An investigation of the potential for Rainwater Harvesting at Nepabunna, North Flinders Ranges, South Australia

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by

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AN INVESTIGATION OF THE POTENTIAL FOR RAINWATER HARVESTING AT NEPABUNNA, NORTH FLINDERS RANGES, SOUTH AUSTRALIA

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The availability of rainwater, harvesting techniques and storage media are investigated. Reasonable quantities of water are available, but are interspersed with long droughts. There are problems with silting up of dams and protecting infrastructure from large flows. Long term surface storage is impracticable because of excessive evaporation and resulting salinisation of the resource. Subsurface storage in aquifers appears feasible, but the aquifer storage capabilities have not yet been tested.

INTRODUCTION

The Department for Water Resources South Australia has been contracted by the Division of State Aboriginal Affairs to undertake a feasibility study of the options for rainwater harvesting at the Nepabunna Aboriginal Community (Figure A4).

The work undertaken addresses 3 aspects:

- Water availability
- Surface storage
- Subsurface storage

The main part of this report summarises the findings and recommendations of the study, whilst the detailed technical data are included as appendices.

GENERAL CONSIDERATIONS REGARDING HARVESTING WATER IN AN ARID REGION

There are certain basic requirements for establishing a continuous water supply in an arid area:

1. Long-term water storage must be away from the effects of evaporation. The combination of long droughts between times of recharge and high evaporation rates precludes any long-term storage of water open to the atmosphere. Such storage results in excessive water losses and, more importantly, a gradual and unacceptable increase in the salinity of the reservoir.

2. There must be large volumes of short term capture storage. Because recharge events (rainstorms) are large and widespaced, and because transfer rates from short-term storage (surface dams) to long term storage (underground - aquifer) are relatively slow, provision must be made for large short term storage volumes to minimise spillage.

3. Large volumes of long term storage are required. Because recharge events are widespaced - conceivably up to 30 years - the long term storage (aquifer) capacity must be sufficient to supply for that time (annual consumption rate * 30 years).

WATER AVAILABILITY

RAINFALL AND RUNOFF IN THE NEPABUNNA AREA

Winter rains are light and are only expected to generate runoff from bare rock faces (not common), roads and roofs (Figure A2).

Summer rains can be much heavier, but are also much less frequent. The gaps between rainfall events are measured in years, and historically may be as much as 25 years (1921–1946). These events result in stream flow and the potential for more extensive water capture.

The average annual runoff will yield 7–8ML/a per sq km of catchment area.

For corrugated iron about 0.5 mm of rain is required before runoff commences. For roads and paved surfaces this figure is 1.5–3.0 mm. For natural surfaces - rock outcrop or soil - the requirement is very variable, but generally much higher than either of these figures.

SURFACE STORAGE

As discussed above, the provision of a continuous water supply is not possible unless water is stored away from the effects of evaporation. Therefore this discussion of surface water storage relates to short term storage only, for the time between the capture of water following a rainfall event and its transfer to a long-term underground storage location.

Because of the high variability of rainfall large capture storage is needed to reduce spillage, and large underground storage will be needed to carry over the long periods of drought.

Also, in general, considerations of geomorphology and flood handling indicate that it is likely to be more economic to capture a large percentage of flow from a small creek than to attempt to capture a small percentage from a large creek. The best storage sites for a capture dam will be at the downstream end of a lower gradient, less confined reach of creek at a point where it becomes more confined, for example, by passing through a line of hills.

The filling of storages by sediments is inevitable. Several dams in South Australia have become unusable through this after only a relatively short period (e.g. Pekina, Terowie). Since unconsolidated river sands and gravels, after deposition, are usually suitable materials for forming aquifers, the sustainable design for water systems in arid areas might include design for the gradual conversion of surface water storages to 'aquifer' storages.

The construction of a dam to retain part or all of the flows occurring in a flow channel, for a period while water is withdrawn to ASR storage, must take into account:

- the amount of water to be harvested
- capital and operating costs
- possible damages due to large flows
- loss of volume of capture or diversion dams/weirs due to filling with silt and gravels
- need to retain environmental flows in downstream reaches.

SUBSURFACE STORAGE

The requirement for a large aquifer volume will be a limiting factor on the choice of potential sites and the size of the scheme that can be implemented.

All existing bore sites are potential ASR sites and attention has been focussed on seeking capture points close to the existing bores. While it is possible to establish bores near the most favourable capture points, previous drilling results indicate that aquifers favourable to ASR development are quite rare and may be difficult to find. The risk of failure is much greater with this approach and costs are likely to be higher.

Three known aquifer systems present a favourable opportunity for aquifer storage and recovery.

- The inter-bedded massive limestone bands (fractured limestone aquifer) within the Wilkawillina Limestone (wells 209 and 149) provide the best opportunities for water supply and injection potential.
- The intersected massive cavities or fractures in the Parachilna Formation between 63 and 81 m below ground during the drilling of Well 96 deserve more consideration. They may have the potential for disposal of large volumes of surface water that can be harvested later. Available information suggests that these formations may be relatively fractured and may be present west and south of Nepabunna area. However, this well is unfortunately not available for immediate testing as it was dry and was backfilled.
- The tested and partially evaluated aquifer at the Nepabunna Community (Wells 77, 97 and 101).

Furthermore, there is substantial potential for fractures along major structures, although the number of wells penetrating them is still very small (Wells 223, 224, 225 and 226). These major structures, particularly the inferred faults located southwest and south of Nepabunna, deserve further consideration and more detailed investigation. In the case of the fault located southwest of Nepabunna, investigation for the storage of a large volume of water could be initiated 500 m southwest of well 224 (5 km west of Nepabunna), which is drilled in the Parachilna Formation. It is anticipated that combinations of major structures and cavities could provide long term sustainable injection/extraction rates. However, at this particular location the surface water catchment area is most likely to be small and consequently runoff would be limited.

The requirement for close proximity between the aquifer and the catchment area has not been evaluated in detail. While it is obviously desirable to have a minimal distance between the two, other considerations may make a separation of several kilometres acceptable.

GENERAL CONSIDERATIONS FOR ASR

The ASR scheme involves injection into the aquifer when surface water is available and extraction during dry periods.

Several issues must be addressed, including:

- The turbidity and chemical/bacteriological characteristics of the injected water and their impact on the aquifer, including clogging potential and the need for pre-treatment prior to injection.
- Continuous chemical reactions and their effect on well efficiency and the quality of the recovered water.
- Building of a pressure head.
- Salinity of the ambient groundwater and recovery efficiency during long periods of storage.
- Storage capacity of the aquifer.
- Groundwater flow rates in the aquifer.

Some of these points will be discussed in this report. Others are dependent on currently unavailable data, the acquisition of which is included in the recommendations.

RECOMMENDED SITES FOR FURTHER INVESTIGATION

Of the potential sites suggested above, two are recommended for further investigation at this time. These sites offer distinct and different options for ASR:

1. The site 2 km west of Nepabunna where a suitable aquifer for ASR is believed to exist (Site A).

2. A site at the downstream end of Nepabunna township close to the town boundary (Site B).

SITE A. 2 KM WEST OF NEPABUNNA

Water Capture

The site is on the main Mt. McKinlay Creek towards its upper end, where its catchment area is only 30 km^2. Figure A5 shows the location and the catchment area.

This site is adjacent to Bores no. 149, 165 and 209, which are believed to be suitable for ASR with recharge rates of 10–12 L/s. Bore No. 96, which is about 1 km upstream of these wells, would be beneath the floor of the storage created by the on-stream dams described below.

The long-term runoff at this location is estimated at about 210 ML/a. Figure A6 shows the amounts that can be captured over the full 115 years of runoff, using a range of different sized surface capture storages at this site. Recharge rates (from the dam to the aquifer) of 1 Ml/day (12 L/s) and 3 ML/day (35 L/s) were examined. Curves are shown for the volumes of water lost to spillage and evaporation, and the volume that can be transferred to the aquifer. The figure shows that at low surface capture storages most of the inflow is lost by spill. As the storage increases, the proportion lost by evaporation increases, particularly when the rate of extraction to ASR is low. Thus increasing the capture rate to 3 ML/day may provide significant benefit.

Figure A7 shows the size of surface and aquifer storage required as a multiple of the desired annual supply. This shows that the aquifer storage needs to be about 20 times the annual supply, mainly because of the extended drought between 1907 and 1946. This storage would cater for another similar drought. The surface storage requirement is much lower, ranging from 2 to 5 times the required annual supply depending on whether that annual supply is 10 or 45% of the mean annual flow. It is clear from this that the capacity of the aquifer to store injected water will control the annual supply that can be expected, which in turn, as a percentage of the mean annual flow, will determine the worthwhile size of capture storage.

