Appendix E

The Non-potable Master Plan Model

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Appendix E The Non-potable Master Plan Model

1.1 The non-potable master plan model

This appendix documents the spread sheet model developed to simulate the potential non-potable water supply network for the ACT. A set of instructions to help a user get started are provided in Section 1.2. More detail regarding model inputs is provided in Section 1.3, constants in Section 1.3.4 and model calculations in Section 1.4. The calibration of the hydrologic and cost components of the model to detailed schemes and observed data is summarised in Section 1.5.

The purpose of the model is to allow a range of different scenarios including different sources and supply networks to be investigated and the costs evaluated to determine the preferred scenario for supply of alternative water sources to meet a range of demands. The model allows a range of sources and demands to be represented as well as the supply pipe network. Infrastructure sizing requirements and costs are provided as input or estimated by the model for storages, pipes, pumps, balancing storages and treatment.

Sources and demands are represented as nodes in the system. The model allows different non-potable sources to be represented, including pond and lake storages, sewage recycling, aquifer storage, transfer and recovery and groundwater. Each source can supply multiple demands up to its maximum capacity. Each source may potentially have multiple outlets to allow representation of pump outlets from a large lake, several ponds in series or a sewage recycling transfer network. The pipe network is represented by links connecting each of the nodes.

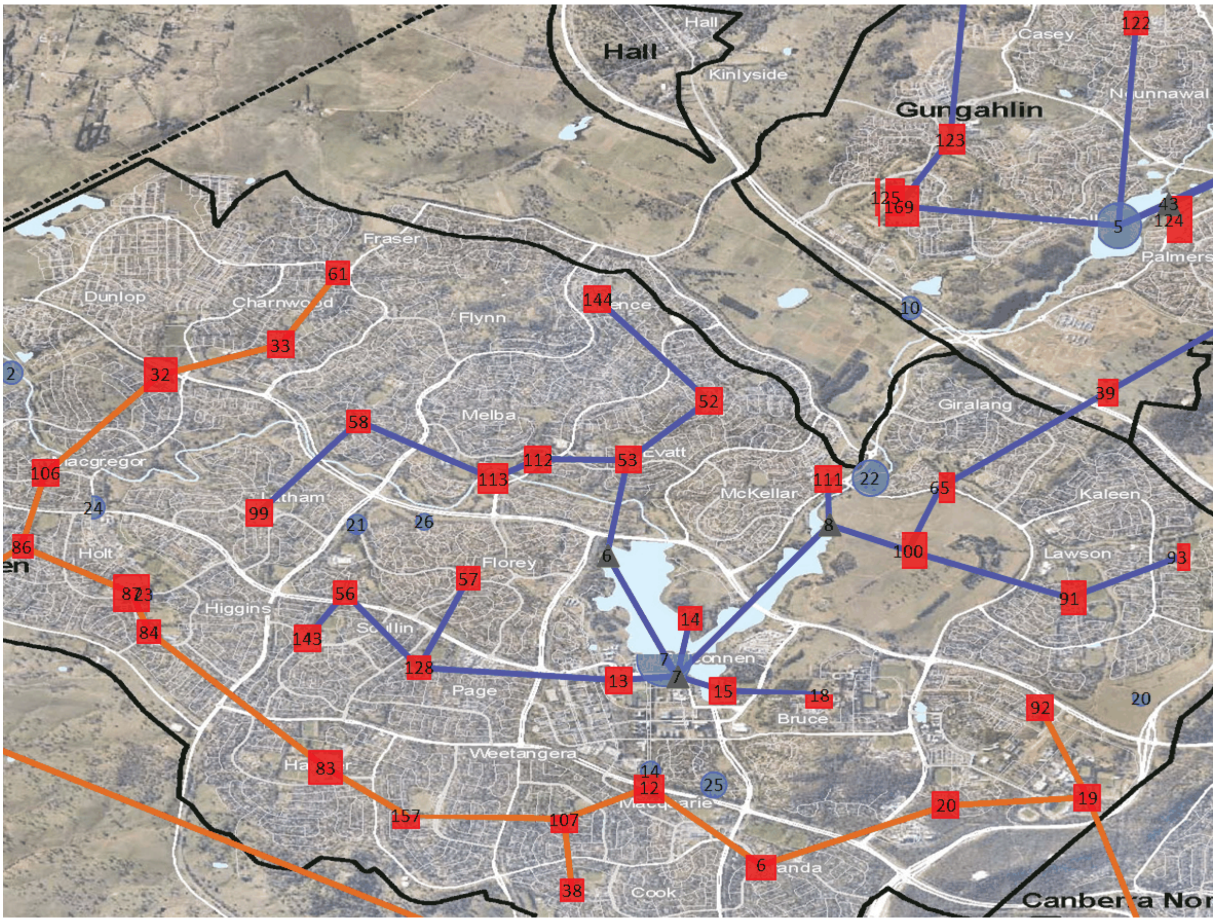


Figure 1 A typical non-potable supply network

1.1.1 Assumptions

The purpose of the model is for master planning of non-potable water supplies. It is a broad conceptual water balance and costing model and uses a range of simplifying assumptions:

- Sources and catchments are independent
- Demands are either irrigation demands with seasonal patterns or constant demands
- A source may supply multiple demands
- A demand can be supplied by only one source
- Only impervious area runoff from catchments or ‘stormwater excess’ is considered and all catchments are assumed to exhibit similar behaviour with flows dominated by impervious flows
- A separate daily spreadsheet based pond model was used to develop lookup tables for estimating reliability based on climate data, catchment flow, storage size and demand. These inputs can be provided by any suitable external model and data for Canberra is included with the model
- Costs are based on information supplied by the client or obtained or calibrated from detailed studies of proposed stormwater supply systems for Lake Ginninderra and Lake Tuggeranong.

1.1.2 Software and hardware requirements

- The model is run as an Excel spreadsheet. Excel 2007 and Windows XP or later is required to run the model.
- The model has not been tested using Windows Vista, Windows 7 or any other version of Excel.
- It is recommended that a large display or preferably two displays are used for effective use of the model.

1.1.3 Program structure

The program is structured as a series of sheets within a single spreadsheet. The spreadsheet comprises several main components.

- User interface
- Map
- Data
- Model constants
- Calculations

User interface

The user interface consists of a dashboard allowing the user to modify the model scenario and provide a summary of key information. The dashboard provides information for a selected source, a graph illustrating the available and allocated supply and a summary of costs. Costs for the master plan scenario being modelled are also provided. Functionality is provided to allow the user to add or modify nodes including sources, demands and special nodes as required. Once nodes have been defined, the user can connect the pipe network to assign various demands to sources or other nodes and progressively construct the pipe network.

Map

The map provides visualisation of the proposed scheme. Nodes are plotted based on the coordinates provided. This is overlaid on an aerial photograph to provide context. The user can navigate around the map using the scroll bars and zoom using the Control key (Ctrl) and the scroll wheel. The current district being viewed in the map can be changed using the user interface.

Data input

All base data for the model are input through forms accessed from the user interface and stored within the model. The following sets of data are required:

- Sources
- Demands
- Special

Data sheets for each of these hold all the background data required to describe all of the various sources, demands and other nodes used to represent the network. Each node may be a source (pond, recycled sewage, ASR or groundwater), a demand, or a special node (junction or source outlet).

Model constants

The model constants sheet contains all the constants and assumptions used by the model. This includes:

- Constants and assumptions used to estimate infrastructure requirements such as friction and length factors
- Cost constants for costing equations
- Assumptions such as catchment flows per ha, irrigation demands, days of balancing storage required and irrigation demands

A schedule of pipes and associated costs is contained in the ‘PipeList’ sheet.

Calculations

Calculations are carried out on a number of sheets that sit behind the user interface. Calculations are performed using a combination of Excel equations and a series of customised algorithms for the tracing of the pipe network and interpolation of reliability curves. These are computed by clicking on the ‘Calculate demands for pipes’ button in the user interface.

1.2 User instructions

There are two stages to building a model:

1) Compile input data and assumptions
2) Create or modify a master plan by developing a non-potable supply network

It is generally expected that the user will be updating the existing master plan by modifying nodes and the network arrangement (stage 2), however it is also possible to create a new scenario if required.

1.2.1 Stage 1 – Set up scenario

The first step is to enter a set of input data into a copy of the ‘Scenario template’ describing all of the potential sources of non-potable water, the potential demand sites and the locations of additional storage outlets or junctions along existing pipelines.

To do this, first save a copy of ‘ScenarioTemplate.xlsx’.

Scenario Template

This is a scenario template for creating new scenarios for the Non potable m

To use it, save a copy of the template, enter the desired scenario data, save
the model to load the new scenario.

Input data for each node type is entered into:

1. **Sources**

2. **Demands**

3. **Special**

Each of these sheets must have a minimum of two nodes. Dummy nodes wi

Pipes contains a schedule of all pipe connections. For a new scenario it can l
from the model, this sheet will be populated.

Figure 2 Scenario Template information page

Inputs are provided for the following data sets:

- Source nodes
- Demand nodes
- Special nodes

Sources

Sources may be stormwater ponds, groundwater bores or a sewage treatment plant supplying treated effluent.

Sources require information to describe the source, identify its location and quantify the catchment and storage volume and area available (for pond storages).

User input

To add a source node, add details for user input (yellow columns) to next row then click

Source No	Source Name	Source type	Source status	District	Scheme	Easting	Northing	Catchment area (ha)	t impervious fraction	inflow or supply (kL) (if known, else 0)	Potential ASR inflow (kL)	Pond FSL area (m^2)	Pond drawdown depth (m)	Pond FSL volume (kL)	Dead storage area (m^2)	Dead storage volume (kL)	Sewage effluent costs: Unit cost of sewage effluent
1	David St Wetland	Pond	0	CanberraCentral	None	210,301	605,838	309	15%	229,700		3,025	0.75	3,025	1,513	756	$ -
2	Jarramlee (Dunlop 1)	Pond	0	Belconnen	None	200,261	612,505	79	30%	108,500		6,985	1.75	13,970	1,746	1,750	$ -
3	Fassifern (Dunlop 2)	Pond	0	Belconnen	None	199,948	613,138	35	33%	52,400		6,955	1.92	13,910	1,740	580	$ -

Demands

Demands are sites with an identified demand for water. The user can provide an area to be irrigated and also an area of development for which it is assumed water will be supplied to households for toilet flushing (constant demand) and irrigation use (seasonal demand). The base assumptions for these including irrigation rates and toilet flushing use are contained in the constants sheet and are applied globally. They can be modified by the user as needed.

Demands require information to describe the demand, identify its location and quantify the demand based on the area to be irrigated or developed.

Input Demand Data

This sheet contains all the demands. All fields must be completed except:
Either public or development area or both can be defined
ASR inflow and comments are optional

Demand Cluster No.	Demand Cluster Name	Easting	Northing	Demand (Existing/Current/ Future)	Public irrigated area (ha)	Development area (ha)	ASR inflow demand (kL)	Comments / Suggested Supply Source	Scheme
1	Acton	210,605	603,370	Current	0.2				Fyshwick A
2	ANU	210,118	604,341	Current	2.1				Fyshwick A

Special nodes

Special nodes may be either storage outlets or junctions.

Storage outlets are used to identify locations where storages have additional outlets. They may represent outlet locations on different sides of a large storage such as a lake, or the location of an upstream or downstream storage where two storages in series have been aggregated together as a single source for modelling. The pipe connection from a storage to a storage outlet is assumed to have no cost.

Junctions can be used to identify junctions, bends and off-take points along supply lines. Demands can be connected to a supply pipeline at any of these junctions to reduce the connection pipe lengths required.

Treated sewage effluent supply lines can be represented by using a series of junction nodes. The inputs for these are entered into the 'Special' sheet. The user will need to assign the connectivity of these nodes by setting the upstream node for each of the nodes in turn to create the sewage effluent supply line.

Junctions can also be used for supply lines from ponds to other demands if desired.

Detailed information about inputs is contained in Chapter 1.3.

User input						
Special No	Easting	Northing	Node type (SourceOutlet or Junction)	Corresponding node type	Corresponding node user no	Scheme
1	202,602	599,041	SourceOutlet	Source	65	None
2	202,127	599,757	SourceOutlet	Source	65	None
3	203,190	599,563	SourceOutlet	Source	65	None
4	202,219	600,703	SourceOutlet	Source	66	None
5	201,856	600,273	SourceOutlet	Source	66	None
6	204,966	610,403	Junction	Source	7	Lake Ginninderra B
7	205,516	609,003	Junction	Source	7	Lake Ginninderra B

Constants

There are a number of assumed constants used in the model calculations for infrastructure sizing and costing. These can be modified by the user by adjusting them in the constants sheet for the scenario, or within the model itself for sensitivity analysis.

Staging

Staging was incorporated into the model to allow proposed schemes to be summarised by stage. Each scheme to be constructed must be defined in the Staging sheet with corresponding data for year of completion and stage number. Sources and demands can then be allocated to a scheme and corresponding stage as required.

Once the user has finished entering the input data, they can save the new scenario and close it.

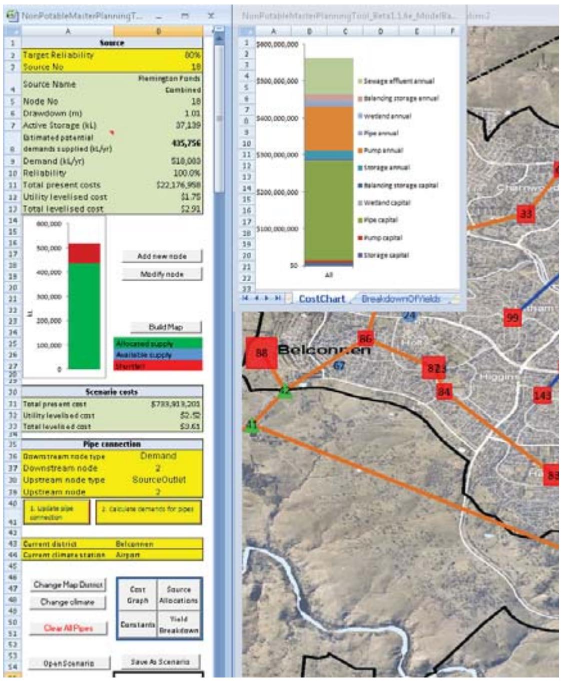
1.2.2 Stage 2 – Developing a pipe network scenario

When the model opens, it should open the user interface and map window by default.

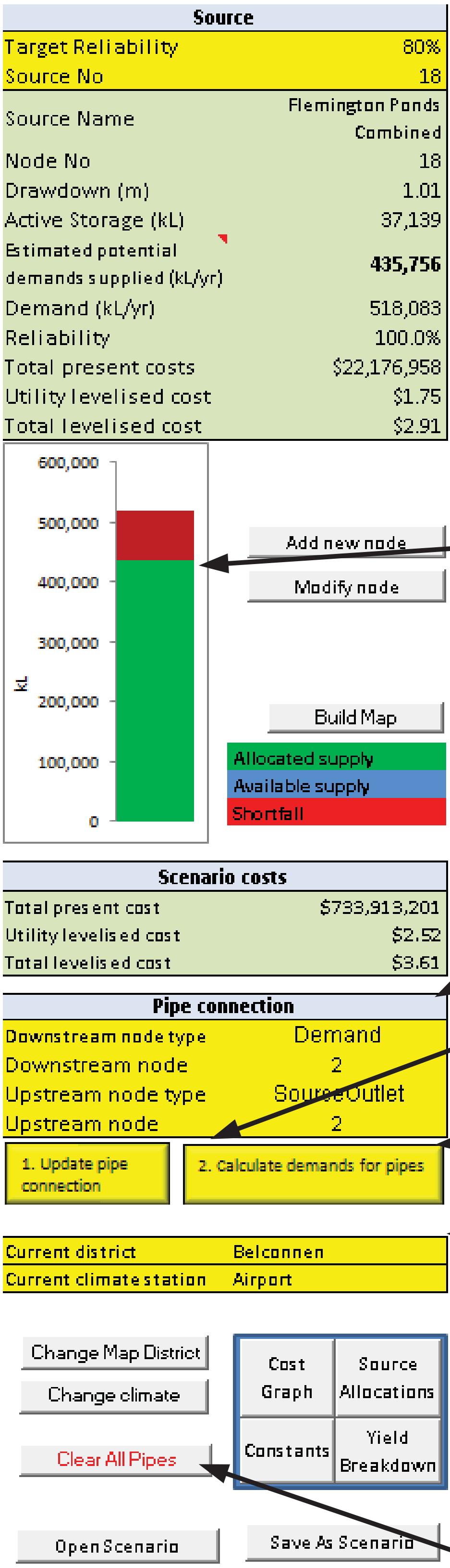
The model maintains a number of Excel windows at any one time. A range of graphs can be brought forward by clicking on them in the blue box in the user interface.

