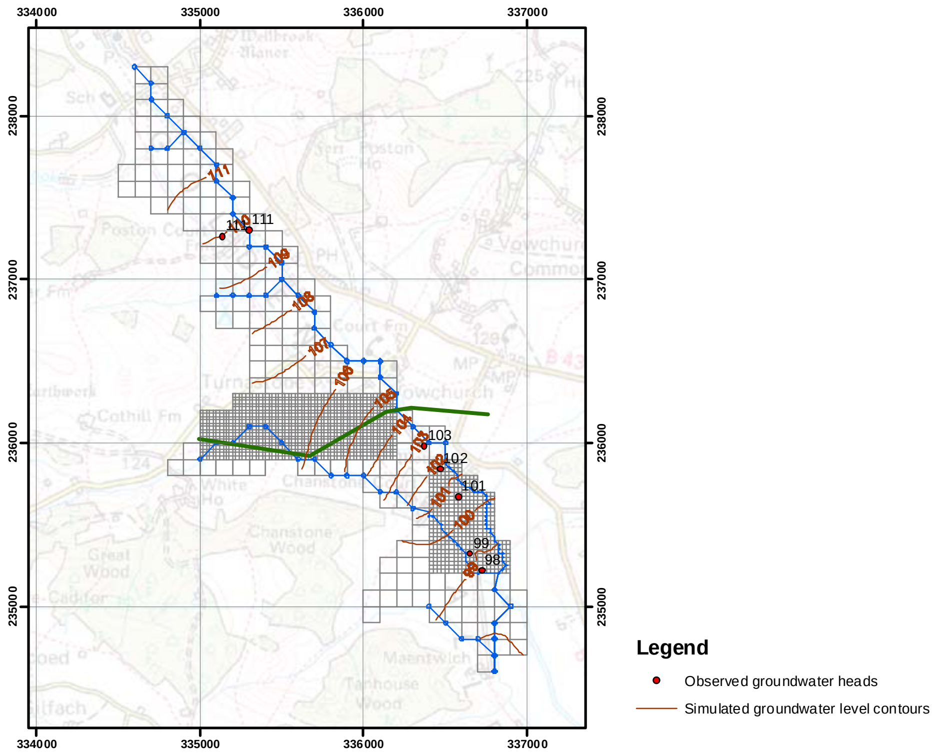


Figure 17. Comparison between the simulated groundwater level contours and the observed groundwater heads.

For the first approach a single layer numerical model is used. The effects of constructing the pipeline and its surrounding materials are accounted for in the model by reducing the values of aquifer transmissivity at the nodes located along the path of the pipeline. Assuming that only the pipeline will inhibit the groundwater flow and that the pipeline diameter is 1.5 m, then one quarter of the aquifer will be made impermeable when the pipeline is constructed. The new transmissivity value at the node representing the pipeline trench is reduced, therefore, from 1500 m^2 day^{-1} to 1125 m^2 day^{-1}. However, this simulation does not reveal any significant changes in the groundwater heads from the original simulation without the pipeline. The plot of the differences between the simulated groundwater heads of this run and those of the original run is shown in Figure 18. This plot shows that the pipeline may cause a 0.06 m groundwater head to build up directly to the north of the pipeline and that a slight decrease (0.02 m) in groundwater heads may occur directly to the south of the pipeline.

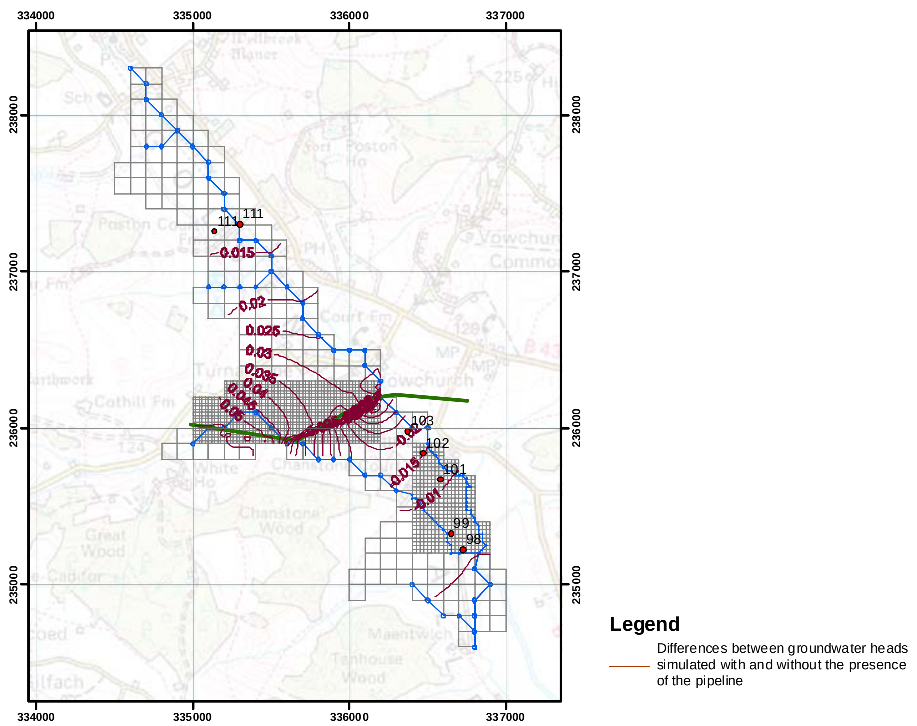


Figure 18. Contours of the differences between the simulated groundwater heads with and without the presence of the pipeline using a one-layer model.