It should be noted that these graphs, and this discussion, have not taken into account any losses that may occur between the injection of the water into the aquifer and its recovery. Such losses will not be assessable until the aquifers have been tested.

The ratios shown in Figure A7 can be applied to smaller and larger catchments within the general area. The requirement for ASR storage to be equal to or more than 20–25 times the supply sought appears to place a relatively low limit to the scale of feasible schemes.

Figure A8 shows the minimum range of storage and varying storage in the aquifer which is required to carry a supply of 100 ML/a through the full 115 years of the simulation period at this site without any break in supply. It is assumed that leakage is zero and that the aquifer is initially full at its maximum capacity. The maximum capacity to just carry the supply rate through the period was found by trial and error.

Capture Dams and Preliminary Costs

The topography just upstream of the Bore appears to lend itself to the construction of a choice of size of on and off-stream dams and spillways ranging from a minimum capture storage of about 50 Ml up to a maximum of about 1500 ML. The on-stream dams would range from about 4 to 14 m high and have crest lengths of about 150 to 300 m long. In all cases a spillway would be cut through to a parallel creek system so that floodwaters are kept away from the dam. Two possible layouts for on and off-stream dams and spillway are shown on Figure A9a and A9b.

Preliminary costs have been calculated by assuming rates of $10 and $20 per cu.m of dam volume. Volumes have been calculated for a simple dam x-section geometry with crest width of 5 m and side slopes of 1:4.

Figure A10 shows the variation of water capture and the bulk volume of an on-stream dam at this site versus the dam height. The division of the bulk volume by the water capture indicates that the cost of water harvested will rise as the amount diverted rises.

Table 1 gives a preliminary calculation of cost per kL harvested by constructing a range of different dams at this site. These figures do not include treatment and retrieval costs.

Dam Ht m	Dam Bulk Volume m3	Water Harvested ML/a	Water Cost $/kL At $10/m3 and 10% Discount	Water Cost $/kL At $20/m3 and 10% Discount
6.5	22 370	80	0.28	0.56
8.5	44 120	125	0.36	0.72
10.5	75 790	165	0.46	0.92
12.5	118 520	185	0.64	1.28

An extrapolation of the graphs indicates that the cost of water for this and similar sites may plateau out (or even increase again) for lower dams. This is partly due to the fact that, as the size of the dam reduces, an increasingly larger proportion of the total dam bulk volume is required to form a safety surcharge above the spillway level.

Further map analysis will be needed to check for other dam sites conforming to minimum cost criteria. Field investigations will be required to verify the suitability of dam sites, including survey levels and depths to rock, etc.

ASR Potential

Wells 209 and 149 intersected a confined fractured rock aquifer in a steeply dipping limestone band within the Wilkawillina Limestone at this location (Figure B1). Airlift yields of 11–22 L/s were achieved from a depth of 80–120 m, the groundwater ranging from 700–1000 mg/L total dissolved solids. The standing water level was 51 m. Information on this aquifer is sparse.

The potential for rainwater harvesting at this site is totally dependent on the storage capacity of the aquifer intersected by wells 209 and 149. This should be evaluated before any further consideration be given to the scale of water harvesting that might be possible.

Further testing of Wells 209 and 149

Although both wells 209 and 149 are prospective targets to supply the required quantity of groundwater, only well 149 has so far been utilised as a water supply. Neither well has been pump tested. Of the wells drilled to date in the Nepabunna area, these offer the best prospect for long term extraction and injection sustainability

Discharge tests are recommended for both wells 149 and 209. If the results are satisfactory, with high sustainable yields being proved, then injection testing is recommended for well 209. This will be sufficient to trial the characteristics of the aquifer, while not endangering the water supply from well 149 which is currently needed for the community.

There is a problem with a supply of feedstock for the injection testing, if required, and the easiest solution will be to utilise the water from discharge testing. This, in turn, requires a storage facility for this water, amounting to approximately 5 ML, which can be fulfilled by building a test dam or pit to this capacity. Such a dam would hold the water for a week or two while testing (discharge, recovery and injection) is carried out and would also serve as a pilot operation for the larger dam facility required for harvesting rainwater. There is also the possibility that the construction work could be incorporated into final dam structure. It is likely that any pit or dam will require lining with plastic sheeting as the materials are highly porous and no local supply of clay for sealing purposes is known.

Preparatory Work - Dam Building

A dam is to be built, possibly at the off-stream storage site some 1 km upstream of the wells but subject to final surveys. The off-stream site is preferred because this avoids any complications with rainfall before or during the testing period. It is possible, and preferable, that a site closer to the wells might be used, and this might be a pit rather than a dam. In either case the structure will be lined to minimise leakage.

Estimated Cost	$10 000

Stage 1 - Discharge Testing

Discharge Test – Well 209

- Conduct 3 step drawdown tests, each for a duration of 100 minutes.(1.2 l/s, 7 l/sec and 15 l/sec)
- Conduct a constant discharge test for at least 3 days at the maximum yield indicated by the step drawdown tests (optimally 15 l/sec). The discharge water will be stored in the dam
- Monitor the drawdown in well 149, which should be taken off-line during the testing period
- Monitor the effectiveness of the dam - leakage

Estimated cost

Test pumping	
Mobilisation (once only)	$6135
Operations 4 days x 24hrs x $85	$8160
Geophysical logging	
Mobilisation (once only)	$1900
Operations	$500
Water chemical analysis	$1200
Hydrogeological supervision and reporting	$2160
Total costs for Well209, stage 1	*$20 055*

Discharge Test - Well 149

- Conduct 3 step drawdown tests, each for a duration of 100 minutes.(1.2 l/s, 7 l/sec and 15 l/sec)
- Conduct a constant discharge test for at least 3 days at the maximum yield indicated by the step drawdown tests (approximately 15 l/sec). The discharge water will be stored in the dam
- Monitor the drawdown in well 209
- Monitor the effectiveness of the dam

Estimated cost

Test pumping operations	$8160
Geophysical logging	$500
Water chemical analysis	$1200
Hydrogeological supervision and reporting	$2160
Total costs for Well 149, stage 1	*$12 020*

Stage 2 - Assessment of Pressure Head Build-up during Injection

Injection Test - Well 209

Conduct a field injection experiment using Well 209 as an injection well and previously extracted water from surface storage as injection water. This water may have to have a tracer element added to permit subsequent differentiation from native groundwater.

The work at this site involves the following:

- Conduct constant injection testing at the maximum possible pumping rate (possibly 10 l/sec)
- Monitor the pressure build up in Well 209.
- Monitor chemical changes over residence period of time for Well 209
- Conduct recovery efficiency testing for Well 209

- Monitor the pressure build up in Well 149 and the movement of water into it from the injection well.
- Assess the clogging potential.

Estimated cost

Injection operation	$8160
Water chemical analysis	$1200
Hydrogeological supervision and reporting	$2160
Total costs for stage 2	*$11 520*

SITE B. NEPABUNNA TOWNSHIP

Catchment, Runoff and Capture

The runoff from the majority of the Nepabunna township could be directed to a dam site in a gully at the north-east corner of the township. The catchment area and dam site are shown on Figure A11. Once again side spillways could be established through adjacent ridges. The ridge to the north side is higher than that to the east, thus if a larger storage was required, an earth levee may be needed along the east side. A survey of the site will be needed to check these preliminary calculations and enable final design to proceed.

Runoff from the south part of the town which drains to the east along the south side of the road into the adjacent creek should be directed under the road to keep it within the catchment. The calculations assume that this will be done.

The catchment area includes the roofed and paved areas of the township. Inspection of the township map shows 21 houses with roof areas of about 200 m^2 each. An allowance has been made for 50% addition for garages, sheds, workshops etc. Thus the total roofed area is 6300 m^2.

Whilst the roof runoff is collected in household tanks, these are likely to overflow in the large rainfall events. Modelling has assumed 20 kL tanks with water withdrawn at the rate of 0.3 kL/day. Under the modelled condition the tanks only provide 196 kL/a to direct supply, whilst spill is 969 kL/a.

The paved road area has been estimated at 1155 long by 6 m wide i.e. 6930 m^2. Runoff from the roads is shed into unlined swales. If it assumed that these are 2 m wide, with a 5 mm infiltration capacity and 1155 m long, this infiltration loss equates to only 25 kL. To model this loss a 50 kL open dam was assumed to exist which is required to be filled before runoff enters the capture dam. This loss dam is subject to evaporation and would require filling to the level lost by evaporation since last runoff event.

The remainder of the catchment was estimated at 5.5 ha. Much of this consists of trampled earth and this is expected to have a higher runoff than natural catchment surfaces. The rainfall runoff model used provided a mean runoff of 16 mm from this surface over the 40 years of simulation.