The user may find the *SourceAllocationGraph* and *Summary* useful for live updating of storage availability and cost estimates.

The user can also rearrange the windows and access other sheets as required.



The first step is to open a scenario in the model (Open Scenario) from User Interface.



A global target reliability (%) can be set for the model. Typically it will be in the range of 70-95%

Select a source to work with using the *'Source No'* drop-down box. The selected source will then be summarised and graphed below.

The model will look-up the potential supply the storage is able to provide, taking into consideration the target reliability, catchment flows and storage size.

The source status graph indicates the available supply for the source, the supply that has already been allocated to demands and identifies any shortfall if demands exceed available supply.

The user allocates demands to each of the storages by connecting demands to sources or other connected nodes. To do this, select the Demand or downstream node type and number. Click *'Update Pipe Connection'* to make each connection.

The user should then click *'Calculate demands for pipes'* to re-run all calculations to update the results.

The user can use the map to help guide their decisions. The area shown can be changed by selecting a different district from the *'Current District'* drop-down box then by clicking *'Change Map District.'* Note that the corresponding background image will need to be loaded as well.

The window buttons can be used to display different graphs and tables.

The user can *'Clear All Pipes'* to create a new blank network. Be careful to save before clicking this.

Figure 3 User interface

Source allocation graph

The source allocation graph provides a visual illustration of the sources and the allocations to each. The user can see the status of all sources on the SourceAllocationGraph, which is similar to the Source Status Graph in the dashboard but updates to reflect changes in the pipe network for all sources simultaneously.

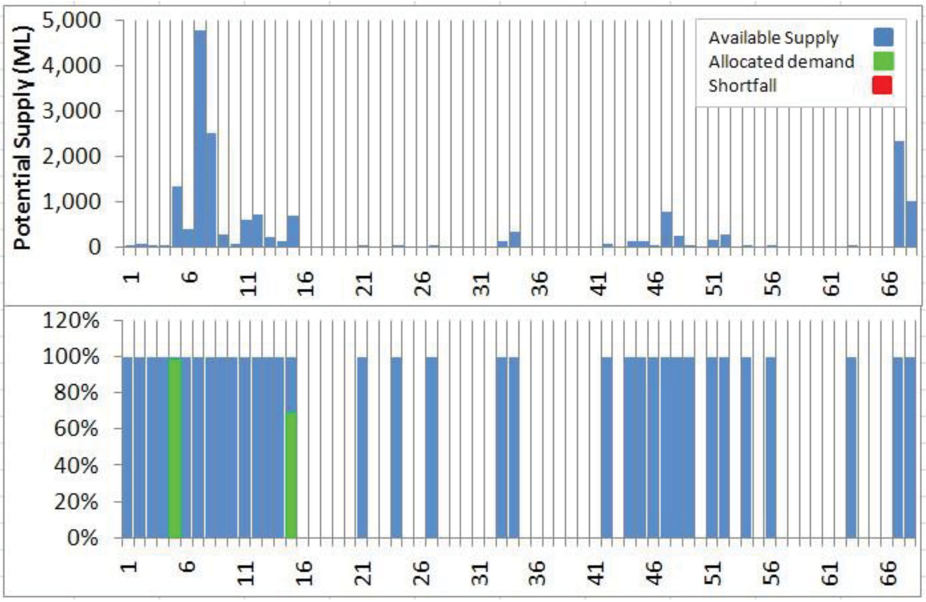


Figure 4 Source Allocation Graph

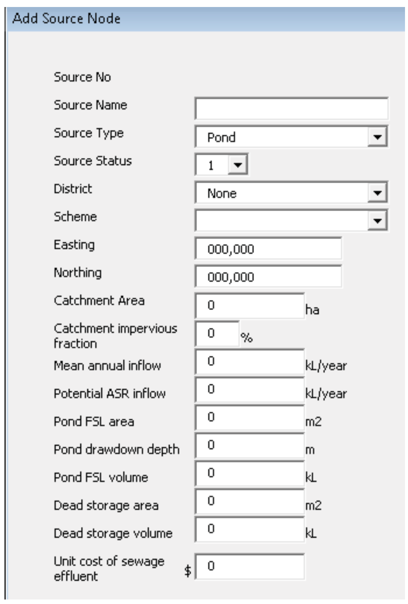
At any time while changes are made, the user can click "*Calculate demands for pipes'* to update the calculations, including the total demands being sourced from a given source.

Connection limitations

Demand and junction nodes can be connected to any upstream node.

Storage outlets must be connected to a source node (intervening junctions can be used)

Source nodes are not listed in the schedule of downstream nodes as they can only be upstream nodes (sources cannot be connected in series although they can be lumped as a single source with multiple outlets)



Adding and modifying nodes

The user can add or modify nodes using the 'Add new node' and 'Modify node' buttons then selecting the type of node to add or modify.

If a node is no longer required, it can be modified with null data and relocated off the map. Further details on node inputs are provided in Section 1.3.

1.2.3 Outputs

Summary of costs

The model provides a number of outputs with the main one being the Summary which summarises the costs of the system and other key metrics. These include total demands, storage volumes, pipe length and also yields taking into consideration reliability.

Region	All	Belconnen	CanberraCentral	Gungahlin	None	NorthCanberra	Tuggeranong	WestonCreek
Total demands (ML/year)	28,533	0	0	2,852	53,492	0	0	0
Total storage volume (ML)	10,105	1,059	6,705	874	0	59	921	428
Total yield (ML/year)	2,160	0	0	2,160	0	0	0	0
Total pipe length (m)	83,962	0	0	50,078	33,884	0	0	0
Total storage capital costs	$0	$0	$0	$0	$0	$0	$0	$0
Total pump capital costs	$154,409	$0	$0	$154,409	$0	$0	$0	$0
Total pipe capital costs	$12,394,858	$0	$0	$12,394,858	$0	$0	$0	$0
Total capital costs	**$12,549,266**	**$0**	**$0**	**$12,549,266**	**$0**	**$0**	**$0**	**$0**
Total storage annual costs	$8,738,383	$1,057,386	$5,187,971	$1,003,452	$0	$72,714	$1,150,106	$196,206
Total pump annual costs	$14,541	$0	$0	$14,541	$0	$0	$0	$0
Total pipe annual costs	$74,964	$0	$0	$74,964	$0	$0	$0	$0
Total annual costs	**$8,827,888**	**$1,057,386**	**$5,187,971**	**$1,092,957**	**$0**	**$72,714**	**$1,150,106**	**$196,206**
Annual revenue	$4,320,738	$0	$0	$4,320,738	$0	$0	$0	$0
Present value of total storage annual costs	-$128,668,499	-$15,569,502	-$76,390,386	-$14,775,352	$0	-$1,070,678	-$16,934,766	-$2,889,039
Present value of total pump annual costs	-$214,108	$0	$0	-$214,108	$0	$0	$0	$0
Present value of total pipe annual costs	-$1,103,810	$0	$0	-$1,103,810	$0	$0	$0	$0
Present value of total annual costs	-$129,986,418	-$15,569,502	-$76,390,386	-$16,093,270	$0	-$1,070,678	-$16,934,766	-$2,889,039
Total present value of costs	**-$142,535,684**	**-$15,569,502**	**-$76,390,386**	**-$28,642,537**	**$0**	**-$1,070,678**	**-$16,934,766**	**-$2,889,039**
Present value of revenue	**$63,620,801**	**$0**	**$0**	**$63,620,801**	**$0**	**$0**	**$0**	**$0**
Net value	**-$78,914,882**	**-$15,569,502**	**-$76,390,386**	**$34,978,265**	**$0**	**-$1,070,678**	**-$16,934,766**	**-$2,889,039**

A breakdown of costs is provided with capital and annual costs for storages, pumps and pipes. These costs include items such as contingency, design, supervision, administration, etc. Refer to Table 2 for a complete list of cost add-on percentages.

Revenue is estimated from the yield and assumed value of water. The net present value of all costs, of revenues and of the net are provided to allow the cost-effectiveness of different scenarios to be considered.

The summary shows results for the overall master plan as well as a breakdown by the five major districts. Note that demands are allocated to the district of their corresponding storage.

Network map

The network maps can be copied from the map by taking a screenshot to provide a record of the layout of the network.

1.3 Model input data

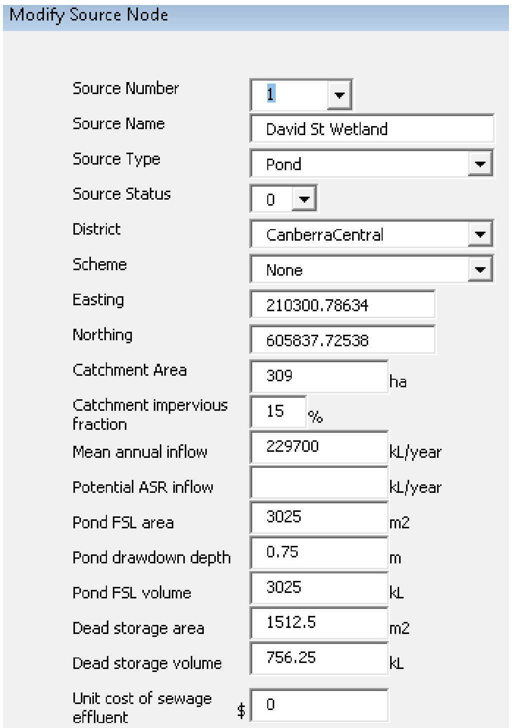
The model requires a range of inputs from the user to describe the system. Each source, demand or special node is represented using a single line or record in the spreadsheet.

1.3.1 Sources

Sources or source nodes are used to represent each potential non-potable source of water. A source may be a single pond, a number of ponds that are linked in series or a groundwater resource.

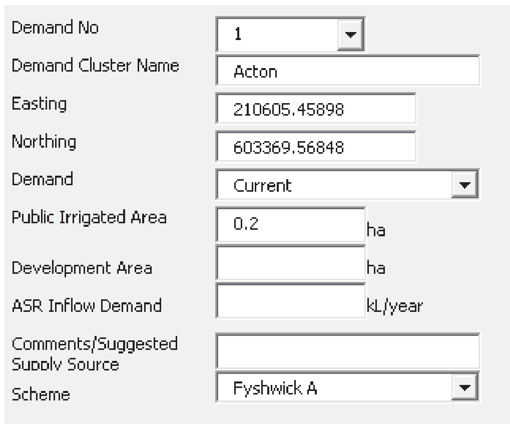
The following information is required:

- Source number – sorted in ascending order
- Source name
- Source type (Pond, Recycled Sewage, ASR, Groundwater)
- Source status (0 for existing or 1 for future)
- District



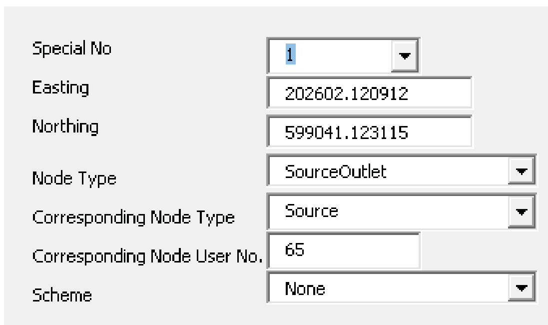
- Scheme
- Location details (easting and northing)
- Catchment details (pond sources)
    - Catchment area (ha)
    - Catchment impervious fraction
    - Mean annual inflow or supply (0 if to be calculated from catchment)
- Potential ASR inflow if applicable
- Pond details (pond sources)
    - Pond full supply area (m^2)
    - Pond storage drawdown depth (m)
    - Pond full supply volume (kL)
    - Dead storage area (m^2)
    - Dead storage volume (kL)
- Unit cost of sewage effluent (for Recycled Sewage)

1.3.2 Demands



- Demand cluster number
- Demand cluster name
- Location details (easting and northing)
- Demand status (current or future)
- Public irrigated demand area (ha)
- Development area (ha)
- Comments or suggested supply source
- Scheme

1.3.3 Special nodes



Special nodes are used to represent additional outlets for sources, junction nodes, and access points along supply lines such as for sewage recycling.

The following information is required:

- Special node number
- Location details (easting and northing)
- Node type (storage outlet, junction)
- Corresponding node type
- Corresponding node user number
- Scheme (nominal, can be modified later)

1.3.4 Model constants

The model requires a number of constants to define infrastructure requirements and cost curves. These are summarised in the tables and dot points below.

A set of cost coefficients is used in equations to estimate infrastructure costs, based on size. These coefficients were derived from the *Canberra Integrated Waterways* study [*Maheepala et al.*, 2009] while a number of assumptions were revised based on the Lake Tuggeranong study [*GHD*, 2010]. Initial estimates for disinfection were added. These disinfection costs were based on a very limited sample of small systems and further work to refine these estimates is recommended.

Table 1 Cost coefficients [*Maheepala et al.*, 2009]

Parameter	Storage [1]	Pipes	Pumps	Balancing Storage [2]	Ground water	Wetland	Disinfection [3]
Capital cost coefficient, a_1	100	8*	4000	6,390.8	89,000	100	6435
Capital cost coefficient, a_2	50		80,000		15,000		0.71
Edge works coefficient, a_3	100					1	
Planting coefficient, a_4						4.5	
Cost threshold			102				
Capital cost coefficient, b_1 [2]		1.45*	0.71	0.6586	0.02		
Capital cost coefficient, b_2		75	0.38		0.015		
Maintenance cost coefficient, c_1	185.4					185.4	
Maintenance cost coefficient, c_2	0.478					0.478	
Maintenance cost coefficient, *ß*		0.005	0.015				
Renewal cost coefficient, *ß*	0.014					0.014	0.014

[1]Based on costs for Lake Tuggeranong [*GHD*, 2010
[2]These were replaced with a schedule of pipes and costs developed based on the costs for Lake Tuggeranong [*GHD*, 2010
[3]Estimated

Table 2 Add-on cost percentages [*Maheepala et al.*, 2009]

	Storage	Pipes	Pumps	Balancing Storage	Ground water	Wetland	Disinfection
Contingency, investigations, design, supervision	40%	36%	36%	36%	70%	40%	40%
Capital cost insurance, administration and procurement	4.6%	4.6%	4.6%	4.6%	4.6%	4.6%	4.6%
Maintenance design, supervision	12%	12%	12%	12%	22%	12%	12%
Maintenance administration, procurement	8%	8%	8%	8%	8%	8%	8%

Pipes

- Friction factor for Darcy equation, f = 0.02
- Pipe friction loss factor, *PipeFrictionLossFactor* = 0.02
- (adjusted to estimate variable friction loss with length to replace constant friction loss, h_f = 25 from CSIRO method)
- Length factor, *LengthFactor* = 1.33, from calibration to Lake Tuggeranong (CSIRO 1.25)

Pump energy costs [*Maheepala et al.*, 2009]

- Head (m), h_t = 10
- Energy cost (c/kWh), *EnergyCostCentsPerkWh* = 15
- Efficiency, n = 0.75
- Pump constant *a1*, *a1_pump_energy* = 8.76
- Density of water, ρ = 1000
- Acceleration due to gravity (m/s^2), g = 9.81

Balancing storage

- Days supply for constant demands (Days), *ConstantDaysSupply* = 3
- Days supply for irrigation demands (Days), *IrrigationDaysSupply* = 3
- Assumed depth for balancing storages (m), *BalancingStorageDepth* = 2

These were reduced from the GHD assumption of 6 days as 2.3 days is sufficient, a slightly higher allowance of 3 days was made to be conservative and provide flexibility for users to vary irrigation rates and pressures.