The actual groundwater head changes are expected to be lower than the derived values because the modelled pipeline trench is 40 m wide due to the node separation, whereas the real trench will be only 1.5 m wide. The results of simple analytical solutions confirm the range of the simulated head differences (Figure 19a) and show that groundwater differences can be as small as 0.003 m if a trench width of 2 m is used in the calculations (Figure 19b). These figures derive from an aquifer transmissivity of 1500 m^2 day^{-1} and a hydraulic gradient of 0.005 m m^{-1} along the main flow direction. The transmissivity along the line of the pipeline trench is set to 1125 m^2 day^{-1} in both cases and this increases the hydraulic gradient to a value of 0.007 m m^{-1} yielding a groundwater head differences of 0.083 m and 0.003 m with trench widths of 50 m and 2 m respectively.

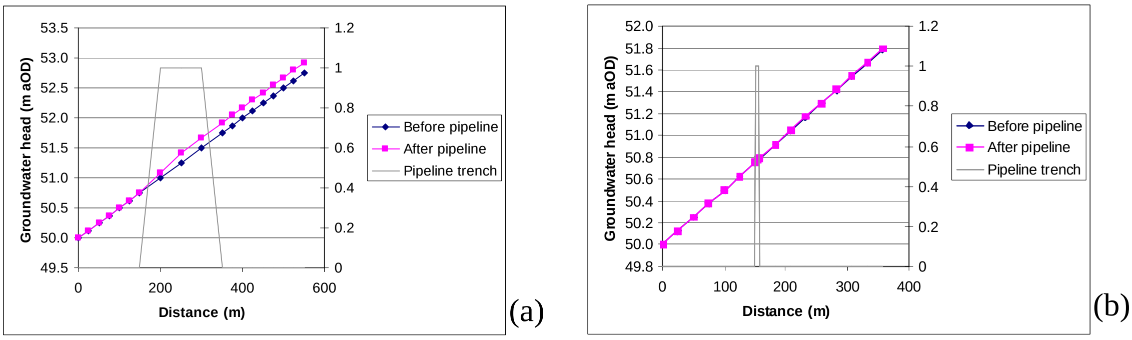


Figure 19. Spreadsheet calculations to investigate the pipeline impact on the groundwater heads. (a) Width of pipeline trench is 50 m. (b) Width of pipeline trench is 2 m.

The impact of the pipeline on river flow is investigated by studying accretion profiles along the River Dore and the Slough Brook. Figure 20a and Figure 21a show the accretion profiles along these rivers respectively when no pipe is included in the model. Figure 20b and Figure 21b show the differences in groundwater heads and flows in these rivers with and without the pipeline. Figure 20b indicates that the flow in the River Dore increases by a maximum of 42.5 m^3 day^{-1} just above the pipe when the pipe is in place. The continuous increase in river flow is not maintained along the whole section of the river because the river loses more water to the aquifer directly after the pipeline where groundwater heads drop. Figure 21b, on the other hand, indicates that the placement of the pipe has no effects on the flows of the Slough Brook for approximately 2000 m from its upstream end. This is because the groundwater heads are below the riverbed of this section of the river. The drop in groundwater heads, however, causes the river flows to decrease by 2.63 m^3 day^{-1} at its downstream end.

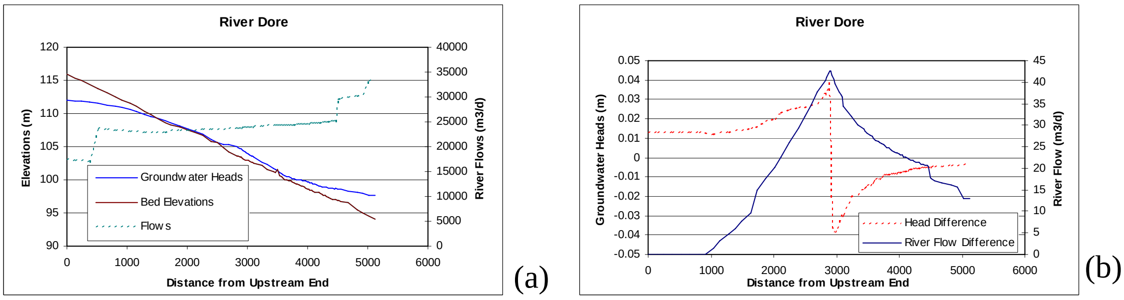


Figure 20. (a) River Dore accretion profile. (b) Differences between groundwater heads at nodes located underneath the River Dore and differences between the River Dore flow with and without pipe.

