



6. Qualitative Assessment of Inflows

6.1 General

Impacts to groundwater from an environmental or construction perspective can generally be mitigated or eliminated by engineering controls. There are, however, factors that may influence or constrain the final pipeline route selected and the location of associated infrastructure, such as the intake structure / pump station. For example, constructing pipeline through bedrock may have lower groundwater inflows, and thus lower dewatering costs, but excavation costs may be higher.

The following section discuss the anticipated groundwater management requirements for the various sections of the pipeline corridor. The methodology for the qualitative assessment of groundwater inflows involved the following steps:

- Hydrogeological environment mapping
- Calculation of construction impacts in terms of:
 - Trench inflow
 - Intake structure inflows
 - Tunnel inflow
 - Radius of influence of impacts
- Qualitative assessment of groundwater constraints.

The discussion includes an analysis which was undertaken using existing published information, information obtained from site inspections and information based on hydrogeological experience in similar geologic terrains. The analytically derived inflows were based on simplified versions of natural, often complex, groundwater flow systems. Little quantitative hydrogeological data was available, consequently the analysis was not based on any site-specific data and all results should be considered preliminary, subject to review and confirmation following on-ground works.

In each case, it has been assumed that long term impacts (identified in Section 6) were eliminated by placing trench cut-offs, and appropriate backfilling and reinstatement works.

6.2 Mapping of Hydrogeological Environments

As a first step in the assessment process, the study corridor was divided into sub areas so that hydrogeological parameters could be assigned accordingly. The subdivision of areas was undertaken in two stages and was based as follows:



Stage 1

- Division of the corridors into the three conceptual hydrogeological models, as described previously, based on geological mapping, site inspection and aerial photograph interpretation, in order to identify those areas likely to:
 - Be in greater hydraulic connection with surface water systems
 - Have differing (broad scale) hydraulic properties and thus different dewatering requirements.

Stage 2

- Overlaying of the depth to water table mapping. Again, a broad scale definition was applied to identify those areas with water levels expected to be:
 - Less than 5m, and thus more likely to require some dewatering effort
 - Greater than 5m below the natural surface, and thus less likely to require significant dewatering effort.

The objective of this stage was to identify, on a broad scale, where dewatering may or may not be required and where impacts to surface water systems and GDE may occur.

The following should be noted:

- The differentiation of the hydrogeological model boundaries was based on broad scale (1:50,000 scale) mapping or greater. Whilst in some areas this may be accurate, the reliability may become questionable near the boundaries of differing geological formations; specifically, the contact between the alluvial floodplain sediments and the basement rocks. In such cases, the boundaries were placed conservatively i.e. the boundary of alluvial sediments were placed further from the river compared to that shown on the published mapping. By erring conservatively, groundwater inflows would be likely to be slightly overestimated, nevertheless for the purposes of this preliminary assessment, this approach was considered to be acceptable. On-ground works are required to accurately identify such boundaries
- The depth to water table information was not based on actual groundwater data because of the dearth of information. There is no understanding of the accuracy of the derived map, or the seasonal behaviour of the water table
- Where surface water systems were identified during site inspections, it was assumed that these were groundwater fed and therefore indicative of the depth to water table at these locations
- The differentiation of the basement aquifer and regolith aquifer hydrogeological models cannot be completed without specific intrusive investigations. In most cases the bedrock was conservatively rated as a regolith model, in which groundwater inflows would be expected to be shallower (refer below). The bedrock model was applied where obvious bedrock was mapped or observed during site inspections.



6.3 Construction Inflow Calculations

6.3.1 Methodology

A semi-quantitative analysis of groundwater inflows into various elements of construction works was undertaken. Analytical models were used to estimate the inflows per unit of length. Three aspects of the study corridors were selected, and for each aspect a number of different scenarios were examined. The three aspects were:

- Pipeline:
 - Alluvial / colluvial type hydrogeological environment (conceptual hydrogeological model 3)
 - Bedrock aquifer (conceptual hydrogeological model 1). Note that although three hydrogeological models were conceptualised, differentiation of the bedrock and regolith aquifers cannot be undertaken without intrusive subsurface investigations.
- Goulburn River Intake structure:
 - This was essentially modelled as a deeper, larger version of pipeline trench.
- Tunnel:
 - Bedrock aquifer (conceptual hydrogeological model 1).