Wetland

- Wetland detention time (days), a_5 = 3
- Wetland extended detention depth (m), a_6 = 0.45
- Wetland permanent pool and flood detention depth, a_7 = 0.55

Flows and demands

- Stormwater excess flow per ha (kL) for 100% impervious, *FlowPerHa_kL* = 5,830
- Peak monthly demand (% of annual), *PeakMonthlyDemandPercentage* = 20%
- Pumping hours per day (hours), *PumpingHoursPerDay* = 10
- Non-potable water use per person per year (kL), *WaterUsePerPerson* = 21.9
- People per household, *PeoplePerHousehold* = 3
- Households per ha, *HouseholdsPerHa* = 15
- Irrigable area per household (m^2), *HouseholdIrrigableArea* = 320 (ACTPLA, this is considered to be high)
- Irrigation demand (kL/year/m^2), *IrrigationDemand* = 0.6

Sewage plant costs

- Fyshwick sewage effluent cost, *FyshwickSewageCost*, $4.00 (ACTEW, 2010)

- Lower Molonglo sewage effluent cost, *LowerMolongloSewageCost*, $0.70 (ACTEW, 2010)

Present value and levelised costs

- Discount rate, *DiscountRate* = 6.5%
- Lifespan (years), *Lifespan* = 50
- Water selling price ($/kL), *WaterValue* = $2.00

1.4 Model calculations

The model undertakes a variety of calculations to determine the storage capacity, demands and the costs of the storages, pipe network and pumps. The calculations to estimate reliability and also the costs are summarised below.

1.4.1 Reliability calculation

The reliability calculation is a tabular lookup based on a dimensionless table. It relates the storage size, the demand volume, the inflow volume and the corresponding reliability.

The dimensionless table works on the principle that the ratio of storage/flow and demand/flow determines the reliability of a system.

A series of model runs have been undertaken to derive a range of curves. One curve represents a single Demand/Flow Ratio and shows how the reliability changes as the Storage/Flow ratio changes. As expected, reliability increases as the relative size of the storage increases. This occurs up to a point where evaporation starts to decrease water availability for reuse.

The dimensionless table and method are the keys to the simplicity of the model as it eliminates the need for any live model runs.

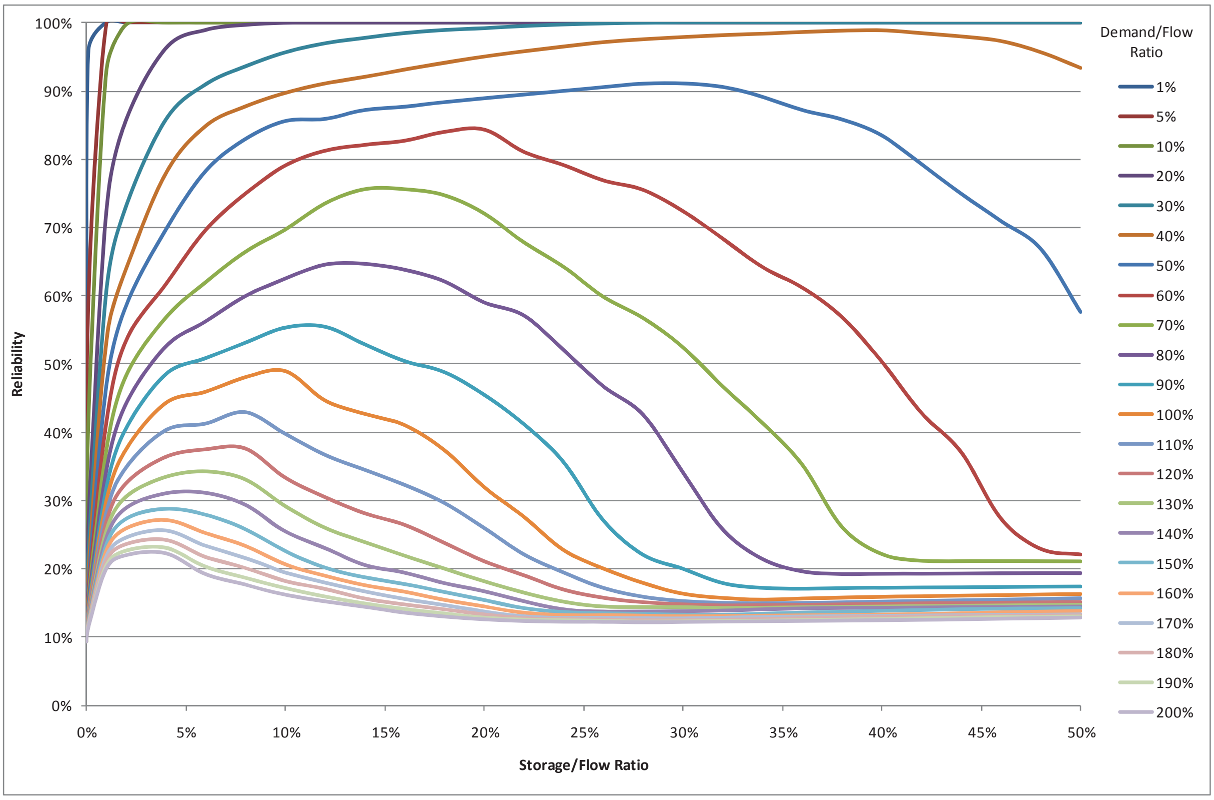


Figure 5 **Source reliability plot**

An example of the calculations undertaken is shown in Figure 6.

NonPotableMasterPlanningTool_Beta1.0.6.xlsm:5

	P	Q	R	S	T	U	V	W	X	Y	Z	AA	AB	AC	AD	AE	AF	AG	AH
1																			
2	Node No	Active pond volume (m^3)	Dead storage depth (m)	Runoff factor	Total Flows (kL)	Irrigation demand (kL/year)	Constant demand (kL/year)	Total Demands (kL/year)	Demand/Flow Ratio	Storage/Flow Ratio	Minimum active pond volume to meet target reliability (m^3)	Revised Storage/Flow Ratio	Reliability - multi bilinear interpolation	Demand/Flow ratio at target reliability	Potential demand supplied at target reliability (kL/yr)	Shortfall (%)	Allocated Demand (%)	Available Supply (%)	Yield
3	1	2,269	0.50	0.6	229,700	0	0	0	0.0%	1.0%	2,269	1.0%	0.0%	1.2%	2,764	0%	0%	100%	0
4	2	12,220	1.00	0.6	108,500	0	0	0	0.0%	11.3%	12,220	11.3%	0.0%	10.1%	11,006	0%	0%	100%	0
5	3	13,330	0.33	0.6	52,400	0	0	0	0.0%	25.4%	13,330	25.4%	0.0%	0.0%	0	0%	0%	0%	0
6	4	11,404	0.33	0.6	20,288	0	0	0	0.0%	56.2%	11,404	56.2%	0.0%	0.0%	0	0%	0%	0%	0
7	5	252,000	1.68	0.6	3,475,473	0	0	0	0.0%	7.3%	252,000	7.3%	0.0%	10.2%	354,540	0%	0%	100%	0
8	6	46,080	0.75	0.6	3,445,400	0	0	0	0.0%	1.3%	46,080	1.3%	0.0%	1.2%	41,559	0%	0%	100%	0
9	7	528,000	3.03	0.6	5,500,000	254,016	86,921	340,937	6.2%	9.6%	528,000	9.6%	98.6%	10.3%	567,440	0%	60%	40%	336,293
10	8	532,980	4.01	0.6	6,128,500	0	0	0	0.0%	8.7%	532,980	8.7%	0.0%	10.2%	626,264	0%	0%	100%	0

Figure 6 **Source calculations**

1.4.2 Infrastructure sizing and costs

Costs were estimated for each different type of infrastructure including capital, annual (maintenance and renewal) and add-on costs. Most of the equations and parameters used for the cost estimation were based on the *Canberra Integrated Waterways* study by CSIRO [*Maheepala et al.*, 2009 while improvements to some were made based on comparison and calibration to recent schemes [*GHD*, 2010]. This was particularly significant for pipe and balancing storage costs.

Add-on costs

Add on costs are summarised in Table 3 after [*Maheepala et al.*, 2009].

Table 3 Summary of 'add-on' costs

Item	Contingency	Special Investigations	Consultant Design & Supervision	Insurance	Administration & Procurement
Capital					
Ponds	20%	12%	8%	0.6%	4%
Pumps	20%	12%	4%	0.6%	4%
Pipes	20%	12%	4%	0.6%	4%
Aquifers	30%	20%	20%	0.6%	4%
Sewer mining	20%	12%	8%	0.6%	4%
Operation, Maintenance & Replacement					
Ponds	0%	0%	12%	0%	8%
Pumps	0%	0%	12%	0%	8%
Pipes	0%	0%	12%	0%	8%
Aquifers	10%	0%	12%	0%	8%
Sewer mining	0%	0%	12%	0%	8%

Ponds

The size of new ponds was based on the proposed design as obtained from the CSIRO study [*Maheepala et al.*, 2009] and updated information where available. Pond capital costs are assumed to be $100/m^3 for the first 20,000 m3 then $50/m^3 for additional volume above that, based on CSIRO [*Maheepala et al.*, 2009].

$$Pond\ Capital\ Cost\ (\$) = \begin{cases} 100\ x\ volume\ , & volume < 20{,}000 \\ 20{,}000{,}000 + 50\ x\ (volume - 20), & volume \geq 20{,}000 \end{cases}$$

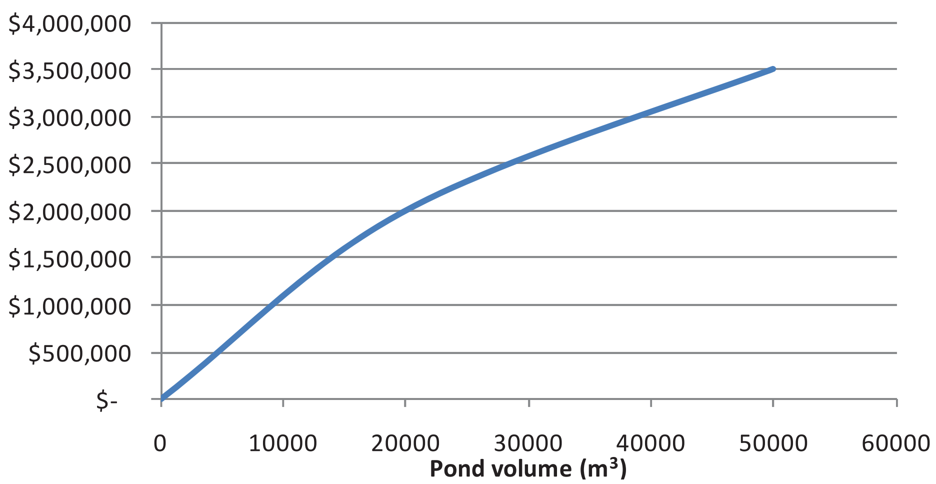


Figure 7 Pond capital cost curve ($)

For existing storages, it was assumed that edge treatment works would be required if the drawdown depth exceeded 0.2 m. Costs were then estimated for revegetation of the edges with an assumed rate of $100/m of length. This is an assumption and should be refined for more detailed investigations.

The perimeter length was estimated based on area as:

$$P = 2\pi\sqrt{(A/\pi)}$$

And the cost was estimated as:

$$Pond\ capital\ cost\ (\$) = \$100 \times P$$

Where,

P = Perimeter length

A = Area

Total annual maintenance (TAM) was calculated based on the formula in MUSIC (Wong et al, 2005)

$TAM = 185.4 * A^{0.478}$

Renewal and adaptation costs (RAC) were also calculated based on the formula in MUSIC (Wong et al, 2005)

RAC ($) = 0.014 * Capital cost

Pond capital costs				Pond total annual maintenance (TAM)				Pond renewal and adaptation		
Pond Capital costs	Pond capital contingency, investigations, consultant design and supervision	Pond capital insurance, administration and procurement	Total Pond capital costs with add-ons	Pond TAM	Pond maintenance design costs	Pond maintenance administration costs	Pond TAM with add-ons	Nominal Pond Capital costs	Pond renewal and adaptation	Total Pond annual costs
$ -	$ -	$ -	$ -	$ 8,549	$ 1,025.84	$ 765.96	$ 10,340	$ 226,875	$ 3,176	$ 13,517
$ -	$ -	$ -	$ -	$ 12,753	$ 1,530.39	$ 1,142.69	$ 15,426	$ 1,222,000	$ 17,108	$ 32,534
$ -	$ -	$ -	$ -	$ 12,727	$ 1,527.25	$ 1,140.34	$ 15,395	$ 1,333,000	$ 18,662	$ 34,057
$ -	$ -	$ -	$ -	$ 11,812	$ 1,417.46	$ 1,058.37	$ 14,288	$ 1,140,400	$ 15,966	$ 30,254
$ -	$ -	$ -	$ -	$ 68,814	$ 8,257.68	$ 6,165.74	$ 83,237	$ 11,086,000	$ 155,204	$ 238,441
$ -	$ -	$ -	$ -	$ 34,962	$ 4,195.40	$ 3,132.56	$ 42,290	$ 3,304,000	$ 46,256	$ 88,546
$ -	$ -	$ -	$ -	$ 140,417	$ 16,850.09	$ 12,581.40	$ 169,849	$ 48,696,000	$ 681,744	$ 851,593
$ -	$ -	$ -	$ -	$ 104,309	$ 12,517.10	$ 9,346.10	$ 126,172	$ 27,649,000	$ 387,086	$ 513,258
$ -	$ -	$ -	$ -	$ 29,732	$ 3,567.83	$ 2,663.98	$ 35,964	$ 2,703,160	$ 37,844	$ 73,808
$ -	$ -	$ -	$ -	$ 29,369	$ 3,524.32	$ 2,631.49	$ 35,525	$ 2,580,000	$ 36,120	$ 71,645
$ -	$ -	$ -	$ -	$ 58,319	$ 6,998.24	$ 5,225.35	$ 70,542	$ 8,308,000	$ 116,312	$ 186,854
$ -	$ -	$ -	$ -	$ 44,631	$ 5,355.70	$ 3,998.93	$ 53,986	$ 4,984,000	$ 69,776	$ 123,762
$ -	$ -	$ -	$ -	$ 31,103	$ 3,732.39	$ 2,786.85	$ 37,623	$ 2,691,250	$ 37,678	$ 75,300
$ -	$ -	$ -	$ -	$ 45,510	$ 5,461.24	$ 4,077.72	$ 55,049	$ 4,750,000	$ 66,500	$ 121,549
$ -	$ -	$ -	$ -	$ 72,710	$ 8,725.18	$ 6,514.80	$ 87,950	$ 11,838,000	$ 165,732	$ 253,682
$ -	$ -	$ -	$ -	$ 348,113	$ 41,773.61	$ 31,190.96	$ 421,078	$ 334,114,000	$ 4,677,596	$ 5,098,674
$ -	$ -	$ -	$ -	$ 27,927	$ 3,351.22	$ 2,502.25	$ 33,780	$ 3,000,000	$ 42,000	$ 75,780
$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -

Figure 8 Source cost calculations

1.4.3 Pipe calculations

The pipes sheet is used to track each of the pipes connecting demands to their respective sources. It allows a network of pipes to be developed connecting pipes from sources to source outlets, junctions and other demand nodes on a path to a demand node.