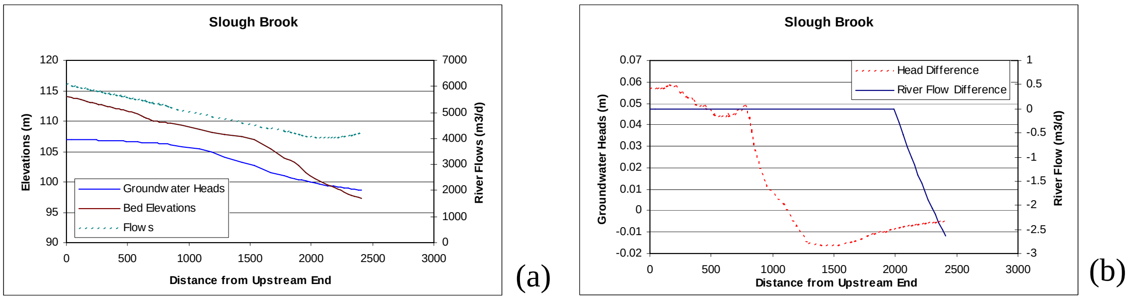


Figure 21. (a) Slough Brook accretion profile. (b) Differences between groundwater heads at nodes located underneath the Slough Brook and differences between the Slough Brook flow with and without pipe.

With the second approach a two-layer model is constructed with saturated thicknesses of the first and second layers set to 2.0 m and 4.0 m respectively. In this simulation it is assumed that both the pipeline and the trench backfill material block the groundwater flow in the whole of the first (uppermost) layer. Consequently, the transmissivity values at the first layer nodes located along the path of the pipeline are set to zero creating a 2.0 m deep flow barrier. The plot of the differences between the simulated groundwater heads of this run and the run without the pipeline simulated in the upper layer of the model are shown in Figure 22. Although the effects of the pipeline in this model setting are amplified by assuming that the fill materials are impermeable, the simulation results show that a groundwater head build up of the order of 0.10 to 0.15 m is required to the north of the pipeline to create a hydraulic gradient that is steep enough to compensate the loss of the cross-section area and to force the groundwater flows beneath the pipeline. The change in heads at the south side of the pipeline is higher than 0.06 m as shown in Figure 22.

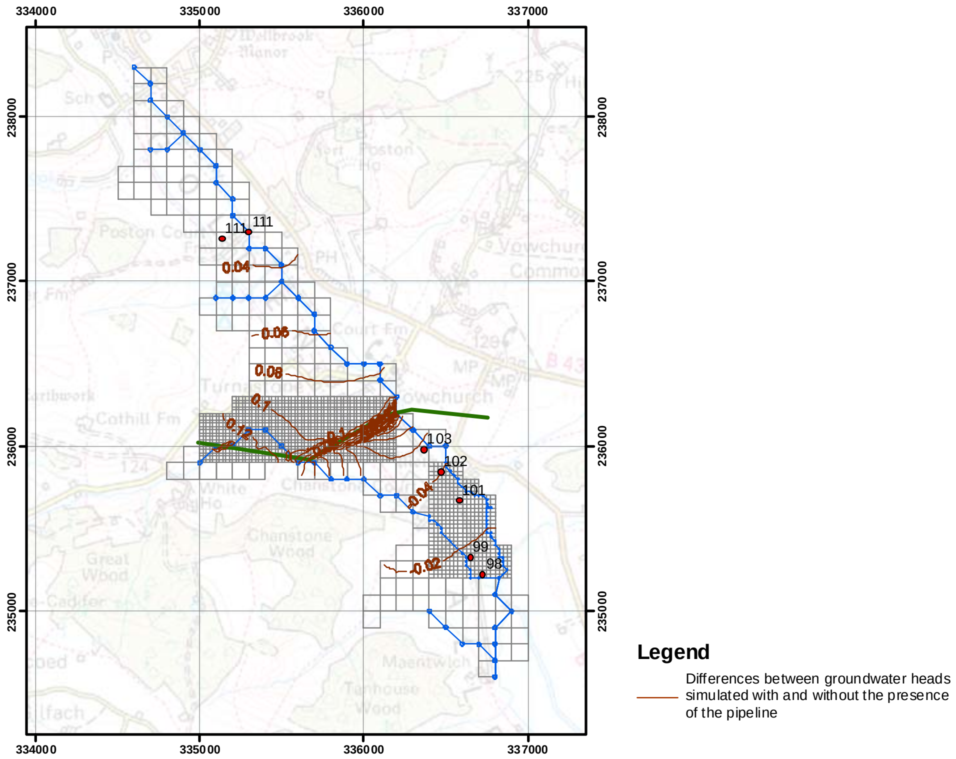


Figure 22. Contours of the differences between the simulated groundwater heads of the upper layer with and without the presence of the pipeline using a two-layer model