The analytical methods used and the results of the inflow calculations have been documented below.

6.3.2 Pipeline Trench Inflow

Analytical Methodology

Simple gravity flow under unconfined conditions into a trench is approximated using the following flow equation:

$$Q_p = \frac{k}{2L} (H^2 - h_o^2) \text{ (fully penetrating) or } Q_p = \left(0.73 + 0.27 \frac{H - h_o}{H} \right) \frac{k_s}{2L} (H^2 - h_o^2) \text{ (partially penetrating)}$$

where:

Q_p = discharge in m³/day per linear metre;

H = head at recharge source (m);

h_o = head in slot above aquifer base (m); and,

L = radius of influence (m), and;



K = hydraulic conductivity (m/day).

Assumptions

The following assumptions were applied:

- Aquifer conditions:
 - The trench was excavated in unconfined conditions i.e. where the aquifer is exposed to the atmosphere
 - Groundwater entered the trench by gravity flow only
 - The aquifer was a single layer, homogeneous system. Note: in reality, conditions may differ significantly within a saturated alluvial aquifer profile with the presence of alternating sand, gravel and clay layers.
- Trench geometry and construction:
 - The excavation face was vertical
 - The excavation was established instantaneously
 - The excavation was long and linear
 - The trench fully penetrated the aquifer.

Inputs

The analytical model relied on a number of input parameters. Actual values for these inputs are currently unknown and have to be derived from on-site investigations e.g. slug testing, pumping tests, lithological logging. General aquifer hydraulic parameters have been previously cited (refer to previous sections) and therefore a range was applied to identify a worst case (high water table, highly permeable aquifer) and best case (low water table, low permeability aquifer) trenching condition with respect to inflows.

A hydraulic conductivity range of 0.001 to 10 m/day was applied to bedrock conditions, assuming that bedrock was likely to be weathered. A range of 0.001 to 100 m/day was applied for the alluvial sediments.

As information regarding the depth to water is unknown, the saturated thickness adopted was based on a maximum trench excavation depth of 5m. Under this condition, a head differential relative to the base of the trench would have to be between 0m (water level at base of trench) and 5 m (water at ground surface). The thickness of the alluvial aquifer is unknown and therefore it is uncertain whether the trench will be fully or partially penetrating.

The radius of influence of dewatering is not known. A range of 20m to 200m was therefore applied so that an approximation could be obtained.

Results

The results of the analyses are summarised in Table 8, for the two radii of influence and two head differentials of 1 m and 5 m, assuming fully penetrating conditions. The results for partially penetrating flow are shown in Table 9.

Table 8 - Estimated Inflows into Trench (m³/day per linear metre of trench) – Fully Penetrating Conditions

Aquifer	Lithology	Hydraulic Conductivity (m/day)	Differential Head = 1m		Differential Head = 5m	
			L= 20m	L = 200m	L= 20m	L = 200m
Alluvial	Clay, sand, silt mix	0.001	-	-	-	-
	Sand and gravel mix	1	0.1	-	1.3	0.1
	Gravel	100	5	0.5	125	12.5
Bedrock	Tight	0.0001	-	-	-	-
	Fractured	10	0.5	0.1	12.5	1.3

Note:

1. In the fully penetrating case, a differential head of 5m is the worst case i.e. a 5m deep trench fully excavated in an aquifer with 5m saturated thickness, required water level drawdown of 5m to the base of the trench.
2. Differential head is achieved at the radius of influence.