The '**Calculate demands for pipes**' button is used to update the pipe sheet by recalculating the total demands to a downstream node and the ultimate upstream storage for all nodes.

Peak flow rate (Q_{peak})

The peak flow rate used for sizing pipes is determined from the peak monthly demand flowing through the pipe. The percentage of demand for this month and the daily pumping duration are constants that can be set by the user.

Q_{peak} (m^3/s) = TotalDemandToDownstreamNode * PeakMonthlyDemandPercentage / (31*86400) * (24/PumpingHoursPerDay)

Node distance

The node distance is calculated as the shortest distance between the nodes.

$$Distance = L_f \times \sqrt{(EastingUpstream - EastingDownstream)^2 + (NorthingUpstream - NorthingDownstream)^2}$$

Pipe length

The pipe length was estimated as:

$$Pipe\ length = Length\ factor \times Node\ distance$$

Where,

Length factor = 1.33

This was determined based on calibration for the direct and actual pipe distance lengths for the Preliminary Sketch Plans for the Lake Tuggeranong scheme [*GHD*, 2010]. The factor is slightly higher than the 1.25 assumed by CSIRO [*Maheepala et al.*, 2009].

Pipe diameter (mm)

The pipe diameter was estimated using the Darcy-Weisbach equation after CSIRO [*Maheepala et al.*, 2009].

$$D = \frac{fLV^2}{2h_f g}$$

Substituting $V = \frac{Q}{A} = \frac{4Q}{\pi D^2}$.

$$h_f = \frac{8fLQ^2}{\pi^2 g D^5}$$

Where,

D = diameter (m)

f = Darcy-Weisbach friction factor (0.02 assumed)

L = Pipe length (m)

V = velocity (m/s)

h_f = head loss due to friction (m)

g = 9.81 m/s^2

The friction head loss was estimated based on an assumed factor of 0.02 multiplied by the length of pipe. This was determined after testing to assume a reasonable factor to use as it was found that the constant head loss used by CSIRO could result in unrealistic pipe sizes, particularly for large recycled wastewater transfer lines from

LMWQCC with high flow rates and a wide range of lengths, for which a variation in total friction head loss would be expected. It is important that pipe sizes should be further refined during detailed investigations.

Nominal pipe diameter (mm)

The nominal pipe diameter was determined as the nearest size up for the calculated pipe diameter. Pipe sizes of 125, 180, 250, 315, 400, 450, 500, 560, 6630, 710, 800, 900, 2x800, 2x900, 4x900 and 5x900 were used. A warning is given if this is exceeded.

Pipe Capital costs were estimated based on the following equation [*Maheepala et al.*, 2009]:

PipeCapital ($) = Pipe Length * 1.45 * Nominal Pipe Diameter (mm)

The constant 1.45 is indicative and can vary from 1.1 to as much as 5.0 for boring. While adequate for conceptual design it is not suitable for detailed design.

Pipe operating and maintenance costs were estimated as follows [Maheepala et al., 2009]:

PipeO&M ($/year) = ß * PipeCapital

Where,

ß = 0.005 and capital cost excludes add-on costs

1.4.4 Pumps

Peak flow rate for pumps ($Q_{peakpump}$)

The peak flow rate used for sizing pumps was determined from the peak monthly demand flowing through the pipe from a source node. It is assumed that each pipeline derived from a source will have its own pump as many of the storages are relatively large and may have multiple offtake points.

$Q_{peakpump}$ (m^3/s) = If(UpstreamNodeType = Source, Q_{peak}, 0)

Average flow rate ($Q_{average}$)

This is used to estimate pump energy costs

$Q_{average}$ (m^3/s) = TotalDemandToDownstreamNode / 365 /86400

Pump capital cost

The formula adopted for estimating pump capital costs was the same as that used by CSIRO [Maheepala et al., 2009] based on advice from William Bencke and Kirrilly Dickson of ACTEW, see Figure 9 and is as follows:

PumpCapitalCost ($) = IF(Q<102, 4000*Q, 80,000*(0.71*Q)$^{0.38}$

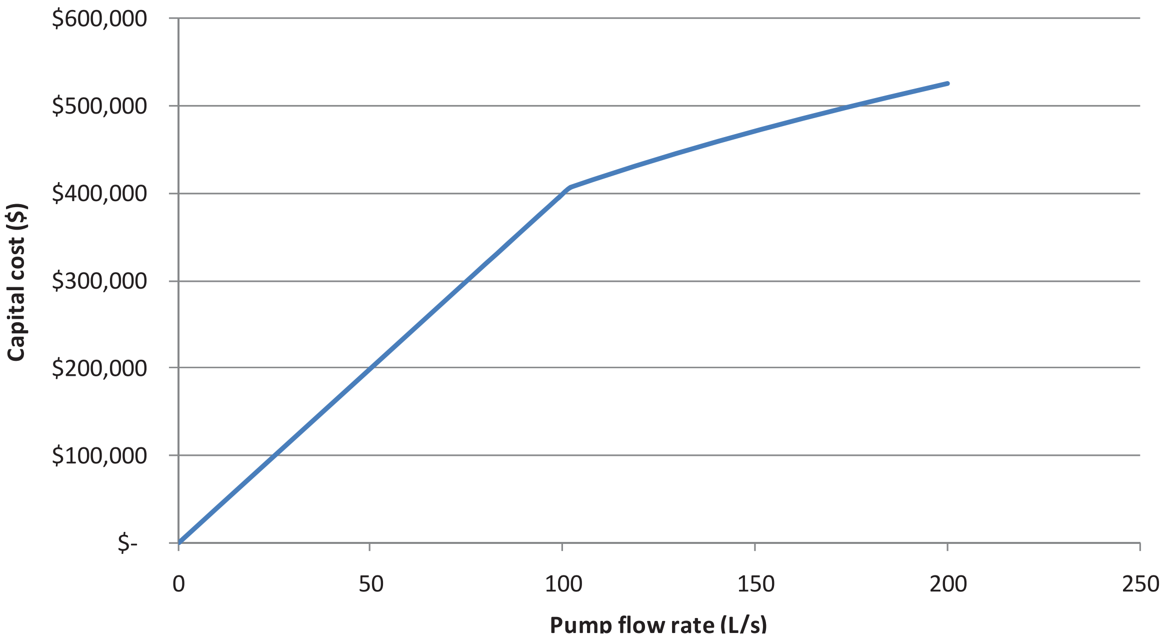


Figure 9 Pump capital cost

Pump O&M

Pump O&M ($/year) = ß * PumpCapitalCost

Where, ß = 0.015 for pumps and capital cost does not include add-ons.

Energy

The annual pump energy cost is calculated as follows [*Maheepala et al.*, 2009]:

$$C_e = \frac{8.76\rho g Q_{avg} h_t R}{\eta}$$

Where,

C_e = Annual pump energy cost ($/year)

ρ = specific gravity

g = acceleration due to gravity

Q_{avg} = average flow (m^3/s)

H_t = pumping head (m)

R = energy cost (assume 15c/kWH)

η = pumping efficiency (assume 0.75)

Total annual pump costs

Total annual pump costs ($/year) = Total pump O&M + C_e

1.4.5 Balancing storage costs

It was assumed that all balancing storages would be underground concrete tanks for security and durability. The costs estimated for the Lake Tuggeranong scheme [*GHD*, 2010] were used as a basis for the cost estimates.

BalancingStorageCapitalCost = $a1_{balancing}$ * BalancingStorageVolume (kL) $^{b1_{balancing}}$

Where,

$a1_{balancing}$ = 6390.8

$b1_{balancing}$ = 0.6586

Total annual maintenance (TAM) was calculated based on the formula in MUSIC for ponds (Wong et al, 2005)

TAM = 185.4 * $A^{0.478}$

Renewal and adaptation costs (RAC) were also calculated based on the formula in MUSIC for ponds (Wong et al, 2005)

RAC ($) = 0.014 * Capital cost

1.4.6 Wetland cost

It was assumed that flow-through wetland treatment would be provided for all stormwater from ponds. In practice, all inflows to ponds would ideally be treated in order to protect the pond as a receiving water from high nutrient and sediment levels that may lead to algal blooms and other problems. As reuse volumes may only be a small proportion of inflows, the proportional cost attributable to the reuse scheme was estimated based on the demand volume. This was used to determine a wetland area and corresponding cost.

The area of wetland required was estimated as:

$$A_{wetland} = Total\ demand\ \left(\frac{kL}{year}\right) \times \frac{Detention\ time\ (days)}{365} \times \frac{1}{Extended\ detention\ depth}$$

Where,

$A_{wetland}$ = area of wetland (m^2)

Total demand = total demand including irrigation and constant demand (kL/year)

Detention time = average time taken for water to pass through the wetland (days), assumed to be 3 days

Extended detention depth = active storage depth of wetland providing treatment (m), assumed to be 0.45 m

The cost of wetlands were estimated using the same excavation costs as ponds while assuming additional costs for planting and topsoil as follows:

$$Cost_{wetland} = c_{excavation} \times A_{wetland}\ (EDD + PP + Flood\ detention) + A_{wetland} \times (c_{topsoil} + c_{planting})$$

Where,

$c_{excavation}$ = excavation cost ($/$m^3$)

$A_{wetland}$ = wetland area

EDD = Extended detention depth (m), assumed 0.45m

PP = Permanent pool depth (m), assumed 0.3m

Flood detention = flood detention depth (m), assumed 0.25m

$c_{topsoil}$ = topsoil cost ($/m^2)

$c_{planting}$ = planting cost ($/m^2)

Wetland total annual maintenance (TAM) and renewal costs were obtained from MUSIC and assumed to be as follows:

TAM = 6.831 x $A^{0.8634}$

Wetland renewal cost = 0.52% * Total acquisition cost

1.4.7 Disinfection treatment costs

Disinfection treatment costs were included based on a very limited sample of costs for relatively small systems. However, the costs were found to be relatively insignificant relative to other infrastructure.

DisinfectionCapacity (L/s) = [Irrigation demand (kL/year) *PeakMonthlyDemandPercentage / 31 + Constant demand (kL/year)] * (1000 / 86400)

DisinfectionCost = $a1_{disinfection}$ * DisinfectionCapacity $^{b1_{disinfection}}$

Where,

$a1_{disinfection}$ = 6435

$b1_{disinfection}$ = 0.71

Total annual maintenance (TAM) and renewal and adaption costs were calculated using formulas of the same form as those used for ponds. However, in the absence of cost data and in recognition of the relative insignificance of these costs relative to other infrastructure, the constants have been set to zero and these costs have been ignored. They can be incorporated for more detailed future studies by adjusting the cost coefficients for disinfection.

1.4.8 Present value and levelised cost

The total cost of each water supply scenario is made up of the following three components:

Capital Costs + Operation & Maintenance Costs + Replacement Costs

Residual value costs have not been included as they would have minimal impact on the overall present value cost given the long time frames of 50 years or more.

It was assumed that all construction is completed in year 0 of the analysis. No allowance was made for staging of implementation at this time due to the simplified nature of the model. The effect of delayed implementation on costs and yields should be considered in further detailed investigations.

The present value of annual operations and maintenance costs are calculated using the following formula:

$$PV_{O\&M} = A\left[\frac{(1+r)^T - 1}{r(1+r)^T}\right]$$

Where,

A = annual cost ($/year)

r = discount rate (%)

T = analysis period (assume 50 years)

Levelised costs were calculated according to the following equation [*Maheepala et al.*, 2009]:

$$LevelisedCost(\$) = \frac{\sum_{n=1}^{t} C_n\,(1+r)^{\frac{1}{n}}}{\sum_{n=1}^{t} V_n\, r(1+r)^{\frac{1}{n}}}$$

Where,

T = analysis period (50 years)

n = year (1 through 50)

C = cost

V = volume of the demand met over analysis period

Disclaimer: *AECOM has compiled these cost assumptions using information available to AECOM and where required, based on assumptions made by AECOM. Prices and quantities in the assumptions may change. AECOM does not represent, warrant or guarantee that the master plan can be completed for the cost estimates based on these assumptions.*

1.5 Calibration of the model

Calibration of the model was undertaken by comparison with detailed designs for proposed stormwater harvesting schemes for Lake Ginninderra and Lake Tuggeranong to determine if the model could produce similar results.

The hydrologic model was calibrated to fit the outputs firstly to the modelled data by GHD and secondly to the observed data to determine appropriate parameters for use in the model.

1.5.1 Hydrologic model calibration to GHD model

To calibrate each pond, the spreadsheet and/or MUSIC was first set up with input data matching the GHD model as closely as practical.

Rainfall

The same rainfall as that used by GHD was adopted, using daily rainfall for a 31 year period from 1979-2009.

For the North Canberra Ponds, Ginninderra rainfall was adopted and for the South Canberra Ponds, Canberra Airport rainfall was adopted.

Table 4 Rainfall data

Station	Mean annual (mm)
Canberra Airport 070014	583
Ginninderra 070169	658

Evaporation

It is apparent that GHD have used Daily evaporation x 0.9 for modelling both catchment and pond evaporation. This is significantly different to the Potential Evapotranspiration (PET) commonly used in MUSIC and also the SimHyd modelling undertaken by CSIRO. It is likely that pond evaporation rates would be higher than reference PET, which refers to evapotranspiration from soil for a reference crop. Therefore, for the MUSIC modelling, the standard PET was used and the evaporative losses from ponds were adjusted upwards (set to 139%) to match the evaporation rates for the GHD modelling.

Table 5 Evaporation data

Evaporation	Mean annual (mm)
Daily evaporation (GHD, from BoM)	1,720
Daily evaporation x 0.9 factor	1,548
Daily evaporation x 0.9 x 0.7 crop factor	1,083
Potential evapotranspiration (PET) from BoM	1,116

Results

Lake Ginninderra

For Lake Ginninderra, good fits between the spreadsheet model and reported results for the detailed design were obtained for both water balance and levels. The calibration resulted in an effective impervious fraction of 48% of the reported total impervious fraction. This suggests that effective impervious fractions may be significantly less than total impervious fractions and these should likely be reduced for North Canberra sites using this rainfall data. For comparison, it is noted that the CSIRO data appears to adopt total impervious fractions.

A water balance was undertaken (infilling gaps in the information provided for the GHD model such as evaporation) and for MUSIC and spreadsheet models for each of the storages. Inflows for the spreadsheet were based on impervious area runoff from MUSIC with the impervious fraction calibrated to match the GHD inflows.

The water balance for Ginninderra indicates that a very good fit was obtained, see Figure 10.

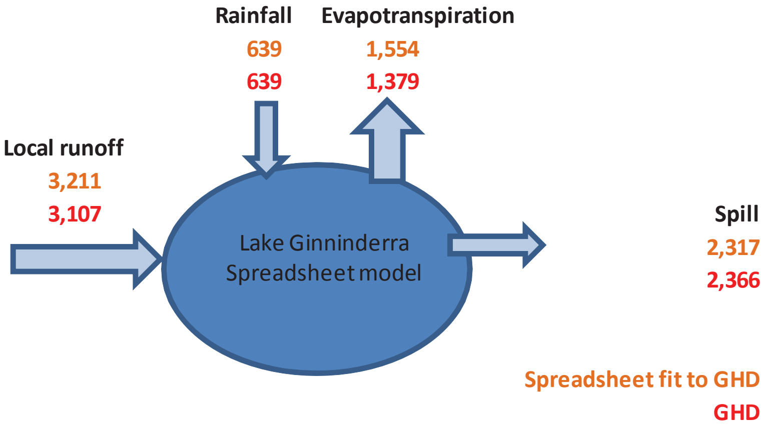


Figure 10 Water balance for Lake Ginninderra

Water levels were plotted for the GHD model, the calibrated spreadsheet model for Lake Ginninderra and observed levels, see Figure 11. The levels were found to be quite sensitive to factors such as the evaporation and direct rainfall, while the presence or absence of flows from upstream storages appeared to have little effect, most likely because they only occurred when overflows were occurring (and the lake was full). The results show that the spreadsheet and MUSIC match the GHD model acceptably well, though tend to have higher levels in the lower part of the range. It is clear that all of these under-predict water levels relative to observed levels. This may suggest there are other unaccounted for (dry period) inflows or that the evaporation used may be too high and that the model could be improved.