5.5 PIPELINE TRENCH CAUSING HIGH PERMEABILITY CONDUIT

The trench backfill material could conceivably be highly permeable and could create a diversionary high permeability zone along the pipeline which might change the groundwater flow system. A two-layer model was constructed with an upper layer saturated thickness of 2.0 m and a lower layer saturated thickness of 4.0 m to investigate this hypothesis. The high permeability zone is created by increasing the hydraulic conductivity values at the nodes located along the path of the pipeline threefold, from 250 m day^{-1} to 750 m day^{-1}. However, the results of the steady state simulation do not produce groundwater level contours that are significantly different from those produced without the high permeability zone. The contours of the differences between the simulated groundwater heads with and without this high permeability zone are shown in Figure 23 and indicate that a maximum drop in groundwater heads of approximately 0.12 m may occur just to the above the pipeline. This maximum drop in groundwater heads is observed next to the Slough Brook end of the trench and, as discussed previously, has no impact on the Slough Brook river flow since the groundwater heads are already below the streambed elevation. The groundwater head decrease is approximately 0.04 m on the River Dore side of the trench and this is expected to have minimal effects of the flows of this river.

5.6 DEWATERING THE PIPELINE TRENCH

The aim of this simulation was to estimate the pumping rate required to lower the groundwater heads along the path of the pipeline to a required elevation during pipeline construction, and to study the impact of dewatering on the groundwater flow system. Two lines of abstraction points parallel to and straddling the pipeline trench were built into the model. Although the numerical grid was refined so that the cell size was reduced to 20 m near the pipeline, the cell size is still too large to create a realistic cone of depression around each of the abstraction points. In addition the lines of abstraction points included in the model are approximately 45 m apart. In practice the distance between these lines will be much smaller since the trench width is only to be about 1.5 m. Consequently, the abstraction rates reported in this section must be treated with caution and are likely to be over-estimates.

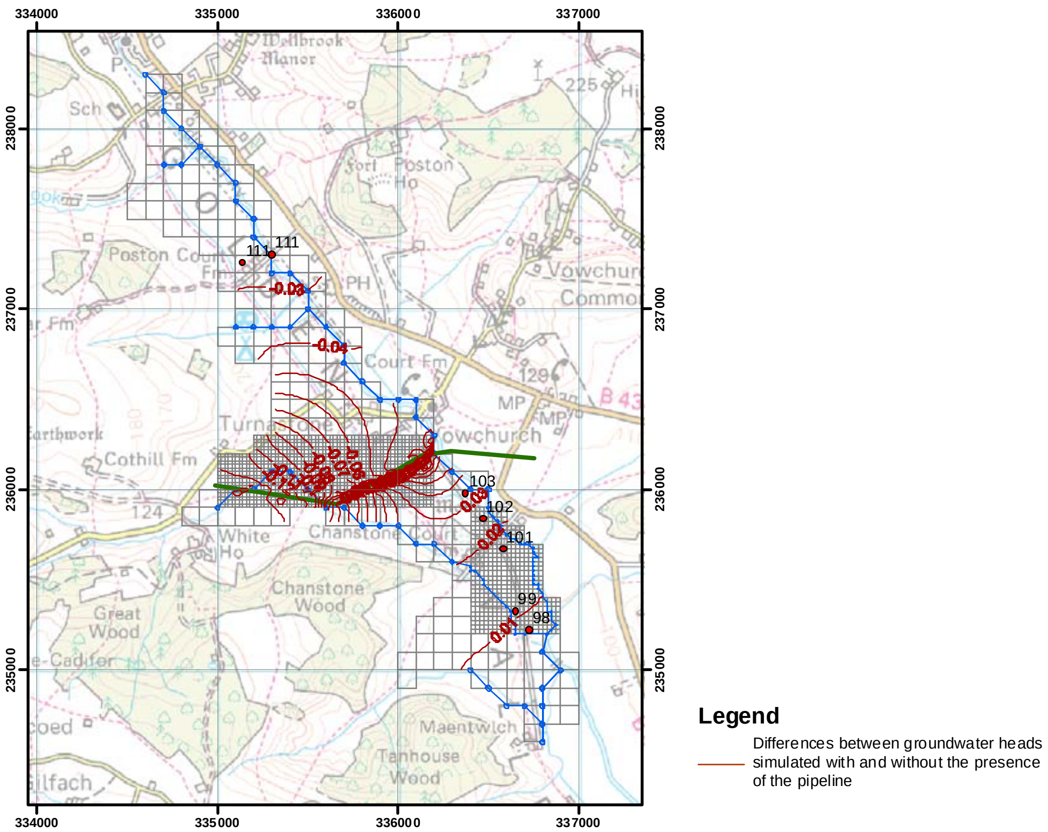


Figure 23. Contours of the differences between the simulated groundwater heads in the upper layer with and without the presence of the high permeability zone using a two-layer model