Daily rainfall runoff modelling indicated that runoff from the three surfaces were about equal, i.e.

Roofs	969 kL/a spill from tanks
Paved areas	944 kL/a before losses
Earth areas	854 kL/a

Hence the mean runoff before losses from the swales were abstracted equals 3009 kL/a. The application of the swale loss mechanism reduced this to about 2500 kL/a.

Figure A12 shows the performance of different sizes of capture dam, with extraction rates of 1 L/s and 3 L/s. It can be seen that a dam of only about 5 ML will capture nearly 100% of the inflow. The smaller sizes of capture dam and aquifer in relation to the supply sought are due to the greater amount and frequency of runoff at this site.

Costs

Contours are only available for part of the damsite area. Extrapolation of the contours indicates that a 2 m water depth is sufficient to provide this storage as shown in Figure A13. A 3 m high dam, with levee extension on the east side has a bulk volume of 1975 m3. If the placement costs of this dam are again assumed to lie in the range of $10–$20/m^3, the indicative cost of 2.2 ML/a of water produced from the dam at 10% discount would in the range of $0.9 to $1.8/kL, excluding any treatment and retrieval costs.

A smaller dam will probably reduce costs faster than water harvesting efficiency. A smaller dam, at an optimum location might yield 1.5 to 2 ML/a at a cost closer to $0.5 to $1.0 per kL

Permanent storage away from evaporation could be established:

- By ASR, by recharging one (or more) of the existing bores, or new bores drilled for the purpose
- In steel or concrete tanks
- In an earth dam with a plastic/floating cover. This could be an enlarged version of the capture dam.

Since the capture percentage is high, even at low assumptions of extraction rate, recharge of local bores appears to be the cheapest form of long-term storage. Care would, of course, have to be taken to ensure that the quality of the injected water did not detract from the quality of the natural groundwater.

The addition of pipework and pumping costs might double the cost estimates. However, they would still be of the order or less than those incurred in providing water via roofs and raintanks, which under Flinders Ranges conditions would be in the order of $5/kL.

ASR Potential

Available pumping test data indicates that a well yield/recharge rate of 2.5L/s is possible for well 101, particularly under pressure injection. Testing of the injection potential is required before the aquifer characteristics are satisfactorily known.

Stage 1 - Injection Testing - Well 101

Conduct a field injection experiment using Well 101 as an injection well and Well 149 as a source of injection water.

The work at this site involves the following:

- Conduct a constant injection test at 1.2 l/sec for 2–4 days using supply from Well 149 and existing infrastructures.
- Monitor the pressure build up in Well 101
- Monitor the pressure build up in Wells 77, 95 and 97 and the movement of water into them from the injection well.
- Monitor chemical changes over residence period of time for Well 101 (collect water samples)
- Conduct recovery efficiency testing for Well 101.
- Monitor drawdown in extraction Well 149.
- Assess the clogging potential.

Estimated cost

Injection operation	$8160
Water chemical analysis	$1200
Hydrogeological supervision and reporting	$2160
Total for this stage	$11 520
Total Cost for both Sites, Stages 1 and 2	$65 115

SUMMARY AND RECOMMENDATIONS

- Harvesting of rainwater at Nepabunna is only feasible if the effects of evaporation can be minimised, probably by storing the water in an aquifer (ASR).
- Low volume rainwater harvesting by ASR appears to be feasible on an annual basis, at the Nepabunna townsite, subject to the results of injection testing of the aquifer at this location.
- Medium to large volume rainwater harvesting appears to be feasible at a site 2 km west of Nepabunna, subject to the results of extraction and injection testing of the aquifer at this location.
- There are other sites for medium to large volume rainwater harvesting, but the knowledge of aquifers for storage is minimal and can only be improved by an extensive drilling and testing program.
- The next stage of the investigation into rainwater harvesting should be a full study of the capabilities of the aquifer at one or both of the two sites referred to above. Should these aquifers not come up to expectations then the choice lies between abandoning the rainwater harvesting project and finding other aquifers that will fulfil the ASR requirements.
- The total estimated cost of testing wells at both sites is $65 115, assuming that all of the work is done at one time. Otherwise additional mobilisation costs will apply.

If the prospects are regarded as being sufficiently favourable to continue this investigation then the recommended sequence of further work is:

1. Investigate the most effective temporary storage method for 5 Megalitres of water to be extracted from wells 209 and 149, Site 1. This will be either a dam or pits, in either case lined with

plastic to minimise leakage. Build dam or pits. Estimated cost $10 000

2. Run discharge tests on wells 209 and 149, site A, with water being stored in the dam or pits. Estimated cost $32 075. 8 days duration
3. Run injection test on well 101, site B, using water from the pits at site 1. Monitor recovery in wells 209 and 149. Estimated cost $11 520. 4 days duration.
4. Run injection test on well 209, using water from pits. Monitor recovery in well 101. Estimated cost $11 520. 4 days duration.
5. Monitor recovery of well 209.

The geophysical logging of the wells can be done separately, if convenient, or at the same time as the above operations. The cost is included in the above figures.

This sequence of operations will minimise costs by optimising the usage of time. Variation may result in additional mobilisation charges or charges for standby time.

The above results will show more clearly what ASR options are available and which water gathering options will be most efficacious. The testing of dam options would follow.

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Appendix A

Estimation of the availability of surface water in the Nepabunna Area of the Flinders Ranges, SA, with long term storage of captured water to be held in Groundwater Aquifiers.

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Estimation of the Availability of Surface Water in the Nepabunna Area of the Flinders Ranges, SA, With Long Term Storage of Captured Water in Local Aquifers.

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Contents

1. Harvesting Water in an Arid Region

1.1 General

Aridity introduces several features that are not present when harvesting water in more temperate climates. These include:

1. The flow is more infrequent, but flood events, when they occur, are generally larger than those in temperate regions. Droughts are longer and more intense. Because of the high degree of flow variability, large carry-over storage volumes are required if reliable supplies are to be provided from water captured from the infrequent flow events.

2. Evaporation is very high. As a consequence much of the stored water may be lost unless it is stored in such a manner as to reduce the effects of evaporation. This feature again increases the carry over storage required. <u>Unless evaporation can be significantly reduced, water supplies derived from surface flows cannot provide continuous, reliable water supply in arid areas</u>. Methods of reducing evaporation include selection of very deep storages with low surface area to volume ratios, establishing roofs or covers over storages, or providing storage in underground aquifers.

3. Because of the harsh climate, vegetation is sparse and thus land surfaces are exposed to the full force of erosive rainfalls and flows. Thus flood flows carry large quantities of sands, gravels and boulders. As flows reduce, these are deposited in the river beds, only to be set in motion in the next flood. Hence on-stream dams tend to fill with sediments and river beds are very unstable and create large problems when trying to establish engineered flow control structures.

To confirm the infeasibility of providing a supply from a surface storage under arid conditions a simulation has been undertaken for a 500 ML dam situated on a creek with runoff characteristics similar to those in the Nepabunna area. The mean annual inflow was 250 ML/a. A supply from the storage of 20 ML/a has been assumed. Evaporation from the stored water surface is calculated at 0.7 times the pan evaporation recorded at Leigh Creek.

The WaterCress model has been used for all system simulation. A short description of the model is given in Appendix 1.

A volume to surface area relation was assumed which gave a surface area of about 100 000 m^2 (10 Ha) when the dam was full. It can be seen that a loss of 1.5 m of water per year by evaporation will remove water at an initial rate of 150 ML/a. This is much greater than the removal rate to supply and will remain so until the storage is drawn well down. Unlike in temperate areas, rainfall on the surface provides very little reduction to this high rate of loss.

Moreover, whereas withdrawal to supply removes salinity from the storage, withdrawal of water vapour in the form of evaporation leaves salt to accumulate in the storage. Since evaporation losses far outweigh supply withdrawals, consideration of the salt balance for the storage indicates that a halving of the storage during periods of no inflow must result in (almost) a doubling of salinity.

Thus, if a flow event fills the storage with water at about 500 mg/L, the stored water will have reached 2000 mg/L after a period of no inflow when the dam still retains 20–25% of its capacity. At about this level the water becomes unusable.

Figure A1 shows the simulated storage and salinity performance of the surface dam under these conditions over 115 years of modelled operation. It can be seen that supply fails because of insufficient storage, or because salinity rises above usable levels on many occasions, many lasting for more than one year. (The small spikes to 50 mg/L should be ignored).

No combination of enlarged dam or reduced supply can overcome this situation. Only reduced evaporation can increase the reliability of supply.