The area at the FSL level based on the detailed stage-storage curve from the GHD model (which differs slightly from the report) was adopted. As there is relatively little change in area at these levels, the use of a constant area was found to be adequate with addition of a variable area to the model having minimal effect although this may not hold for smaller ponds. This validates the adequacy of the simplified model where drawdown levels are kept within reasonable limits.

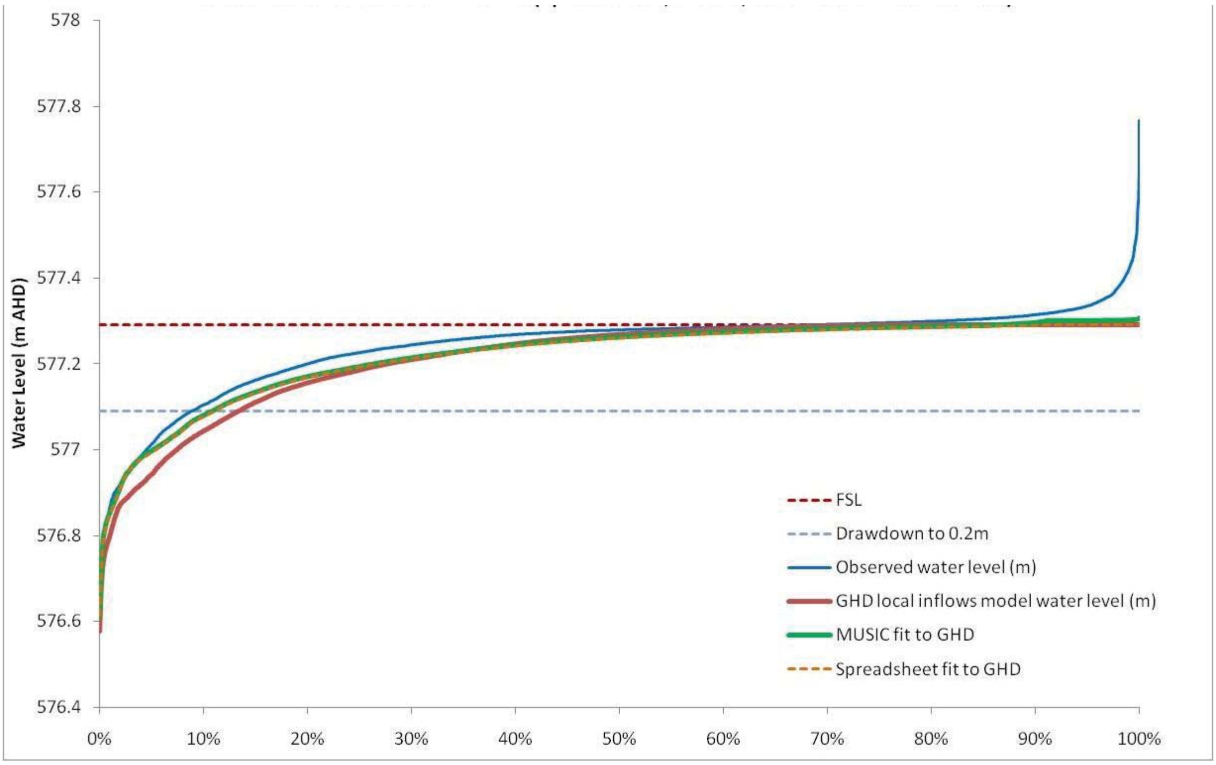


Figure 11 **Water levels for Lake Ginninderra comparing calibrated spreadsheet and MUSIC model with GHD model and observed**

As the results indicate that MUSIC and the spreadsheet produce similar results, only MUSIC calibrations were considered necessary for the remaining ponds.

Lake Tuggeranong and Isabella Pond

Isabella Pond drains to Lake Tuggeranong, therefore they have been considered together.

The results for the water balances show that a good fit can be obtained to the GHD model. However the predicted water levels indicated in the water level percentile curve for Lake Tuggeranong are significantly higher than those for the GHD model and very low impervious fractions would be needed to match them. Conversely, those for Isabella Pond are lower, even using a higher assumed ratio of effective to total impervious fractions. Adjusting the model to improve the fit to the GHD water levels would necessitate significant change in the water balance figures and it is also apparent that improvement in the fit for one of the storages will result in a poorer fit for the other storage. Therefore, a fit was chosen that balances the competing requirements of fitting the water levels and balances for the two storages simultaneously.

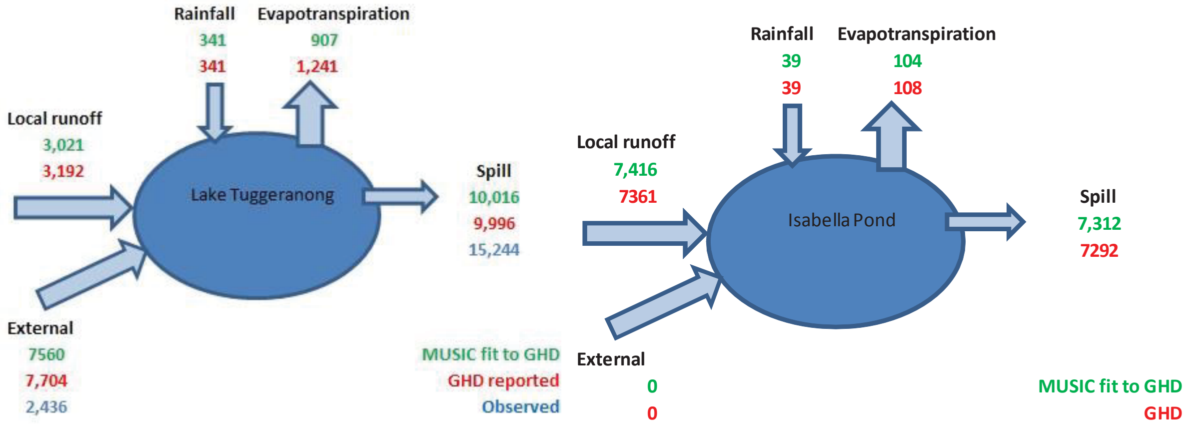


Figure 12 Water balances for Lake Tuggeranong and Isabella Pond

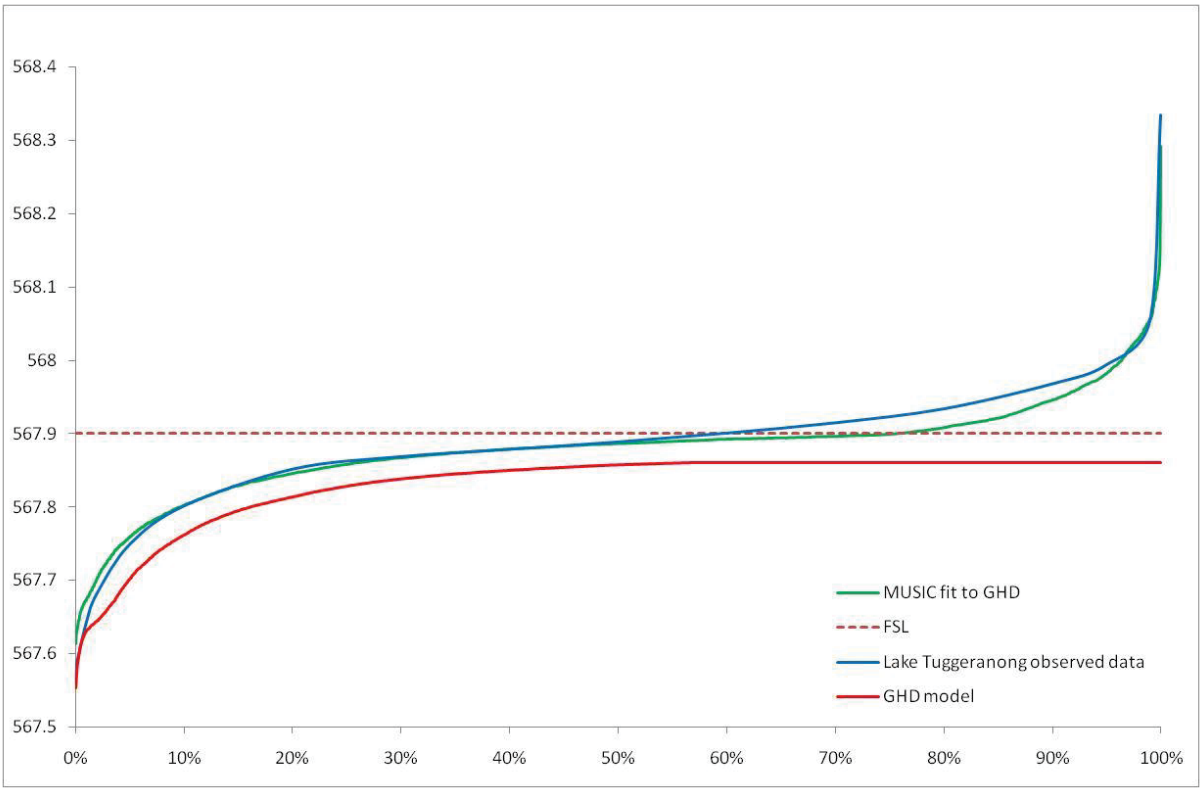


Figure 13 Water levels for Lake Tuggeranong

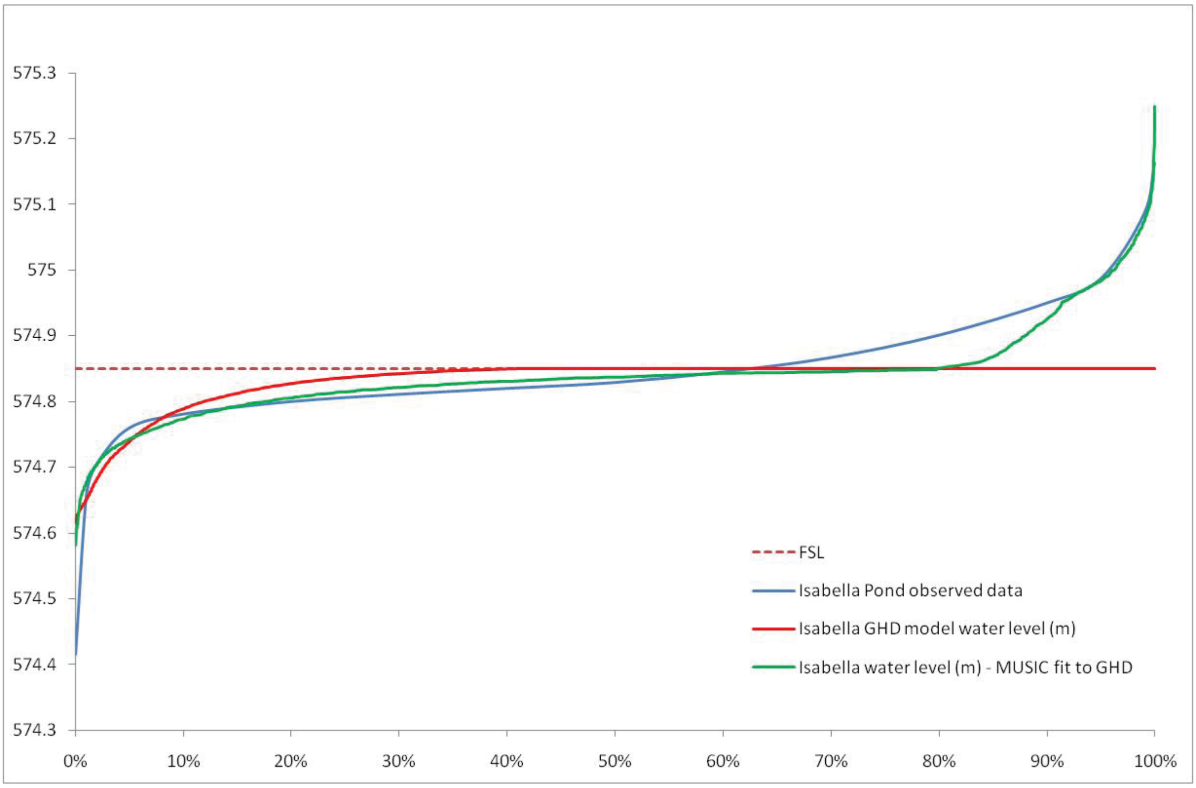


Figure 14 Water levels for Isabella Pond

Through the preparation of the percentile plots it was recognised that there was potential to improve the fits to the observed data. In particular it was noted that:

- There were some significant differences between the GHD and observed water levels when viewed in percentile plots and there may be opportunity to improve the fit through direct calibration to the observed data. These were particularly significant in the likely drawdown range of 0 to -0.4m for Lake Ginninderra
- There is potential to improve the fit to the observed data for the storages within the Tuggeranong system while adopting a consistent approach for each of the catchments draining to Lake Tuggeranong.

1.5.2 Hydrologic calibration to observed data

The calibrations were repeated, targeting the fitting of the observed data. Water level was considered to be the most accurate and important parameter (for the purposes of evaluating potential drawdown volumes that occur mostly during summer periods) while spill volumes were considered secondary and of lower reliability. It is also recognised that as the model only considers the impervious portion of the catchment that flows due to large storm events generating pervious area runoff, mostly during winter, will tend to be underestimated.

Observed data was available for the ponds above and a number of additional ponds were considered. While adjustment of the evaporation rates was considered, it was found that similar variations in results could generally be obtained by adjusting the impervious fractions so the evaporation rates were fixed for consistency across the models.

Lake Ginninderra

The MUSIC model for Lake Ginninderra was adjusted by increasing the assumed impervious fraction and by incorporating overflows from Gungahlin Pond and Yerrabi Pond. The results indicate a significantly improved fit to water levels. In the range 300-400 mm below FSL there is some over-prediction of levels, however there is similarly still a slight under-prediction of levels in the range 0-300 which likely correlates to these. The spill volumes are higher though still well short of the observed volumes. While the impervious fractions could be increased to increase the volumes, this would result in a poorer fit for the water level curve which was considered the primary indicator for the quality of the fit. This is likely to be due to the occurrence of significant pervious area runoff which is not represented. This would mostly occur during large events, when the impervious area is contributing runoff and the lake level is at or above the full supply level. Therefore, while such flows may significantly affect spill volumes, they have less significant implications for lake water levels during dry periods. Overall, the calibration is considered to provide a good fit, which is still likely to be conservative.