A total of 146 abstraction points were built into the model. When these were pumped at a rate of 20 $m^3 day^{-1}$ each, an average fall of groundwater heads of 1.7 m is observed along the path of the pipeline as shown in Figure 24. A maximum drop of groundwater head of 2.2 m is obtained at the Slough Brook end of the trench but this drop in groundwater head is reduced to 1.2 m at the River Dore end of the trench. However, this pumping scheme creates a significant groundwater head decrease elsewhere in the aquifer with values of 0.8 m and 0.3 m head decrease in the north and south of the aquifer respectively. The total abstraction rate specified in this simulation is 2920 $m^3 day^{-1}$.

Dewatering the 200 m section of the trench adjacent to the River Dore was also investigated as in reality the trench is above the water table in the remainder of the flood plain crossing to the west. However, instead of using two lines of abstraction points, abstraction is assumed to take place from twelve abstraction points that are located at the centre of the dewatered trench section. A pumping rate of 300 $m^3 day^{-1}$ is required at the abstraction points to lower the groundwater heads by 2.0 m as shown in Figure 25. The total abstraction rate specified in this simulation is 3600 $m^3 day^{-1}$. As with dewatering the whole trench, the pumping rate in this simulation also causes a significant groundwater head decrease of 0.7 m at the north end of the aquifer and of 0.5 m close to the public supply abstraction boreholes.

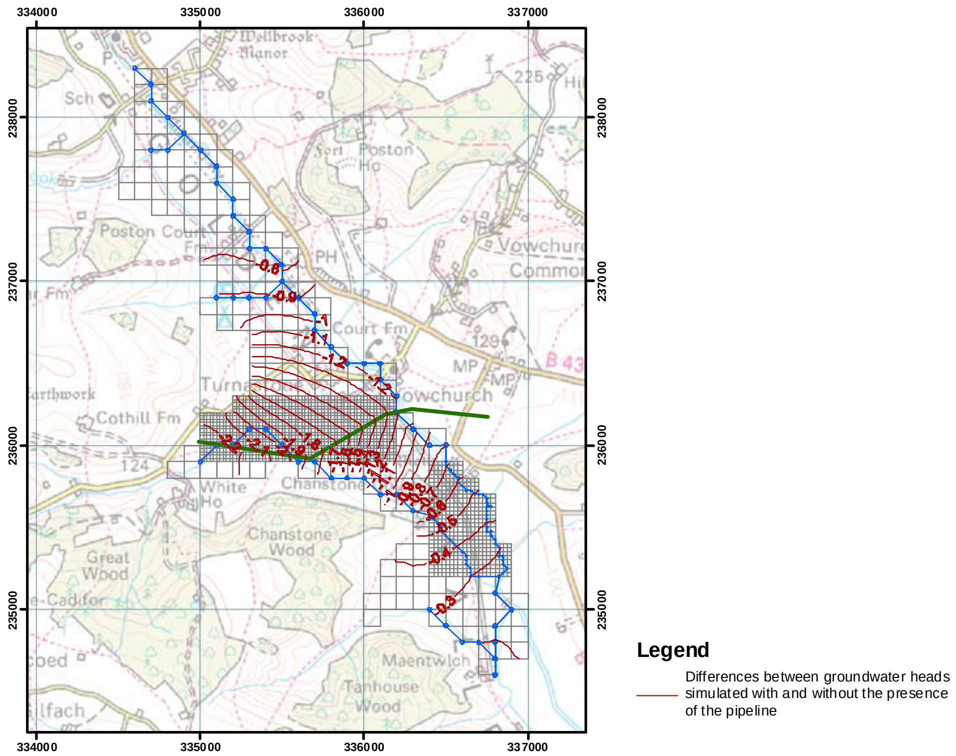


Figure 24. Contours of the groundwater head reduction caused by pumping to dewater the whole trench