Reducing evaporation through surface films and covers has not been successful for larger schemes. Artificial recharge of aquifers (or aquifer storage and recovery - ASR) is the ONLY presently available technology which has the potential to provide reliable low cost water supplies from captured surface water at larger rates and volumes. To be successful, however, the recovery of usable water from storage in the aquifer must be more reliable than would exist for a scheme based on surface storage.

If an aquifer (via ASR) is the intended long term storage location, a short term surface storage is still needed for initial capture and temporary holding of the surface flows. This is because the rate of acceptance of water into the aquifer is slow. Whereas inflows from the river to the capture store may be measured in several hundred ML/day, aquifer recharge rates only reach one or two ML/day. Hence many days may still be required to transfer captured water from the temporary holding store to the aquifer store. If the acceptance rate is too low the effects of evaporation on the amount and salinity of the captured water will start to become apparent again.

The size of the capture dam is a function of the variability of the river flow, the proportion of the river flow which is required for supply and the rate of acceptance of flow into the aquifer. Where the required supply is a significant proportion of the average flow, the capture dam volume must increase to enable it to hold the majority of the largest flow events. This is examined in greater detail in Section 5.

1.2 Comparison of On-stream and Off-stream Initial Holding Storage.

As the name implies, an on-stream dam is built across the flow channel. It captures flow by acting as a barrier and creates a storage pool immediately upstream situated directly over the inflow channel.

Unless the dam is sufficiently high, and/or the pool sufficiently large to contain the largest sequence of flows, a spillway must be constructed to bypass flows which overflow the storage capacity. If the dam is in a confined valley the spillway must be located with the dam. The spillway design must then cope with high energy flow situations, since the spilled water must descend from the spill level to the original channel level, usually over a short distance. Dam failure due to inadequate spillway design has the potential to create downstream damages due to the sudden release of large volumes of previously detained water. Large costs are therefore inherent in such spillway design in order to avoid erosion damage to the dam.

If the dam is in a shallow valley, such as exists at the site 2 km west of Nepabunna (see below), a side spillway may be cut through a saddle in the adjacent hills confining the storage, so that the overflow water bypasses the dam, or is spilled into the adjacent valley. Whilst design to prevent erosion may still be needed, any erosion has less ability to affect the stability of the dam, itself. This offers a cheaper alternative for spillway design.

An off-stream storage is created by diverting water from a flow channel via a diversion channel to a holding basin, usually in an adjacent valley. However, if the main valley is wide enough, a holding storage may be constructed within the same valley. A dam or embankment is needed to create the holding storage. An off-stream storage may therefore be more complex since it must usually consist of some sort of diversion structure in the flow channel, a (possible) diversion channel to conduct the water to the off-stream storage and a dam or embankment to form the storage. In general, however, an off-stream storage location is chosen, and the layout arranged, so that water ceases to be diverted once the storage is full - hence a spillway is usually NOT needed for an off-stream dam.

The channel must usually conduct the flow on a continuous downstream gradient and at a sufficient flow rate to harvest a sufficient volume. The diversion channel may be very short, where the holding basin is located close to the main channel - or may be missing altogether where the holding basin is adjacent to the channel. The diversion may also be a tunnel, but these are expensive, especially where they must be long.

An off-stream storage is harder to locate, since they are not feasible in conventional confined valleys. In certain situations a combination of on-stream and off-stream storages is the most feasible arrangement. In such cases dams are created, and water held in two inter-linked storages on the main flow channel and in an adjacent valley.

Off-stream storage may have advantages in two important respects:

First, the diversion of water into the diversion channel may be controlled to select the better quality of flow, particularly in respect to lower salinity or lesser volumes of associated sediments.

Second, flows may be diverted only after sufficient flow has first passed downstream to sustain the needs of the downstream living environment. Such control is not feasible with an on-stream dam, since overflows only take place when the on-stream dam is full. These overflows may only occur at very long intervals, during which time the immediate downstream river reaches are deprived of all the flows they would have experienced if the dam was not present.

2. Rainfall and Runoff in the Nepabunna Area.

Rainfall in the Nepabunna area is generated by two different mechanisms.

In winter (May to October) the area receives rainfall originating from the southern ocean via the succession of frontal rain bands which pass over SE Australia at about weekly intervals. Rainfall delivered by these systems progressively reduces for more inland areas. At Nepabunna these systems are associated with relatively infrequent rain, usually of relatively low intensity and amount. However, because evaporation at this time of the year is low, successive rainfalls may wet the catchment up. Even if this happens, the runoff from natural surfaces is usually only of relatively small amounts.

Comparison of average monthly rainfalls for stations where rainfall has been measured in the Nepabunna area show that Mt Serle receives the highest winter rainfall and that this reduces for locations further east.

In summer the area receives rainfall originating from large tropical depressions or troughs of moist air which are unstable and associated with local thunderstorms. The depressions form in the tropics and may intensify over the northern oceans to become cyclones. When these cross the coast they take different and unpredictable tracks across northern Australia. Away from the coasts the cyclones lose their intensity and revert to large depressions. About one in three of these may affect the Northern Flinders to a larger or smaller degree. These events cover a wide area and are often associated with several days of heavy rainfall. Where thunderstorms are associated with the depressions, rainfalls may be very intense over local areas, and may cause severe local flooding within a scenario of more widespread general flooding.

Summer rainfalls appear to be relatively uniform over the area.

The annual average rainfall at Nepabunna over the period of rainfall recording from 1939 to 1975 was 249 mm. The variability of annual rainfall is high with the minimum rainfall recorded of 71 mm and the maximum of 642 mm, a range of about approximately +/- 3 times from the average.

Figure A2 shows the mean and maximum monthly rainfall during the period of record. This record is relatively short in terms of its high variability. Stations with longer records are at Beltana, Blinman and Arkaroola. These longer records indicate that the period of record at Nepabunna was about 5–7% wetter than for the 125 years from the 1870's to the present.

Figures A3a–d show the relationships between the annual rainfalls measured at Arkaroola (which has a long and relatively complete record) and the records for the surrounding stations at Nepabunna, Mt. Serle, Wertaloona and Balcanoona, during their period of common recording with Arkaroola.

The best fit dotted lines on these graphs indicate that Nepabunna and Mt. Serle do not appear to experience the same frequency of very low annual rainfalls measured at Arkaroola, Wertaloona and Balcanoona. This may be caused by the location of the latter stations in a winter rain shadow.

For the purposes of estimation of runoff and water capture, a composite record made up of daily rainfalls measured at Nepabunna, Mt Serle, Wertaloona, Balcanoona and Arkaroola (in that order of preference) was assembled. This record commenced in 1884.

The WaterCress simulation model has been used to estimate daily runoff from daily rainfall records, including both summer and winter conditions. The model has been calibrated by adjusting its coefficients until it predicts runoffs best matched to those collected in the Mt McKinlay Creek at a site well downstream of Nepabunna and at three sites on Windy/Aroona Creek. The calibration is described in the Appendix.

Use of the model with the Nepabunna rainfall records gives a runoff depth of 7 mm/a from typical natural catchments within the Nepabunna area. Use of the composite record extending over 115 years from 1885 gives a runoff depth of 8 mm/a.

(The slightly greater runoff from a period of record which has a slightly lower average rainfall is due to the greater variability during the extended period which includes the drought years of the 1920's and 1930's and the wet years of the 1970's).

These figures of 7 to 8 mm/a can be used to predict runoff over any sized catchment at an amount equivalent to 7–8 ML/a per sq km of catchment area. Thus it can be seen that the choice of catchment size will dictate the maximum amount of water which can be potentially harvested. However, it must be recognised that these predictions are made for catchments covering relatively large areas which include, and average out the effects of many different surfaces. At the smaller scale, therefore, runoff may be considerably different to these estimates.

Figure A4 shows the annual rainfall and predicted annual runoff (according to the use of the WaterCress model) ranked from smallest to largest. The results show that:

- Two years out of three (i.e rank 75 of 113) the rainfall is less than the average
- Half of the years (below rank 57) have zero runoff, and
- Only in 1 year in 6 (above rank 84) is runoff equal to or larger than the average runoff.

All these confirm the inherently difficult environment for establishing a reliable water supply system using natural catchment flows in the Flinders Ranges and the need for large storages to carry over the long drought periods.

In areas where runoff is low on account of a high infiltration through sandy or gravelly soils, the infiltrated water may be recharging local groundwater and thus may not be lost from the 'usable' equation. If the locations of storage can be intersected by bores, these may provide the basis for reliable local water supplies.

As an alternative to natural catchments, runoff may be collected from individual house roofs, or paved areas within townships. Runoff takes place from roofs and paved surfaces much more readily, and in much greater proportions of the initiating rainfall. In view of the much greater depth and reliability of runoff generated from these areas, their contributions may also be usefully harvested, despite their relatively small total areas.