Table 9 Estimated Inflows into Trench (m³/day per linear metre of trench) – Partially Penetrating Conditions

Aquifer	Lithology	Hydraulic Conductivity (m/day)	Differential Head = 1m		Differential Head = 5m	
			L= 20m	L = 200m	L= 20m	L = 200m
Alluvial	Clay, sand, silt mix	0.001	-	-	-	-
	Sand and gravel mix	1	1.3	0.1	1.3	0.1
	Gravel	100	125	12.5	125	12.5
Bedrock	Tight	0.0001	-	-	-	-
	Fractured	10	12.5	1.3	12.5	1.3

6.3.3 Intake Structure (Pump Station)

Analytical Methodology

A similar approach was adopted to estimate likely inflow volumes during construction of the low lift pump station, given the nature of the alluvium and the expected hydraulic connection between the pump station excavation and the Goulburn River.

At the low lift pump station, either the bedrock or the alluvial aquifer system will be excavated. Conceptually, the Goulburn River was simplified as a linear recharge boundary, with the intake structure partially penetrating the aquifer. Inflows to the low lift pump station excavation were then approximated using the above equation for trench inflows.



Assumptions

The analytical assumptions were similar to those of the pipe trench inflow. The analytical equation was based on partially penetrating conditions i.e. the intake structure excavation would not intersect the full thickness of the aquifer.

It was further assumed that the intake structure could be modelled as slot parallel to the Goulburn River.

No allowance for gross head changes in the Goulburn River, which operates as an irrigation supply, were made.

Inputs

Two possible conditions were anticipated for the intake structure. The structure could be excavated directly into the bedrock materials e.g. this would occur east of Killingworth Road, or it may be excavated directly into the alluvial sediments.

A conductivity range of 0.001 to 10m/day was applied to bedrock conditions, assuming that bedrock was weathered by alluvial processes. A range of 0.001 to 100 m/day was applied for the alluvial sediments.

It was assumed that the Goulburn River would recharge any adjacent aquifers. The radius of influence was therefore expected to be the separation distance between the river and the intake structure. A minimum of 10m separation distance was applied.

A maximum head at the recharge source (Goulburn River) of 20m and a minimum head above aquifer base of 10m were applied.

Results

The results of the analyses are summarised in Table 10, for two separation distances of 10m and 50m, and two head differentials of 10m and 20m. Results indicated that some significant dewatering effort would be required if highly permeable gravels were intersected at the intake structure.

■ **Table 10 - Estimated Inflows into Intake Structure (m³/day)**

Aquifer	Lithology	Hydraulic Conductivity (m/day)	Differential Head = 10m		Differential Head = 20m	
			L = 10m	L = 50m	L = 10m	L = 50m
Alluvial	Clay, sand, silt mix	0.001	-	-	-	-
	Sand and gravel mix	1	13	2.6	20	4



			Differential Head = 10m		Differential Head = 20m	
	Gravel	100	1,300	260	2,000	400
Bedrock	Massive	0.0001	-	-	-	-
	Fractured	10	130	26	200	40

Note: Inflow calculated from Goulburn River only. As rough approximation flow rates should be doubled to account for contribution for aquifers on other side of trench away from river.

6.3.4 Tunnel Inflow

Analytical Methodology

In the bedrock areas of the study corridor, particularly near the Toolangi State Forest, the pipeline may be constructed through tunnelling. Tunnel inflow volumes were determined using a steady state analytical calculation developed by Goodman *et al* (1965). The approach is outlined as follows:

$$Q_p = \frac{2\pi kH}{2.3 \text{Log} \frac{h}{2r}}$$

where:

Q_p = Rate of groundwater inflow

H = Depth from watertable to the tunnel mid-point

h = depth to water

K = hydraulic conductivity

r = Tunnel radius.

Assumptions

The approach is based upon a number of assumptions which may be more or less valid for the different hydrogeological models; specifically, the assumption that the aquifer is isotropic and homogeneous is less valid for the alluvial aquifer and consequently the approach may over-estimate groundwater inflows for this aquifer. Moreover, in the absence of field data, specifically water levels and aquifer hydraulics for the different aquifer types, input parameters were derived from published literature.

It was further assumed that with tunnelling required to cross surface water systems or shallow alluvial sequences, appropriate water ingress control could be achieved through the tunnelling method adopted.



Inputs

A conductivity range of 0.001 to 10 m/day was applied to bedrock conditions. The depth of tunnelling was unknown and was assumed to be 50m. The depth to water was also unknown and therefore a range was adopted. A tunnel radius of 2m was adopted.