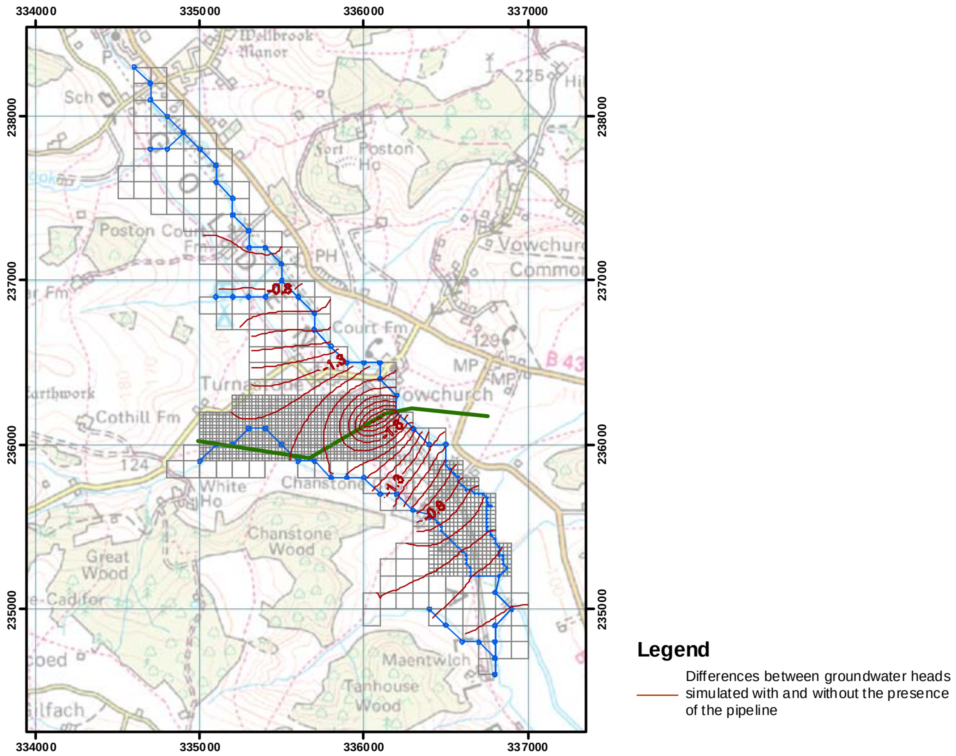
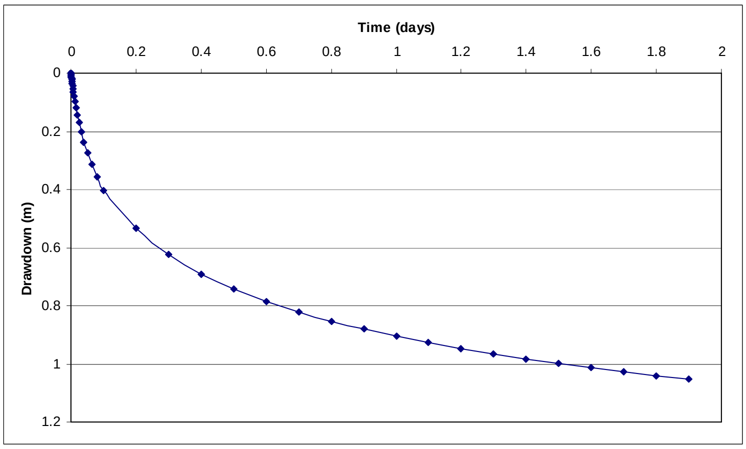
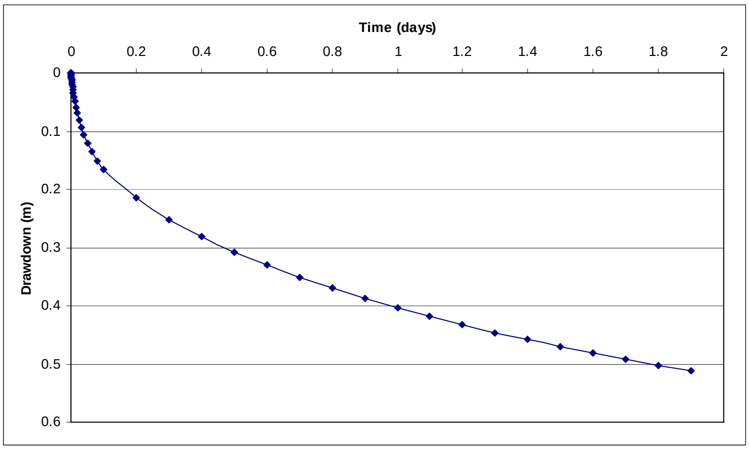


Figure 25. Contours of the groundwater head reduction caused by pumping to dewater the 200m length of trench immediately west of the Dore

The required dewatering rate and dewatering scheme design can be studied using the numerical radial flow model to simulate the fall of the water table. Complementary spreadsheet calculations can be undertaken to apply the method of superposition and to calculate the total drawdown values considering the selected design for the well point distribution. The water table movement is dependent on the hydraulic gradients established in the aquifer system as a result of pumping and on the specific yield of the aquifer. It was not, however, possible to estimate a value for the specific yield in the pumping test analyses since the aquifer was assumed to react under confined conditions. The water table movement is investigated, therefore, using two arbitrary, but feasible specific yield values of 5% and 15%. Figure 26a and Figure 26b show a plot of the total drawdown values versus time simulated at a point and caused by pumping 100 $m^3 day^{-1}$ from 40 well points. The results shown in these figures are simulated using a specific yield value of 5% and assume that there is one row of well points with distances of 5 m and 20 m between the well points respectively. The maximum water table fall after approximately two days calculated at the middle of this row is 1.2 m when the well points are separated by a distance of 5 m and 0.6 m when they are separated by a distance of 20 m. The maximum water table fall at the same location is 0.9 and 0.4 m when the well points are separated by distances of 5 m and 20 m respectively and when a specific yield of 15% is used in the calculations as shown in Figure 27a and Figure 27b.