For corrugated iron in good condition only about 0.5 mm of initial rain is needed to wet the surface before runoff commences. Losses may take place where gutters leak or overflow, but generally roofs are highly efficient at converting even small amounts of rain into runoff.

Roads and paved surfaces require rainfalls of about 1.5 to 3.0 mm to initiate runoff. The depths of runoff depend on the type of surface, smoothness and slope and the existence of potholes, puddles, etc. If the runoff enters a piped stormwater system, further downstream losses are small. If the road surfaces runoff onto adjacent earth or grassed areas, losses will take place until they become saturated.

Within a radius of 3 km from Nepabunna dam sites can be found on water courses with upstream catchments ranging in size from 0 to 100 square kilometres. A generalised 'best estimate' of the runoff that could be intercepted by a single dam/ASR system, based on the estimate of 7 mm/a of runoff, therefore ranges up to 700 ML/a, with about half of this being able to be supplied on a continuous basis assuming that all the problems could be solved.

3. Water Harvesting and Storage.

As discussed above the provision of a continuous water supply is not possible unless water is stored away from the effects of evaporation. Because of the high variability, large capture storages are needed to reduce spillage and large underground storages will be needed to carry over the long periods of drought.

All existing bore sites are potential ASR sites and attention has been focussed on seeking capture points close to the existing bores.

A fundamental consideration which will dictate the size of the scheme will be the amount of storage which can be stored in any one aquifer. If, for example, it assumed that the effective porosity of the aquifer is 10%, then ten times the volume of aquifer is needed than that of the equivalent surface reservoir.

This requirement for a large aquifer volume will become a limiting factor.

Also, in general, considerations of geomorphology and flood handling indicate that it is likely to be more economic to capture a large % of flow from a small creek than to attempt to capture a small % from a large creek. The best storage sites for a capture dam will be at the downstream end of a lower gradient, less confined reach of creek at a point where it becomes more confined, for example, by passing through a line of hills.

The filling of storages by sediments is an inevitability. Several dams in South Australia have become unusable through this after only a relatively short period (e.g. Pekina, Terowie). Since unconsolidated river sands and gravels, after deposition, are usually suitable materials for forming aquifers, the sustainable design for water systems in arid areas might include design for the gradual conversion of surface water storages to 'aquifer' storages.

4. Sources of Surface Water and Choice of System

The construction of a dam to retain part or all of the flows occurring in a flow channel, for a period while water is withdrawn to ASR storage, must take into account:

- the amount and quality of water to be harvested
- capital and operating costs
- possible damages due to large flows
- loss of volume of capture or diversion dams/weirs due to filling with silt and gravels
- compatibility of surface and groundwater qualities to avoid clogging of bores and aquifers
- need to retain environmental flows in downstream reaches.

Several sites for the capture and storage of water in the Nepabunna area have been investigated.

If the size of the schemes finally decided are small, due to limits on the capacity of aquifers, then it may be best to choose capture locations on smaller tributaries. The topography in the Nepabunna area is compatible with small on-stream dams spilling into adjacent valleys. Hence one or more smaller schemes may be better than one large scheme.

The following descriptions are for the two sites which have received greatest attention:

1. The site 2 km west of Nepabunna where a suitable aquifer for ASR is believed to exist.
2. A site at the downstream end of Nepabunna township close to the town boundary.

These have been investigated as a first priority since they introduce considerations spanning a wide range of system sizes. Also, in the case of the first, the existence of a suitable aquifer for ASR is an advantage, whilst in the second, the greater runoff from the townsite is an advantage.

Each is discussed below.

5. Site A - 2 km West of Nepabunna

5.1 Water Capture

This site is adjacent to Bores no. 149, 165 and 209 which are believed to be suitable for ASR, with recharge rates of 10–12 L/s (Figures A5 and B2). Bore No. 96 is about 1 km upstream. It would lie beneath the floor of the storage created by the on-stream dams described below.

The site is on the main Mt. McKinlay Creek towards its upper end, where its catchment area is only 30 km^2. Figure A5 shows the location and the catchment area.

The long term runoff at this location is estimated at about 210 ML/a. Figure A6 shows the amounts which can be captured over the full 115 years of runoff using a range of different sized capture storages at this site. Recharge rates of 1 Ml/day and 3 ML/day were examined.

The figure shows that at low capture storages most of the inflow is lost by spill. As the storage increases, the proportion lost by evaporation increases, particularly when the rate of extraction to ASR is low. Thus increasing the capture rate to 3 ML/day may provide significant benefit.

Under the assumption made about the relation between salinity and flow rate (see Section 7), the salinity of water transferred to aquifer storage was variable, but in general did not exceed 1000 mg/L. A limit of 2000 mg/L was set, but taking a lower value would not significantly effect the amount transferred to storage. Water in the capture dam was assumed to be drained if it exceeded 6000 mg/L. Salinity is not seen as being a major constraint to securing supplies from this site under the expected conditions.

Figure A7 shows the minimum sizes of capture storage and ASR storage (expressed as the number of times they must be larger than the long term level of supply sought from the scheme) versus the level of supply sought (expressed as a percentage of the long term mean annual flow (MAF) at the location from where the supply is to be taken).

These ratios can be applied to smaller and larger catchments within the general area. The requirement for ASR storage to be equal to or more than 20–25 times the supply sought appears to place a relatively low limit to the scale of feasible schemes.

Figure A8 shows the minimum range of storage and varying storage in the aquifer which is required to carry a supply of 100 ML/a through the full 115 years of the simulation period at this site without any break in supply. It is assumed that leakage is zero and that the aquifer is initially full at its maximum capacity. The maximum capacity to just carry the supply rate through the period was found by trial and error.

While the assumption of zero leakage may be unrealistic, natural recharge may also be taking place which will counterbalance any leakage taking place, assuming that the natural recharge is of a usable salinity.

The long period of drought and near drought from 1907 to 1946 is the main cause for the high aquifer capacity required.

5.2 Capture Dams and Preliminary Costs

The topography just upstream of the Bore appears to lend itself to the construction of a choice of size of on and off-stream dams and spillways ranging from a minimum capture storage of about 50 Ml up to a maximum of about 1500 ML. The on-stream dams would range from about 4 to 14 m high and have crest lengths of about 150 to 300 m long. In all cases a spillway would be cut through to a parallel creek system so that flood waters are kept away from the dam. Two possible layouts for on and off-stream dams and spillway are shown on Figure A9a and A9b.

Preliminary costs have been calculated by assuming rates of $10 and $20 per cu.m of dam volume. These have been set much higher than rates used in southern areas of the State (say $3–$5/m3) in order to allow for possible difficult site conditions and also additional costs of all associated works. Dam volumes have been calculated for a simple dam x-section geometry with crest width of 5 m and side slopes of 1:4. More detailed costing may be undertaken after the size and basic form of the scheme become apparent.

Figure A10 shows the variation of water capture and the bulk volume of an on-stream dam at this site versus the dam height. The division of the bulk volume by the water capture indicates that the cost of water harvested will rise as the amount diverted rises. This result is due to the inherent shapes of the curves.

Table 1 gives a preliminary calculation of cost per kL harvested by constructing a range of different dams at this site. These figures should be taken as preliminary order of magnitude estimates only.

Dam Ht m	Dam Bulk Volume m3 (Rock and Earth Fill)	Reservoir Water Holding Storage ML	Water Harvested ML/a	Water Cost $/kL At $10/m3 and 10% Discount	Water Cost $/kL At $20/m3 and 10% Discount
6.5	22 370	265	80	0.28	0.56
8.5	44 120	560	125	0.36	0.72
10.5	75 790	1040	165	0.46	0.92
12.5	118 520	1850	185	0.64	1.28

An extrapolation of the graphs indicates that the cost of water for this and similar sites may plateau out for lower dams and smaller supplies. In fact the costs per kL of supply may start to rise again for smaller schemes since, as the size of the dam reduces, an increasingly larger proportion of the total dam bulk volume is required to form the safety surcharge allowance above the spillway level.

Note that such a relation 'shape' will apply in general for all individual sites, but should not be applied as a predictor between different sites. Because of economies of scale, at their optimal size, small systems, in general, will yield higher costs per kL supplied than larger systems. This can be seen in respect to the scheme described in Section 6.

Further map analysis will be needed to check for other dam sites conforming to minimum cost criteria. Field investigations will be required to verify the suitability of dam sites, including survey levels and depths to rock, etc.

6. Site B - Nepabunna Township

6.1 Catchment, Runoff and Capture

If it assumed that each household in the town uses 200 kL/a then total water use will be about 4 ML/a. This is presently supplied from roof runoff and bores.