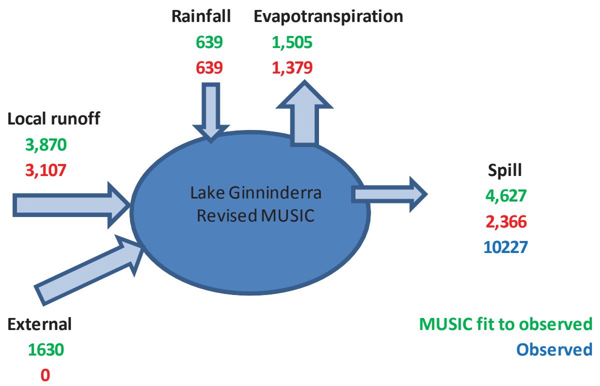


Figure 15 Water balance for Lake Ginninderra with calibration to observed

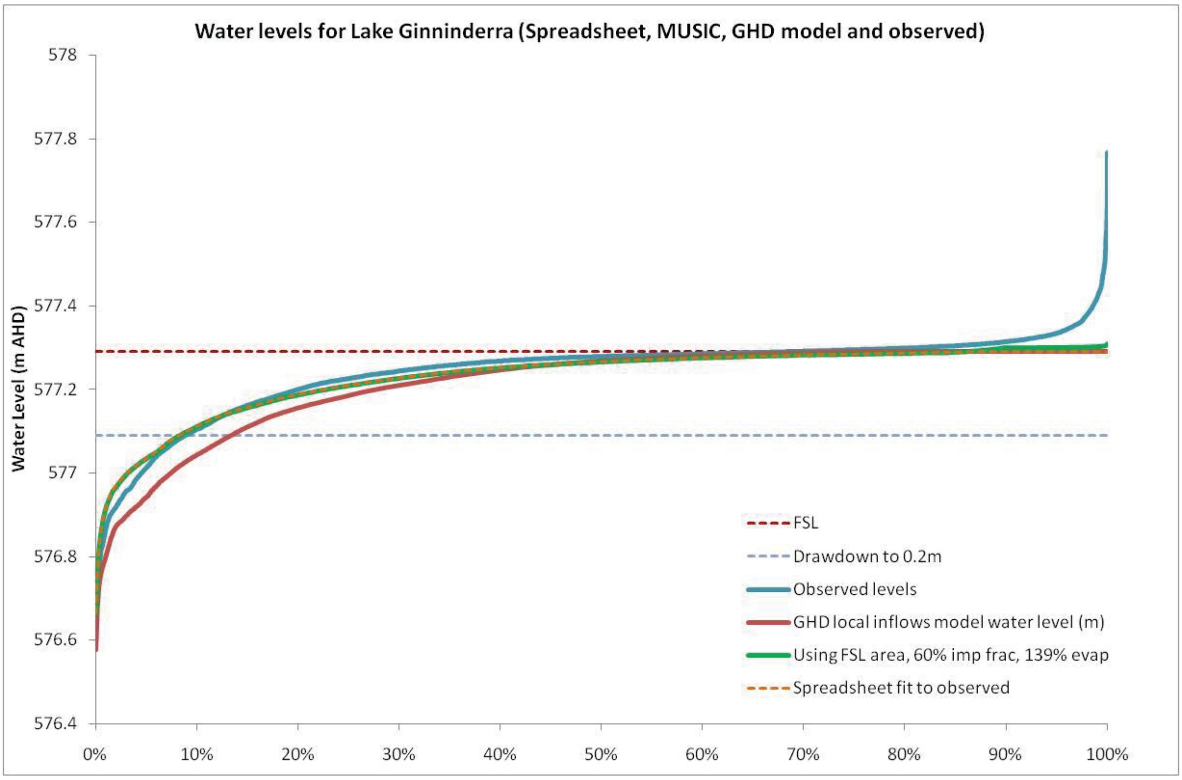


Figure 16 Water levels for Lake Ginninderra with calibration to observed

Isabella Pond, Tuggeranong Weir and Lake Tuggeranong

Isabella Pond drains to Tuggeranong Weir and then into Lake Tuggeranong. These were represented in a single MUSIC model to allow the integrated system to be evaluated.

Data for spill flows from Tuggeranong Weir and Lake Tuggeranong were available. These suggest that the model predicts higher external flows and lower local flows than observed, while the catchment parameters adopted also have a lower ratio of effective to total imperviousness for the local Lake Tuggeranong catchment. The impervious fraction was increased for Lake Tuggeranong to increase the local flow volumes and water levels and this is consistent with the reported impervious fractions, which suggest a higher impervious fraction for Lake Tuggeranong than Isabella Pond. While the impervious fractions adopted are higher than those reported, the ratio of adopted/reported impervious fractions was kept consistent for each of the three catchments. This resulted in an improvement in the fit for Lake Tuggeranong, without adverse impacts on the fit to the other storages.

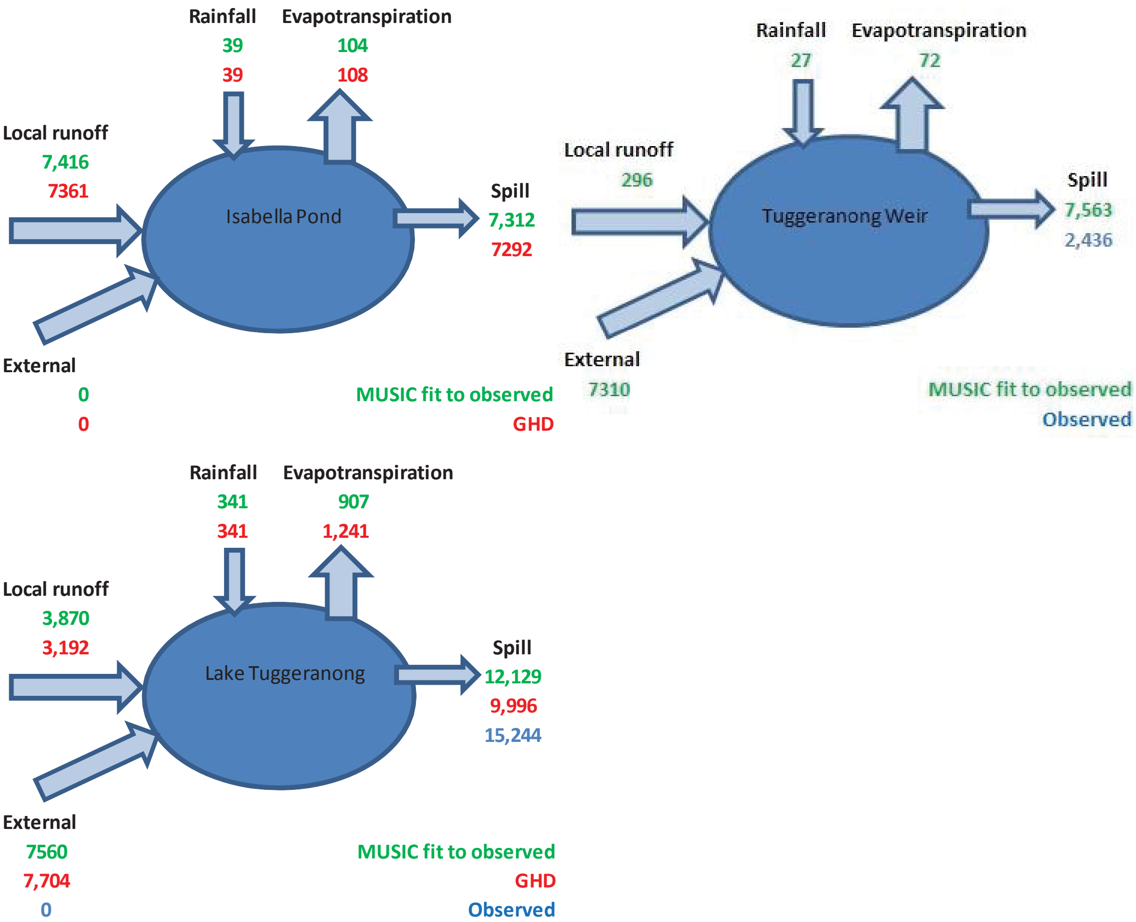


Figure 17   Water balance for Isabella Pond, Tuggeranong Weir and Lake Tuggeranong

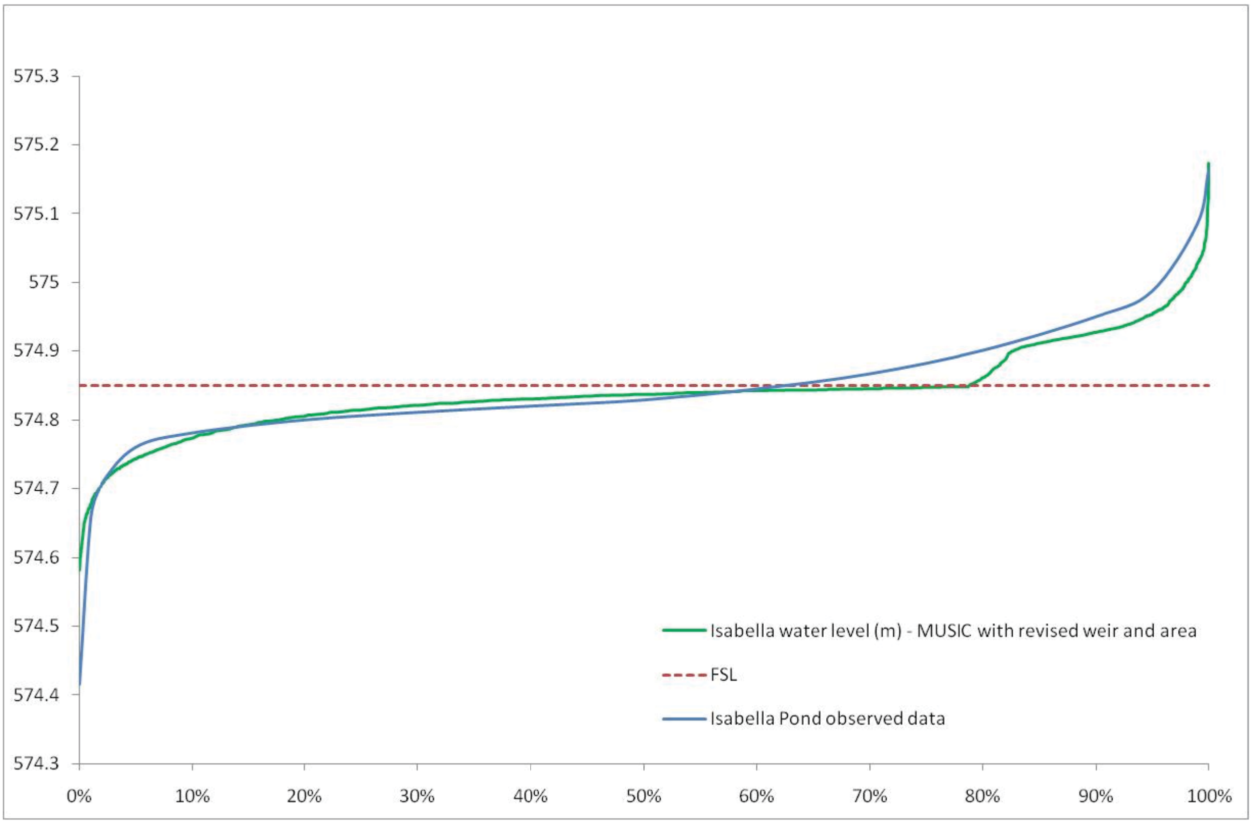


Figure 18 **Water level for Isabella Pond**

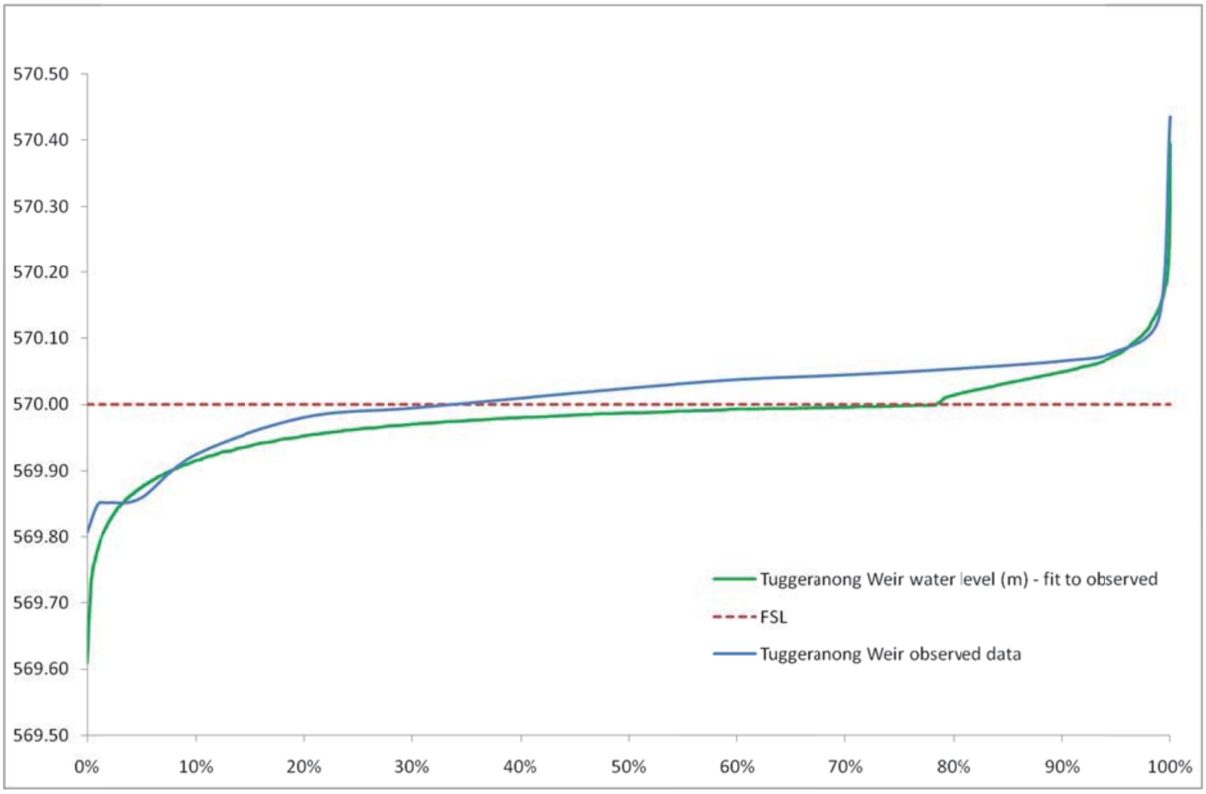


Figure 19 **Water level for Tuggeranong Weir**

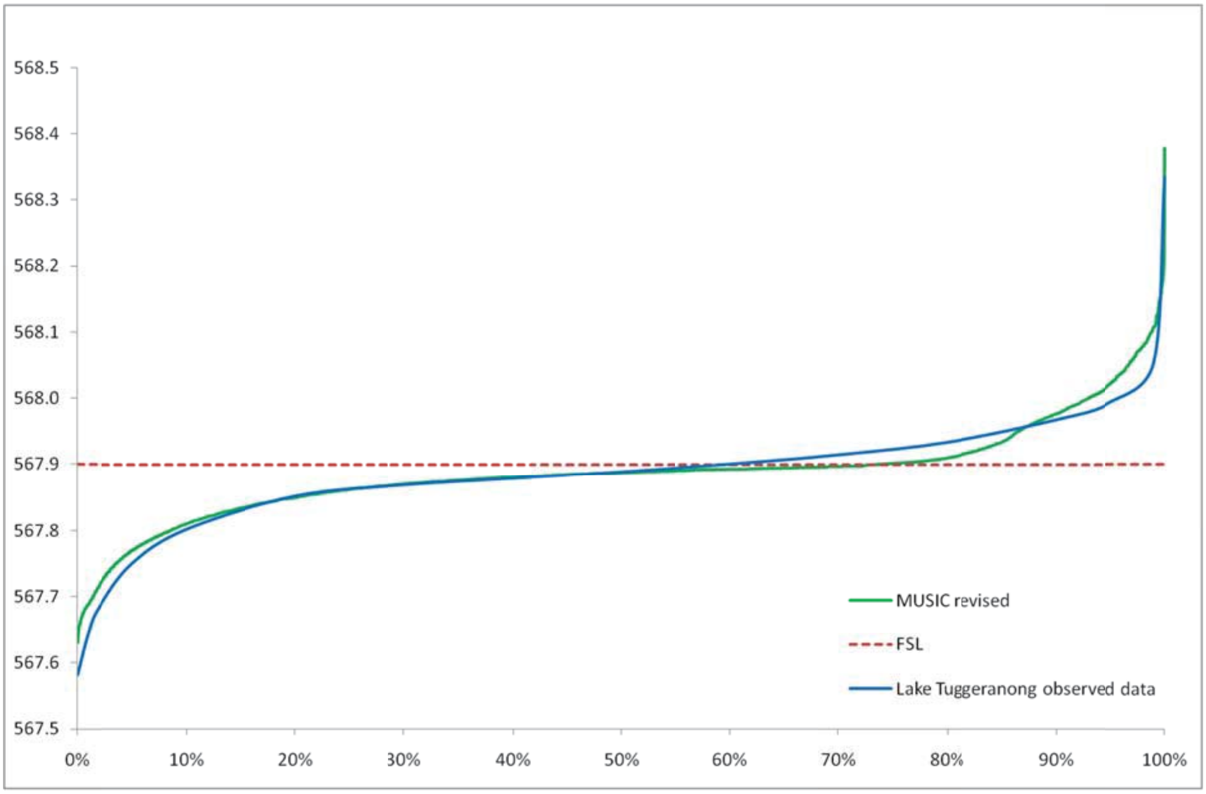


Figure 20 **Water level for Lake Tuggeranong**

The results of the calibrations in terms of input parameters for the models are summarised below.