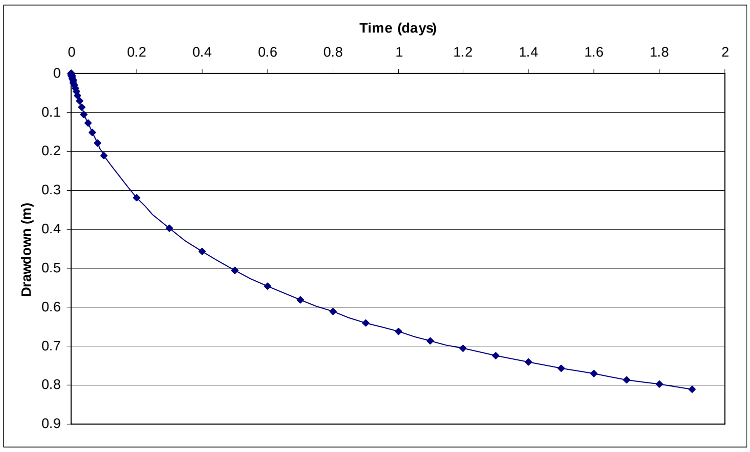


(a)

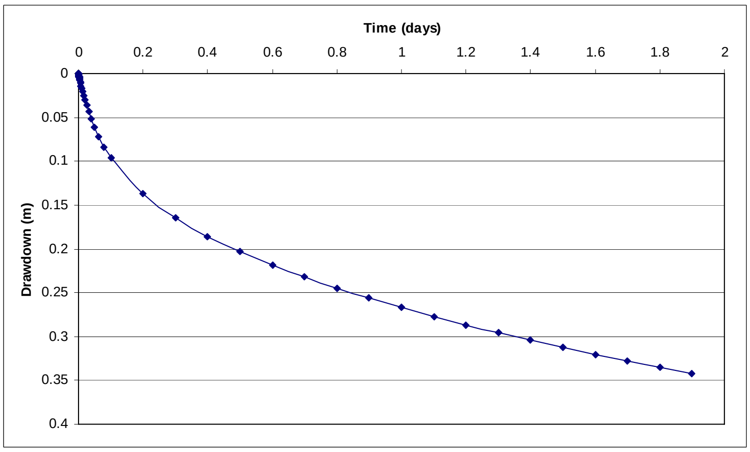


(b)

Figure 26. Total water table fall using 40 well points (a) 5 m apart and (b) 20 m apart with aquifer specific yield of 5%.



(a)



(b)

Figure 27. Total water table fall using 40 well points (a) 5 m apart and (b) 20 m apart with aquifer specific yield of 15%.

5.7 PARTICLE TRACKING. WELL CAPTURE ZONES.

A particle tracking numerical model ZOOPT (Jackson and Spink, 2004) was used to define the capture zones of the public abstraction boreholes after the construction of the pipeline. This model used the groundwater heads and the flow terms at the sides of the grid cells produced by ZOOMQ3D to study the movement of particles in the groundwater flow system. To define the well capture zones, particles were initiated around the abstraction boreholes and tracked backward until they reached a source of water such as rivers, springs or the water table.

The two-layer numerical model is used for this purpose. At each abstraction borehole 50 particles are positioned at equal spaces from each other at a 25 m diameter ring. Two simulations are undertaken. In the first the particles are positioned in the centre of the lower 4.0 m thick layer. The simulated borehole catchments for the three boreholes are shown in Figure 28a. In the second simulation the particles are positioned in the centre of the upper 2.0 m thick layer and the simulated borehole catchments for the three boreholes are shown in Figure 28b. Figure 28b shows that the particles move beneath the trench, represented as an impermeable barrier in the upper layer, and then head upwards and terminate at the water table. The capture zone of borehole D in Figure 28a ends at the Slough Brook.