Excess runoff from the majority of the Nepabunna township could be directed to a dam site in a gully at the north-east corner of the township. The catchment area and dam site are shown on Figure A11. Once again side spillways could be established through adjacent ridges. The ridge to the north side is higher than that to the east, thus if a larger storage was required, an earth levee may be needed along the east side. A survey of the site will be needed to check these preliminary calculations and enable final design to proceed.

Runoff from the south part of the town which drains to the east along the south side of the road into the adjacent creek should be directed under the road to keep it within the catchment. The calculations assume that this will be done.

The catchment area includes the roofed and paved areas of the township. Inspection of the township map shows 21 houses with roof areas of about 200 m^2 each. An allowance has been made for 50% addition for garages, sheds, workshops etc. Thus the total roofed area is 6300 m^2.

Whilst the roof runoff is collected in household tanks, these are likely to overflow in the large rainfall events. Modelling has assumed 20 kL tanks with water withdrawn at the rate of 0.3 kL/day. Under the modelled condition the tanks provide a total of 743 kL/a to direct supply, whilst spill is 472 kL/a.

The paved road area has been estimated at 1155 m long by 6m wide. I.e 6930 m^2. Runoff from the roads is shed into unlined swales. If it assumed that these are 2 m wide, with a 5 mm infiltration capacity and 1155 m long, this infiltration loss equates to only 25 kL. To model this loss a 30 kL open dam was assumed to exist which required to be filled before runoff entered the capture dam. This 'loss' dam was subject to evaporation and would require re-filling to the level lost by evaporation since the last runoff event.

The remainder of the catchment was estimated at 5.5 ha. Much of this consists of trampled earth and this is expected to have a higher runoff than natural catchment surfaces. The rainfall runoff model used provided a mean runoff of 19 mm from this surface over the 40 years of simulation.

Daily rainfall runoff modelling indicated that runoff from the three surfaces were:

- Roofs 470 kL/a spill from tanks
- Paved areas 1220 kL/a before losses
- Earth areas 1055 kL/a

Hence the mean runoff before losses from the swales were abstracted equals 2745 kL/a. The application of the swale loss mechanism reduced this to about 2550 kL/a.

Figure A12 shows the performance of different sizes of capture dam, with extraction rates of 1 L/s and 3 L/s. It can be seen that a dam of only about 5 ML will capture nearly 100% of the inflow. The smaller sizes of capture dam and aquifer in relation to the supply sought are due to the greater amount and frequency of runoff at this site.

6.2 Storage and Costs

Contours are only available for part of the damsite area. Extrapolation of the contours indicates that a 2 m water depth is sufficient to provide this storage as shown in Figure A13. A 3m high dam, with levee extension on the east side has a bulk volume of 1975 m^3. If the placement costs of this dam are again assumed to lie in the range of \$10–\$20/m^3 (in order to make allowance for all costs), the total cost would only be of the order of \$40 000. The indicative cost of 2.2 ML/a of water produced from the dam at 10% discount would in the range of \$0.9 to \$1.8/kL. As per Table 1 (Section 5.2) a smaller dam might provide a lower yield of say 1.5 to 2 ML/a at a cost closer to \$0.5 to \$1.0 per kL.

Permanent storage away from evaporation could be established:

- By ASR, by recharging one (or more) of the existing bores, or new bores drilled for the purposes
- In steel or concrete tanks
- In an earth dam with a roof or plastic/floating cover. This could be an enlarged version of the capture dam.

The cost of testing the existing ASR bores will be about $12 000. The recovery efficiency of water recharged cannot be predicted until these tests are completed. Other costs involved with an ASR scheme are not likely to be large.

Costs of tanks are high; a 1 ML tank, erected with all associated fittings, will cost of the order of $75 000. Floating covers are also expensive. A 6000 m^2 cover for Yunta reservoir was tendered at close to $160 000. A sheet iron roof over a supported frame might be constructed for less, but maintenanace costs might still be large.

Hence recharge of local bores appears to be the cheapest form of long-term storage, assuming that testing provides suitable results. The total scheme cost estimated above would cover ASR costs, but probably not any surface storage alternatives. Even if the costs were doubled, the cost of water supplied would still be of the order or less than those incurred in providing water via roofs and raintanks, which under Flinders Ranges conditions would be in the order of $5/kL.

7. Water Quality

Very little data is available on water quality of rivers in the Flinders Ranges. Two main water quality concerns will centre on:

- the salinity of the harvested water. A too high level will limit use for irrigation
- constituents which may cause precipitation and clogging in the bore during ASR operations, for example iron.

Figure A14 shows salinity levels determined from samples taken at a range of low flows at the Mt McKinlay gauging station. Note that these only cover low flows. The flow rate is shown in mm depth of runoff per day so that its use may be extrapolated to other locations with different catchment areas. (For Big John Creek , with a catchment area of 1010 km^2, 1 mm/day flow rate = 11.6 m^3/s).

If it assumed that the catchments are in salt balance, then the load of salinity introduced in rainfall, plus any salinity produced by solution taking place within the catchment, must equal the load of salinity removed in streamflow. The effect of solution is usually not large relative to that of evaporation and may be initially ignored. The salinity of rainfall in the Flinders Ranges may be in the order of 10–20 mg/L. With 250 mm of annual rainfall and 7 mm of annual runoff, the achievement of a balance must dictate that the mean salinity of runoff should be of the order of 360–720 mg/L. In view of the much higher salinity levels measured at low flow, salinity at high flows must be considerably reduced, if these average values are to be attained in practice. The measured data confirms a reducing trend, but considerable extrapolation is required to cover the highest flow ranges. (The highest daily flow recorded for Big John Creek occurred in January 1974 at 19 000 ML/day. This gives an average flow rate for the day of 220 m^3/s or 19 mm/day).

A fitted relation to the data shown in Figure A14 is:

*Mean daily salinity mg/L = 0.6*10^(3.2–0.16*log10(mean daily flow rate in mm/day))*

The use of this expression in conjunction with the daily rainfall-runoff model gives a long term mean flow weighted salinity flowing into the capture storage of 587 mg/L. The salinity of water transferred to ASR storage at 1 ML/day from a 500 ML capture storage is shown in Figure A15. It can be seen that transfers commence within the range 500 to 1500 mg/L with the higher salinities occurring when the inflow is small (i.e when lower flows form a larger proportion of the total inflow). The increasing salinity of water being transferred can also be seen, as the water held in the capture dam is subjected to evaporation.

The volume weighted mean salinity of water transferred to the ASR bore is calculated at 800 mg/L. This will fall if a higher transfer rate is achieved.

The low salinities indicated by modelling are due to the weighting of the average salinity values towards the values for the higher flow rates. The vast majority of inflow is associated with the infrequent large floods and flow rates (say 5–15 mm/day every 5–6 years). The extrapolated formula predicts these to have low salinities. Moreover, the model assumes complete mixing of the daily inflow with the previous storage, hence even if lower flows with higher salinities enter the capture dam, their volumes are not sufficient to have major influence on the salinity of the water already stored.

The smaller capture storages will have a greater salinity variability and, since they will spill more low salinity water, other things being equal, the average salinity of water supplied might be worse. However, the smaller storages will also provide greater opportunities for salinity management through selective capture and release.

Overall salinity of water captured from natural catchments is expected to not be a problem, provided that salinities up to 1000 mg/L are acceptable.

Salinity of water captured from roofs and paved areas within the town area is likely to be considerably better than that from the natural catchments. A much greater risk exists, however, of spillages of harmful substances which may then be carried into the capture storage. This risk should be reduced by public education and the development of an action program to counter the effects of any spills.

Recent research shows that storage in aquifers can reduce the levels of some undesirable substances which may be found in stormwater, notably pathogenic micro-organisms.

8. Discussion and Recommendations.

1. The investigations have shown that amounts of water from about 1 ML/a to over 300 ML/a (or more) can be captured from flows at locations within a distance of about 3 km of Nepabunna. However, the feasibility of providing reliable and economic supplies from these captured amounts, above about 1–2 ML/a (which can be stored in covered surface tanks), will depend on the ability to develop storage in underground aquifers.

2. System modelling has shown that in the Nepabunna region a capture dam needs to be between 2 to 5 times the desired mean annual supply rate, depending on the percentage that the required supply rate is of the mean annual flow. In general this size will not be a limiting factor, although this ratio is much higher than in more temperate regions and will add cost to the scheme.

3. System modelling shows that the volume of storage within the aquifer must be 20 to 25 times the mean annual supply rate. This is again much larger than is required in temperate regions. Finding a contiguous aquifer with a sufficiently large water storage capacity to warrant the establishment of a scheme may be a limiting factor to the establishment of larger schemes.