Storage	**Lake Ginninderra**	**Gungahlin Pond**	**Yerrabi Pond**	**Isabella Pond**	**Lake Tuggeranong**	**Tuggeranong Weir**
Area (ha)	4847.8	3034.8	2019.3	4511.9	2002.6	70.1
Reported impervious fraction	22.8%	14.1%	12.3%	19.0%	26.0%	43.0%
GHD A1 proportion*	13%	-	-	40%	40%	-
Impervious fraction for fit to GHD	11%	7%	6%	37%	30%	84%
Impervious fraction for fit to observed	13.7%	8.5%	7.4%	37%	51%	84%
Full supply level (m)	577.29	-	-	574.85	567.9	570
Full supply area (m^2)	970,490	263,800	228,900	67,150	584,484	46,300
Permanent pool volume (m^3)	3,878,639	460,600	665,400	103,708	1,766,777	95,700
Reuse (ML/year)	-	300	-	-	-	-

*While AWBM does not explicitly represent the impervious area, it is loosely represented by the A1 portion of the catchment

1.5.3 Reuse

The next stage of the calibration was to compare the predicted reuse volumes for the spreadsheet relative to the more detailed GHD models. To do this, the assumptions as per the GHD model (i.e. same inflows) were adopted.

The irrigation demands used in the GHD model (supplied by the client) and estimated based on daily evaporation are shown below. It can be seen that the adopted distribution is skewed towards summer demands with the assumption of no winter demands. The adopted demand of 5ML/ha/year is also less than the predicted demand of 10 ML/ha/year. It is assumed that irrigation systems will only be switched on during the dry summer months with no irrigation during winter.

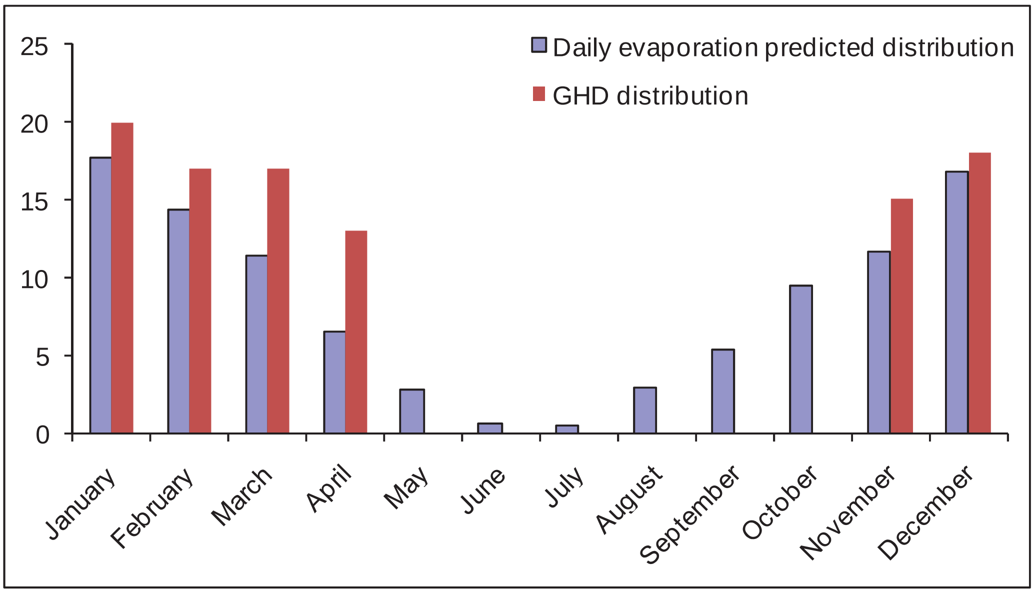


Figure 21 Irrigation distribution - comparing a predicted distribution based on daily evaporation and Ginninderra rainfall with the adopted distribution

The model (as fit to the GHD model) was run for a range of different drawdown levels for reliabilities of 95% and 75%. The potential irrigation demands that could be met were estimated for comparison with the GHD results. The results are quite similar, confirming that the spreadsheet model can produce similar results to the GHD model.

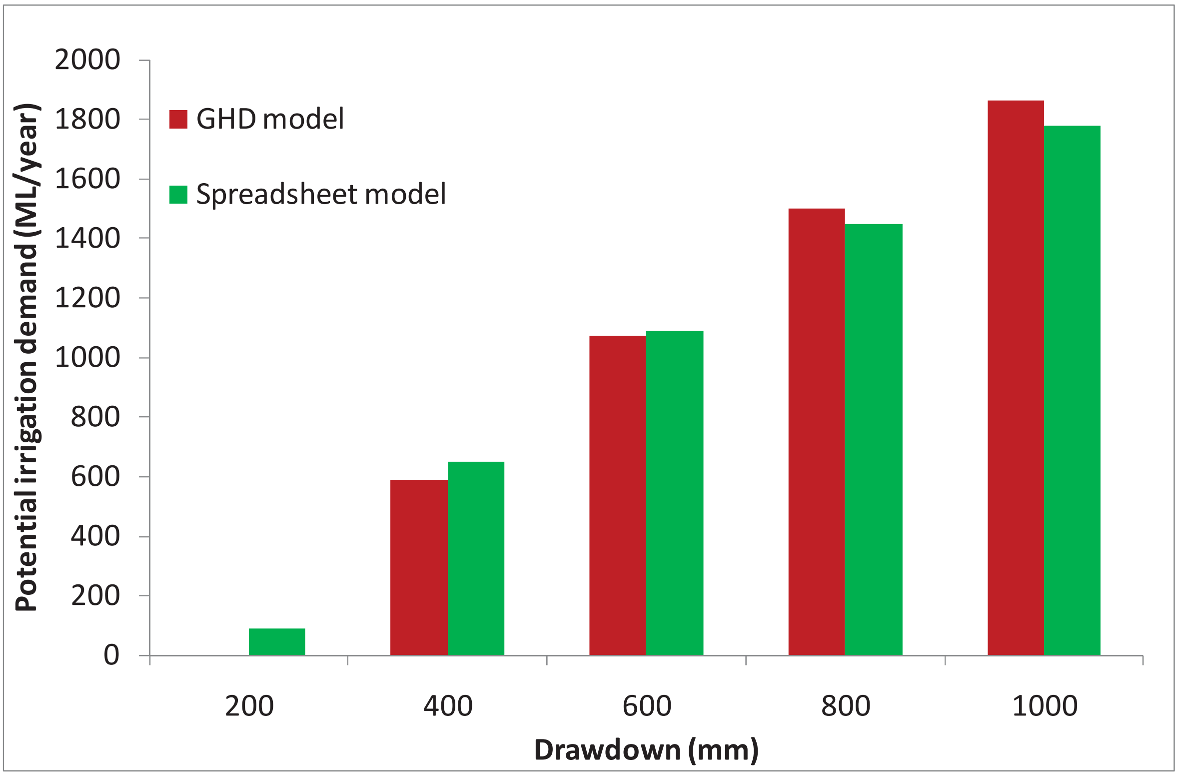


Figure 22 **Predicted irrigation demands that can be met at 75% reliability for Lake Ginninderra**

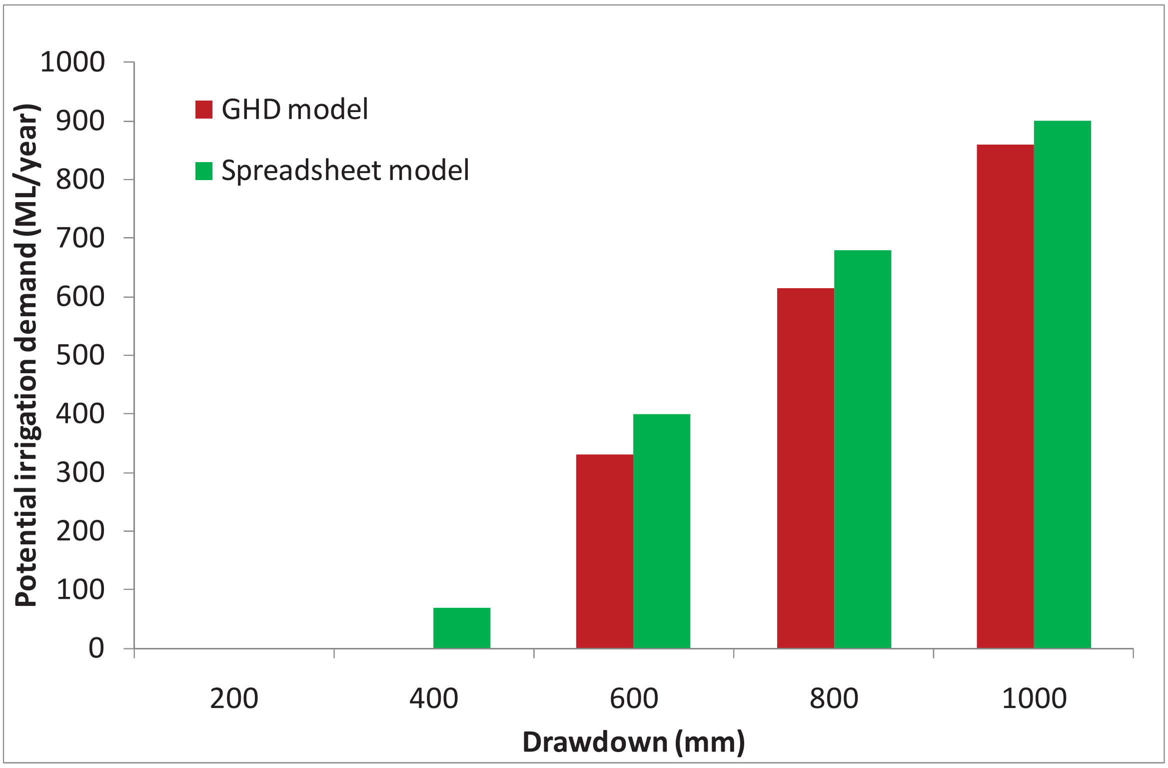


Figure 23 **Predicted irrigation demands that can be met at 95% reliability for Lake Ginninderra**

The model as calibrated to the observed data was also run to predict the potential irrigation demands. The results indicate that somewhat higher volumes may potentially be available.

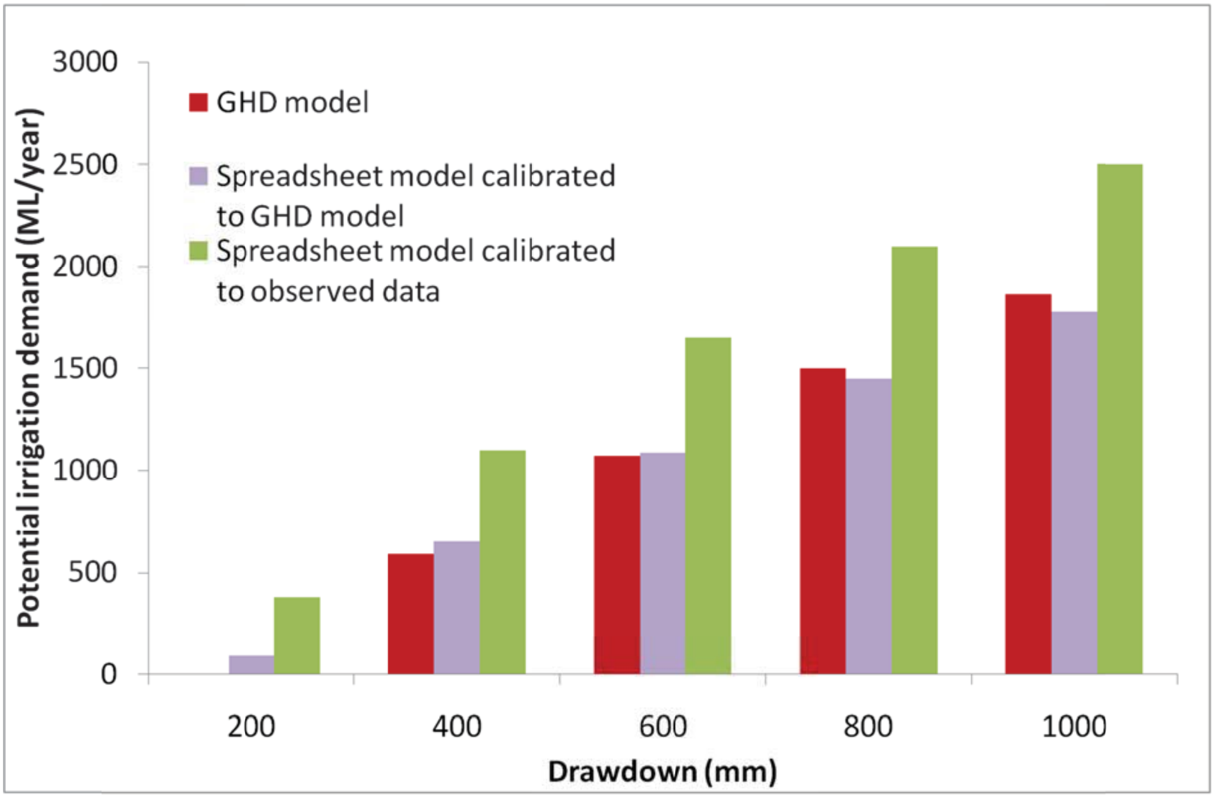


Figure 24 Potential irrigation demands at 75% reliability for Lake Ginninderra

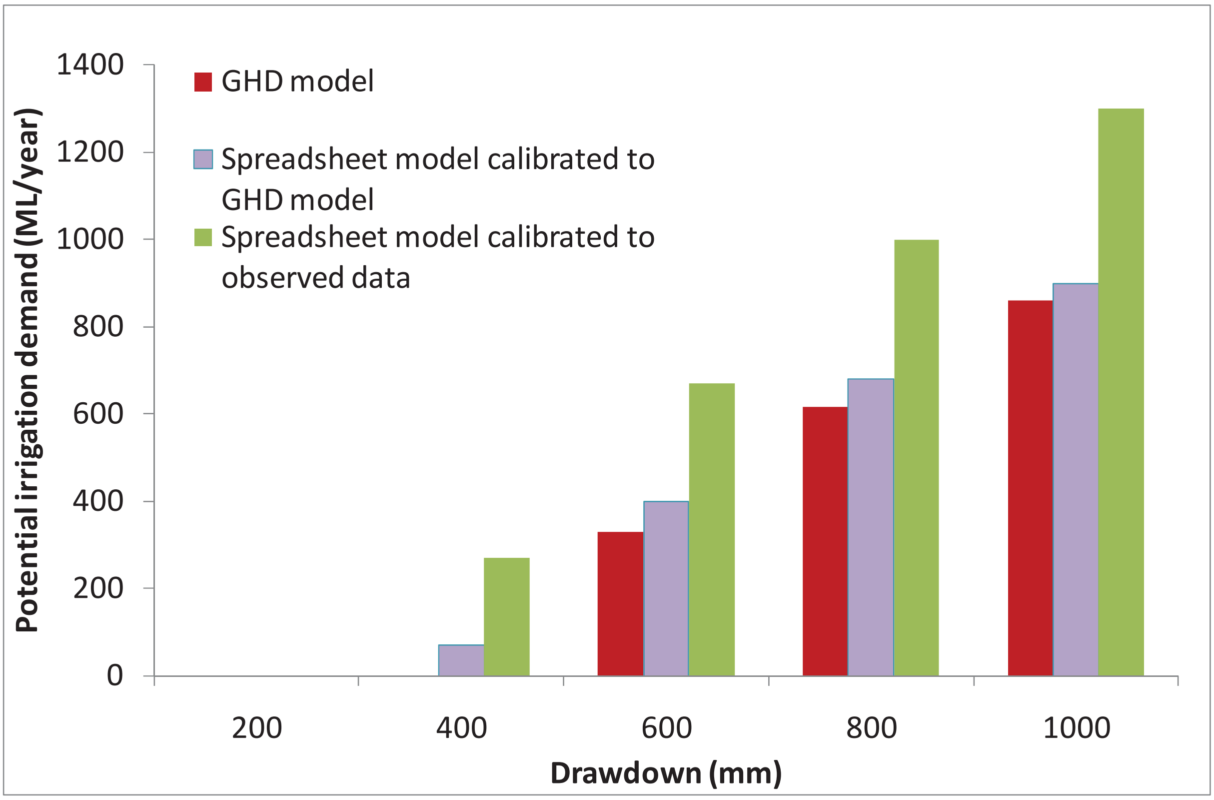


Figure 25 Potential irrigation demands at 95% reliability for Lake Ginninderra

Based on the results obtained, it is clear that a drawdown of at least 400 mm is necessary to achieve significant reuse volumes, while it is also apparent that the pursuit of a higher reliability target of 95% constrains the potential demands that can be met, while accepting a lower reliability would allow a much greater demand volume to be satisfied. For existing storages, the drawdown must be balanced with aesthetic objectives.