Results

The results of the tunnel inflow calculations have been presented in Table 11.

■ **Table 11 - Estimated Inflows into Tunnel (m³/day/linear metre of tunnel)**

Aquifer	Lithology	Hydraulic Conductivity (m/day)	Standing Water Level			
			5m	10m	20m	30m
Bedrock	Tight	0.001	-	-	-	-
	Weathered / fractured	0.1	1.22	1.3	1.6	1.8
	Fractured / shear zone	10	122.25	133.37	155	178

6.4 Radius of Influence Calculations

Analytical Methodology

An understanding of the radius of influence is required with any dewatering (or construction water supply) assessment because it can be used to identify the impacts of pumping. In general terms, maximising the distance between dewatering and the impacted feature is required to minimise impacts.

The radius of influence is dependent upon the aquifer hydraulics and the duration of pumping. For the purposes of this analysis the radius of influence has been estimated from the Thies non-equilibrium well equation:

$$h_o - H = \frac{Q}{4\pi T} \left[-0.5772 - \ln u + u - \frac{u^2}{2.2!} + \frac{u^3}{3.3!} - \dots \dots \dots \frac{u^n}{n.n!} \right] \text{ and } u = \frac{r^2 S}{4Tt} \text{ where:}$$

Q = pumping rate

H = hydraulic head at time t since pumping began

h_o = the hydraulic head at the start of pumping

r = radial distance from the pumping well to the observation well

T = aquifer transmissivity



t = time

S = aquifer storativity.

Assumptions

The non-equilibrium equation relies on a number of assumptions:

- Discharge from pumping is instantaneous with decline in pressure
- The excavation fully penetrates the aquifer
- Well storage is assumed negligible
- Flow to the well screen is radial, horizontal and laminar
- The aquifer is homogeneous and isotropic
- Aquifer thickness is uniform
- The aquifer remains saturated during pumping
- The aquifer is infinite in aerial extent and bounded horizontally above and below by impermeable beds
- All water is derived from the cone of depression in the watertable.

Inputs

A conductivity range of 0.01 to 10m/day was applied to bedrock conditions, assuming that bedrock was likely to have been weathered. A range of 0.001 to 100m/day was applied for the alluvial sediments. These conductivities were converted into a transmissivity assuming a saturated thickness of 20m for the alluvial sediments, and 100m in the bedrock. An aquifer storage coefficient of 0.1 was assumed for unconfined conditions of the alluvial sediments, and 0.001 in the bedrock.

The radius of influence is a time-dependent parameter. Generally the draw downs are rapid, but decline over time as the radius of influence expands to deliver water. A maximum pumping duration of 14 days was applied on the assumption that this reflected the longest period at a location where pumping would be required. Shorter pumping periods would result in a reduced radius of influence.

Results

The results of the calculations for bedrock conditions, for a pumping rate of 0.5ML/day (6 L/s) suggested that the radius of influence could be tight and less than 350m after 14 days of pumping in unfractured bedrock ($K = 0.01$ m/day), or considerably broader and over 800m in fractured bedrock ($K = 10$ m/day).



In terms of the alluvial aquifers, the storage coefficient was expected to be greater than that of the bedrock and therefore the radius of influence reduced. At storativities of 0.1, and pumping rates of 1ML/day, the radius of influence was less than 300m.

6.5 Summary

The above approaches were used in the absence of any site-specific data on hydrogeological conditions (depth to water table, aquifer hydraulics, etc). This notwithstanding, the methods were considered to provide an adequate first order approximation of groundwater inflow volumes for developing pipeline dewatering methods and groundwater disposal options for pipeline construction.

Installation of piezometers and aquifer hydraulic tests scheduled for the forthcoming months will provide field data which will be used to refine groundwater inflow estimates, and along with water level measurements, will assist in delineating those areas where dewatering may be required. A work program has been presented in the next section to acquire and fill a number of the key hydrogeological data gaps that have the strongest bearing on the inflow estimates provided above.