4. The aquifer site 2 km west of Nepabunna may be a suitable site, but needs additional testing. Capture dam sites covering a wide range on scheme sizes could be located in this vicinity. IT IS RECOMMENDED that tests proceed, with a small capture dam being originally built, as necessary. This could be progressively enlarged, up to the level proved for the aquifer and required to satisfy any identified demands.

5. About 1.5 to 2.2 ML/a could be captured from the township site by establishing a capture dam in the gully at the north-east corner of the town and recharging local wells, even at ASR rates as low as 1 L/s. The capture dam need only be 3 ML volume, or less. This would provide a low cost addition of about 30–40% to the town water system. Costs would be comparable, but probably less than for existing schemes. It is recommended that the survey of this site is completed and a system designed. Recharge of water captured from this site via a temporary dam would enable tests to be undertaken on the suitability of this water for recharge.

6. In general, the costs of supplying water will be similar across the general range of scales and sites, but may be reduced by finding locations where dam (and other) costs can be minimised. The initial outlay on capital works for the town system may be as small as $50–75 000.

7. Salinity effects should not place unmanageable limits on system design or operation. However, this conclusion is based on unproven assumptions. Samples of water should be taken from flows in the rivers and creeks, and tested, to confirm the assumptions.

8. Sedimentation of the capture dam will take place with an on-stream storage being more vulnerable than an off-stream storage. Sedimentation may be reduced by catchment management techniques. A sediment management and operation strategy should be developed as part of the system final decision and design. Experience with the Aroona reservoir should be investigated and applied as appropriate.

APPENDIX 1 WATERCRESS WATER SYSTEM SIMULATION MODEL

APPENDIX

Introduction

Flow gauging stations were established at several locations in the Flinders Ranges during the early 1970's to assist with estimation of flows for water supplies and flood designs. One of these was on Mt McKinlay Creek downstream of Nepabunna.

First estimates of flow within the Nepabunna area can be made by assuming that these would be similar, on a per unit catchment area basis, to those measured at the downstream gauging location.

The gauging station was destroyed in a large flood in 1989. The 16 year period of record is insufficient to make reliable predictions of long term performance of (say) capture dams established on the creek or its tributaries.

While flow records only span 16 years, records of rainfall, which produce the flow, go back over 100 years in many locations. Hence it is feasible and desirable to establish a model of the processes linking rainfall and flow within the area, to calibrate this model during the period of flow measurement and to then use this model to predict flows over the full period for which rainfall has been recorded.

This process of choosing a rainfall-runoff model, calibrating it over a period during which records of rainfall and flow have been measured at a particular location, and then using the model to predict flow from rainfall records over a different period and even at a different location utilises standard techniques developed in hydrology.

Summary of Process and Results

The calibration of a daily rainfall-runoff model against the Mt McKinlay flow record (Gauging Stn. No. AW004508) proved to be uncertain since:

- No rainfall records existed within the catchment during the period of flow gauging.
- The gauging station has a poor recorded depth to measured flow rating, which had to be guestimated above low to medium flows.
- The catchment is very large and different parts are likely to have different rainfall to runoff relations. It is believed that large losses may also occur within the river channels. Thus the processes taking place may be more complex than those in temperate regions.

Because of these difficulties it is unlikely that a good calibration would be possible. With this in mind, the main aim for calibration has been to reasonably represent flow occurrences, and hence reasonably reproduce the duration of dry periods between flow events.

To check ion the transposability of the calibration and improve its accuracy, calibration and comparison of results was extended to 4 adjacent gauging stations at:

- Windy and Emu Creeks at Aroona Dam (AW510500)
- Emu Creek (AW510511)
- Windy Creek (AW510510)
- Windy Creek at Maynards Well (AW510507)

The calibration derived for the gauging station on Mt McKinlay Creek has been used to estimate the frequency and amount of runoff occurring within the vicinity of the Nepabunna Township. This is located in the upper reaches of the Creek where catchments have areas up to about 100 km sq.

Calibration of Mt McKinlay Creek AW004508

Mt McKinlay Creek (also known as Big John Creek) has a 1000 square kilometre catchment at the gauging station. Figure A1 shows the boundary of the catchment and the location of rainfall gauging stations closest to the catchment. All rainfalls are recorded at the daily interval.

During the measuring period (1973 to 1989) rainfalls were not measured at Nepabunna, hence the four closest stations were used for the calibration. These were located at Mt Serle (Met Stn 017 035), Wertaloona (017 052), Balcanoona (017 010) and Narrina (017 041).

For calibration the catchment was divided in four approximately equal areas;

Rainfall Station	Catchment Area
017 035	23 600 ha
017 052	18 000 ha
017 010	29 600 ha
017 041	28 800 ha

During the gauging period, 25 significant flow events (> 50 ML total) occurred. Large events (> 5000 ML) occurred 10 times, five of these in 1974.

The representation of the processes linking rainfall to runoff were assumed to be best described by the WC-1 model. This mode is an adaption of the Hydrolog model widely used for this purpose in Australia. The WC-1 model has been found to have minor advantages under the conditions experienced in South Australia.

The WC-1 model is incorporated into a water systems model WaterCress. WaterCress enables flows from several catchment to be merged. It also enables the user to assemble water systems involving reservoirs, aquifer storage, water demands, etc. and to assess the performance under different assumptions of component sizes and rates, etc.

The aim of calibration of the WC-1 model against the Mt McKinlay flow records was to get the model to reproduce as closely as possible:

- the total volumes of flow over the duration of the rainfall events, and
- the duration of zero (or low) flow periods between the events - as recorded for the gauging station.

The WaterCress model layout used for the structuring of flows within the sub-catchments upstream of the gauging system is shown in Figure A2.

The calibration assumed the same WC1 model parameters for each of the sub-catchments (but using the different rainfall inputs for each sub-catchment). The WaterCress model combined the sub-catchment estimates and enabled the modelling of channel losses via the interpolation of the 'ghost' reservoirs represented by the nodes 9, 10 and 11. The nodes 6, 7 and 8 are used to model flow attenuation. The rainfall-runoff model requires the choice of 10 parameters. Experience shows that only 4 of these account for 95% of the accuracy of prediction. Several parameters were set to zero since it was decided to ignore groundwater baseflow estimation, in view of its insignificance in the record.

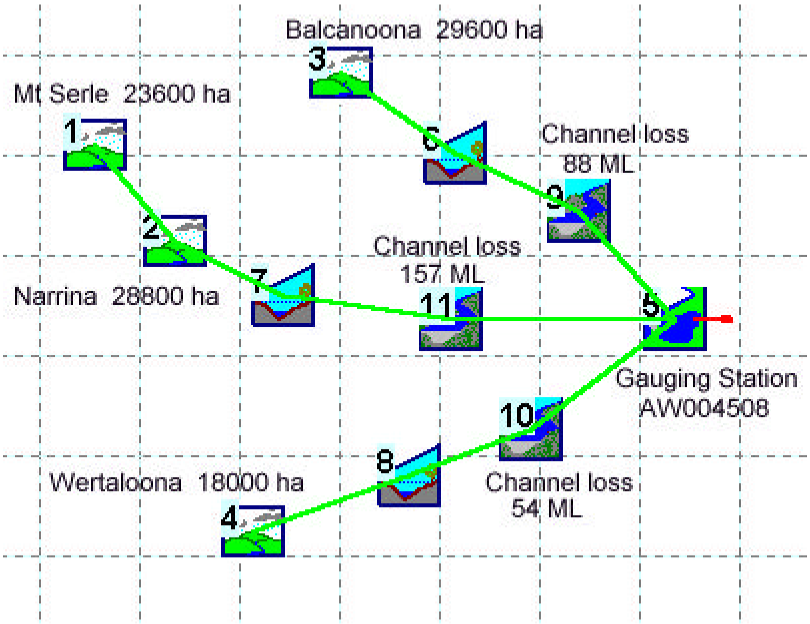


Figure 1 *WaterCress layout for Big John Creek*

The best calibration fit was found with the main parameters of the WC-1 model set at:

Median soil moisture MSM = 90 Interception Storage IS = 40
Catchment Distribution CD = 55 Soil Pan Factor PF = 0.68

These parameters were not the best fit for Mt McKinlay Creek, but were considered the best overall fit with consideration of the nearby catchments.

Parameters also had to be chosen for the sizes and rates associated with the channel attenuation and loss nodes. Due to limited and poor gauging data they could not be calibrated with any certainty. Channel attenuation was set with a volume = 0.0084 times the catchment area. This seemed to provide a reasonable estimate of the flow recessions, however there were numerous examples where hydrograph recessions could not be reproduced according to the records. Many of these did not appear to follow hydrology 'logic' and it is likely that there is error in the streamflow record in the low-flow area.