The adoption of a drawdown of 400mm and a reliability of 75%-80% would allow substantial reuse 1,000 ML from Lake Ginninderra.

1.5.4 Cost calibration

The costs estimated by the model were compared with the more detailed costs prepared by GHD for the Lake Tuggeranong scheme. Raw costs (before design, contingencies and administration) were compared for the major infrastructure components:

- Pipes
- Balancing storages
- Pumps

The demand clusters were used to ensure that the effects of clustering on estimates were considered. The estimated demands for each cluster were adjusted to match the GHD individual demands as much as practical, leaving only some minor discrepancies in the datasets that could not readily be resolved due to missing data and other issues.

Pipes

The pipes conveying flows from storages to the various demands usually represent the largest proportion of the cost of schemes, except where a new storage is constructed. As such, it is important that the length, and hence the cost of pipes are estimated as accurately as possible, while recognising that some assumptions must be made at the broad planning level.

When a pipe connection between two nodes is made, the direct distance ‘as the crow flies’ is calculated. However, the actual pipe length will clearly be longer. In an earlier study, CSIRO adopted a factor of 1.25 to allow for the difference between the direct and actual distance. [*Maheepala et al.*, 2009]

To test this, an estimate was made of direct distances from Lake Tuggeranong to each of the demands serviced by the proposed Lake Tuggeranong scheme [*GHD*, 2010] (to create a reasonable sample size). The detailed distances from the Lake Tuggeranong design were then compared. Using this method a factor of 1.33 was calculated to minimise the sum of squared differences with the total distances summarised in Figure 26.

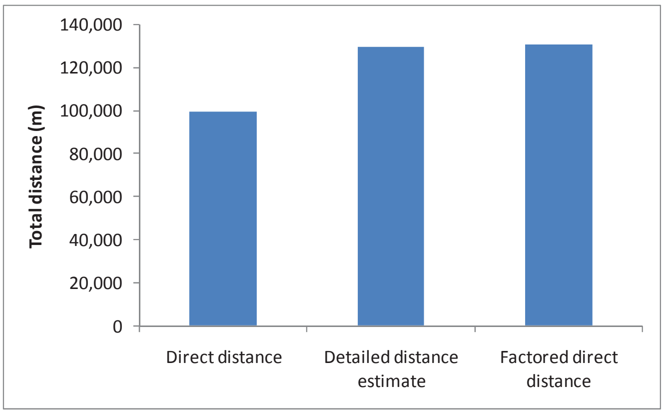


Figure 26 **Comparison of direct and detailed distance estimates and result of factoring direct distance**

The direct distances and the differences were plotted to establish if there was a correlation between the pipe length and the size of the error. The results in Figure 27 indicate there is a correlation as might be expected, although it is relatively weak. Therefore there may be some inaccuracy for schemes with mostly very long or very short connections. This could occur for the first few initial connections to a storage or an area with more dispersed sites where lengths may tend to be longer. As more sites are connected to a scheme, the effects of long and short pipes will tend to be averaged out and the use of a single factor will be reasonable for most cases.

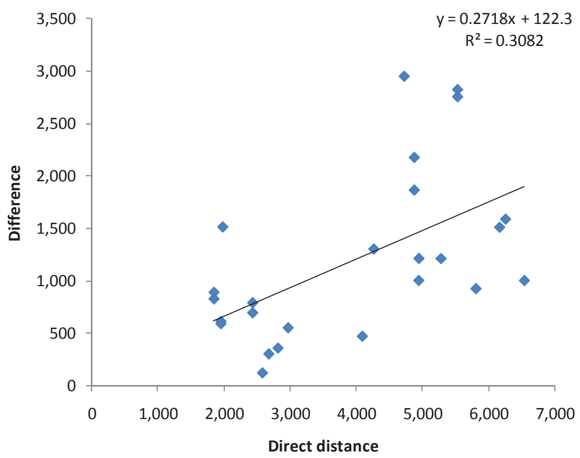


Figure 27 **Direct distance vs. difference (detailed distance - direct distance) showing that there is a weak correlation**

The overall weighted average pipe capacity for the model was found to be within 10% of that estimated for the detailed scheme, given appropriate selection of the friction head factor. This provided confidence in the pipe sizing estimates.

The costs of pipes were based on the cost estimates for the Lake Tuggeranong scheme [*GHD*, 2010], including supply and installation costs. These were compared with the costs from CSIRO and it was noted that the costs for the scheme are significantly higher and increase more rapidly above the 375mm pipe size. The GHD estimates were considered to be more realistic and a similar schedule of pipe sizes and costs were adopted for consistency. The table of pipe sizes and costs are located in the pipes table (in the constants sheet) and can be adjusted if needed.

The costs of crossings (over water supply, sewerage, roads, electricity and other services were estimated by dividing the total crossing costs by the total length of pipe. Crossing costs for the Tuggeranong and Ginninderra schemes were estimated at $110 and $40 per lineal metre of pipe respectively. An average of $75/m was adopted. These costs were excluded for comparison of raw pipe costs between the model and GHD design.

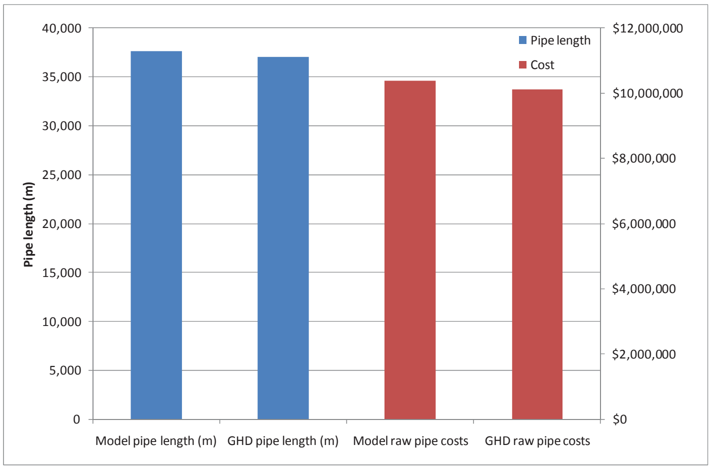


Figure 28 **Comparison of pipe lengths and costs**

Balancing storages

For the purposes of fitting the costs from the model to the Lake Tuggeranong design, it was assumed that 6 days of storage was required, consistent with the GHD assumptions.

Cost curves were fitted to the raw estimates based on tank size and more detailed cost calculations undertaken by GHD. The results indicated very good consistency between the two curves and the fit to the detailed estimates was adopted. It was considered that costs were quite conservative, particularly for small storages, but within the expected range of $400-$600 per kL for larger storages.

The estimated total storage size was slightly higher, being within 2% of the GHD detailed estimate, while costs were 2% lower, with the difference due to the effect of clustering on estimated storage sizes. Given the level of uncertainty in the cost curves these differences were considered insignificant, while it is noted that clustering will tend to result in under-estimation of costs and this should be accounted for if larger clusters were to be used.

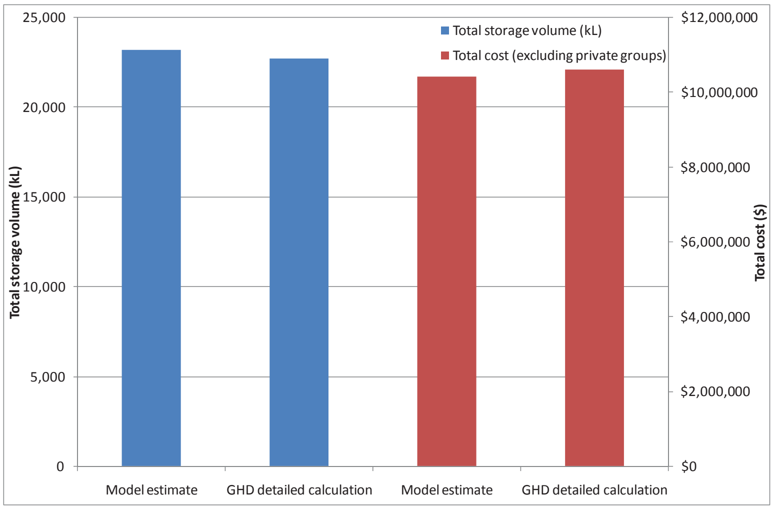


Figure 29 **Comparison of total balancing storage volumes and costs**

Pumps

The pumps were sized based on flow rates for the peak month of January. It was difficult to compare the estimates as the CSIRO equations adopted for use in the model were based on a cost for pumps, whereas the estimates for Lake Tuggeranong, Lake Ginninderra and Flemington Ponds were significantly more detailed and allowed for ancillary infrastructure such as a pump station and road access with significant variation in the resulting estimates. There is a need for a number of estimates for schemes of different sizes to be collated to allow these to be effectively utilised. As the pump and associated infrastructure account for less than 10% of the total cost, the original (CSIRO) equations were retained and it is noted that cost estimates are assumed to not include ancillary infrastructure at this time. While some infrastructure is site specific and difficult to estimate, it is recommended that further work is undertaken to develop a more accurate cost curve for pumps that accounts for the ancillary infrastructure.

Overall costs and outcomes

The overall raw costs were compiled to confirm the fit between the model and design estimates.

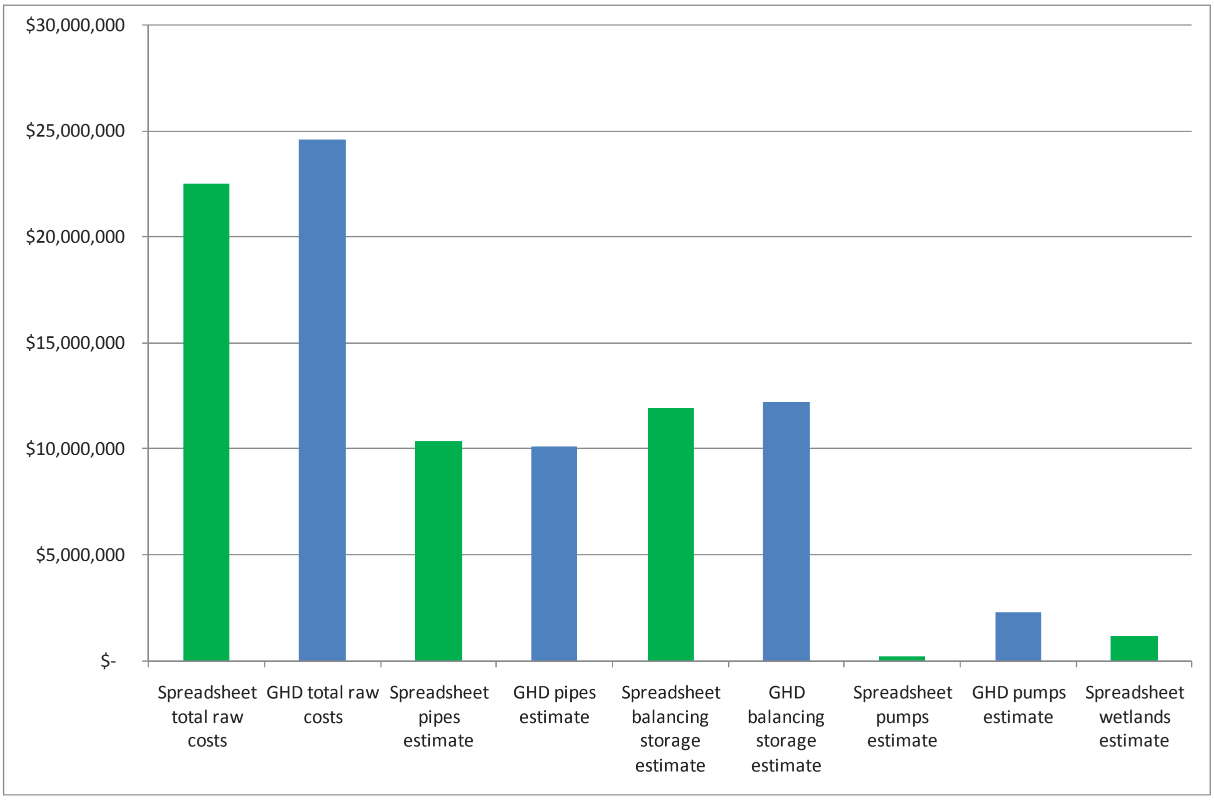


Figure 30 Overall cost estimate for Lake Tuggeranong (raw costs)

*Pump costing by GHD includes ancillary infrastructure
**Wetlands not included in GHD analysis

It was found that the overall costs matched fairly well, with the GHD estimate being about 10% higher due to the additional pump station costs. This can be reduced by adopting a higher pump costing estimate. The cost of wetland treatment for stormwater was not considered by GHD.

The unit rates proposed by GHD for pipes and balancing storages were adopted for consistency. The pipe schedules were slightly adjusted to reduce the pipe sizing required. For the balancing storages, it was considered that dead storage can be assumed to be minimal in storages of this size and assumed to be zero. The number of days of storage was reduced to 3 as 2.3 days of storage are sufficient to ensure sufficient supply when pumping during irrigation days is taken into account and this allows for some flexibility for users. This change from 6 days significantly reduces the cost of balancing storages although this is partly offset by increased pipe costs.

A multiple-barrier treatment train approach is preferred for the treatment of stormwater prior to reuse. This may involve the use of sediment basins, wetlands, bioretention systems and either sub-surface irrigation or UV disinfection prior to use for surface spray irrigation. Treatment is important to ensure sediment concentrations are minimised for effective operation of the system including pumps, UV and spray nozzles, nutrient loads are reduced to minimise leachate into groundwater and discharge into surface runoff and for the protection of human health. It was assumed that wetland and UV disinfection treatment would be required and these costs have been added to the model. It is noted that further data is required for maintenance costs for UV disinfection although these are insignificant relative to other costs and uncertainties.

The model cost procedure provides a reasonable first estimate of costs using simplified equations. The costing algorithms and equations can be refined and improved as more data becomes available. It is important to recognise that while these are sufficient for comparative purposes between schemes, further design and analysis is required to provide more accurate cost estimates for evaluation of any specific scheme.