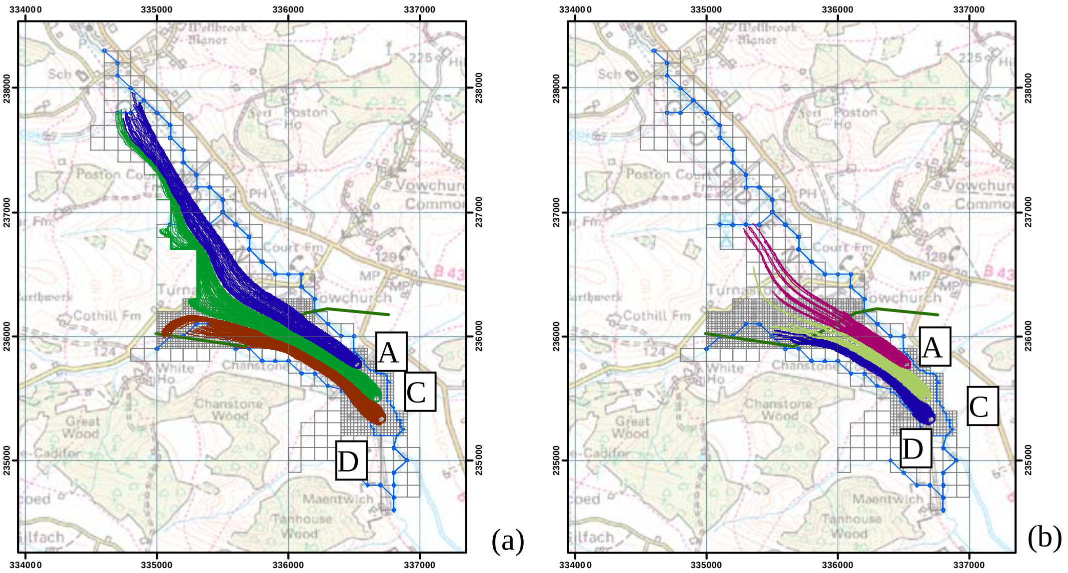


Figure 28. Abstraction borehole capture zones. (a) Backward tracking of particles positioned in the centre of the lower 4.0 m thick layer. (b) Backward tracking of particles positioned in the centre of the upper 2.0 m thick layer.

The travel time required by a particle located at the pipeline trench to reach the abstraction borehole can also be investigated using the ZOOPT numerical model. However, the velocity at which a particle moves is dependent on the hydraulic gradients, which are already determined by the groundwater model, and the porosity of the aquifer. A porosity of 25% was used in this simulation. 28 particles were released from the centre of the upper layer and along the trench of the pipeline. The tracks of these particles are plotted in Figure 29 which shows that the abstraction boreholes capture almost all the particles. The calculated travel time from the trench to abstraction borehole A is between 100 and 215 days, to abstraction borehole C is between 430 and 450 days and to abstraction borehole D is between 215 and 530 days.

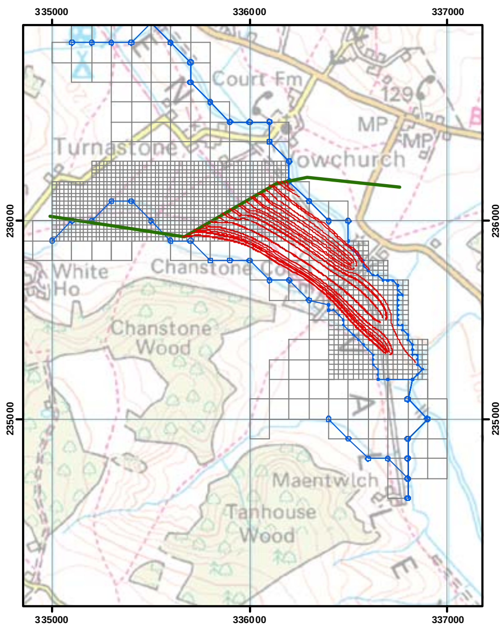


Figure 29. Tracking of particles positioned along the trench

6 Conclusions

The hydrogeological assessment of the impact of the pipeline during and after construction indicates that there will be minimal impact on the water table in the shallow gravel aquifer at Vowchurch. The pipeline trench will cut through overbank deposits and till and will not penetrate the coarser material of the gravel aquifer. During construction, the effect of drawing the water table down with a well point system to maintain a dry trench will cause a likely drop in the groundwater head at the Vowchurch pumping station of up to 0.5 m. However, this will only be for a brief period of several days and will have no longer term effect on the aquifer in this vicinity. The trench will only be sub-water table in the 200 m section adjacent to and to the west of the River Dore. Any turbidity created by trenching is likely to be lost in the aquifer as the travel time from the trench area to the pumping station is at least 100 days.

The impact of the completed pipeline is minimal. The damming effect of the pipeline itself on the water table is very small, and the effect of varying the permeability of the backfill material is equally small. Although the relationship with the River Dore will be very slightly modified, that with the Slough Brook remains unchanged as it is perched in the vicinity of the pipeline.

The river crossing itself will be through overbank silts and, unlike a trenchless crossing, will neither penetrate the till nor the gravel aquifer. The placement of this section of the pipeline should, therefore, little effect the groundwater heads in the gravel aquifer.

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Most of the references listed below are held in the Library of the British Geological Survey at Keyworth, Nottingham. Copies of the references may be purchased from the Library subject to the current copyright legislation.

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