The instream reservoir nodes simulate the storage and removal by evapotranspiration of flows in gravel beds. They therefore remove the very small flows generated by the WC-1 models. A storage volume equivalent to 0.3 mm of the upstream catchment area was assumed. As most of the flow occurred as large events during a few days per year, this storage had only a minimal effect on most of the annual totals. However, in wetter years, with more frequent smaller events more significant volumes were removed. More detailed investigation was not attempted as there is insufficient data to confirm the results.

Modelling the catchment from 1973 to 1989 required a fit to 57 runoff 'events'. The largest 36 of these (flow > 50 ML) are summarised in table 1.

Comparison of modelled to recorded data yielded an R^2 of 0.82 for annual data and 0.62 for monthly data. The matches between predicted and recorded annual and monthly flows and flow exceedences days comparison are shown in figure 2, 3, 4 & 5.

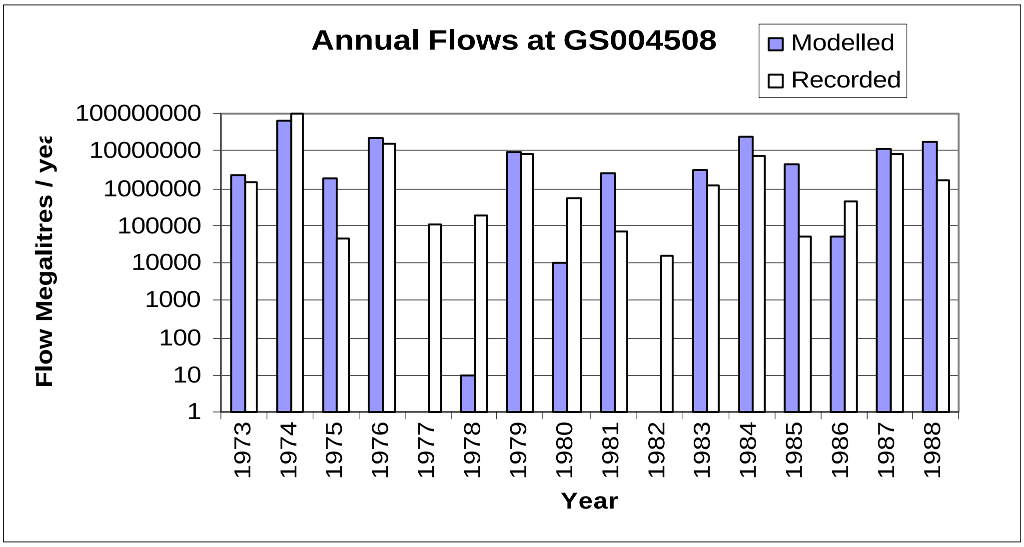


Figure 2 *Annual flows at Big John Creek*

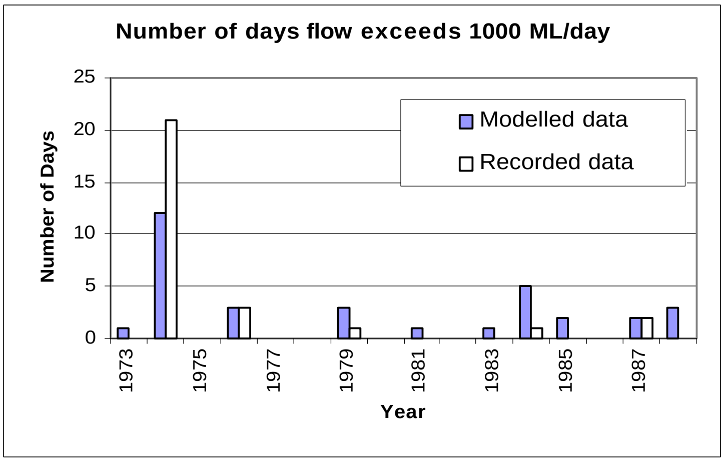


Figure 3 *Number of days flow exceeds 1000 ML/day at Big John Creek*

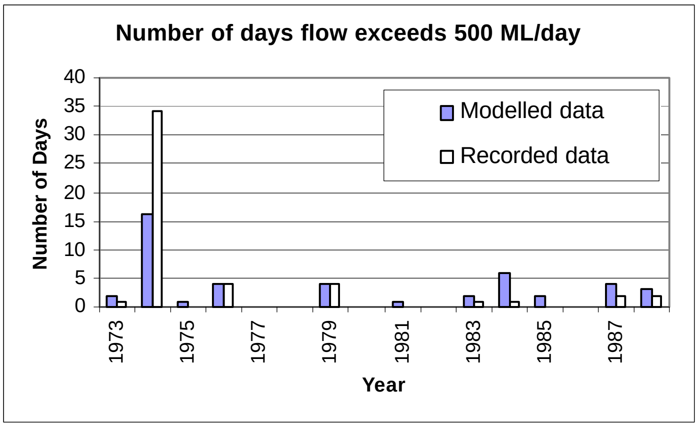


Figure 4 *Number of days flow exceeds 500 ML/day at Big John*

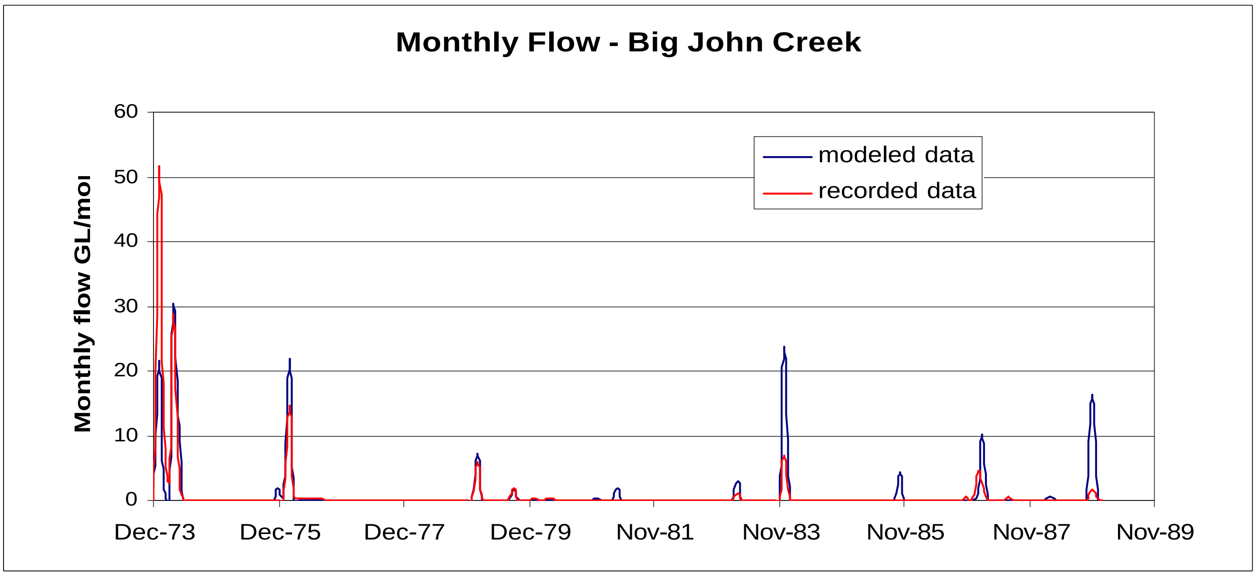


Figure 5 *Monthly Flow Comparison at Big John Creek*

For the period 1973 to 1989 the mean annual runoff from Big John was 9.53 gigalitres / year or 9.4 mm. The median runoff for the same period was 0.6 mm and the 25% percentile 8 mm. The mean annual rainfall for the same period was 280 mm. Mean annual runoff was therefore 3.4% of the annual rainfall.

Date of start of Event	Recorded flow ML total over event	Quality Code	Modelled flow ML total over event
18 Oct 73	156	good	0
26 Oct 73	1019	Exceed rating	0
30 Dec 73	140	Exceed rating	2236
1 Jan 74	5.4	good	515
14 Jan 74	11 435	Exceed rating	836
28 Jan 74	47 347	Exceed rating	21 842
8 Apr 74	12 506	Exceed rating	18 292
19 Apr 74	6750	Exceed rating	4020
22 Apr 74	8475	Exceed rating	7986
9 May 74	0	No recording	9148
13 Dec 75	30	Good	1617
28 Dec 75	6	Estimate	242
12 Jan 76	8	Estimate	601
8 Feb 76	14 521	Exceed Rating	21 838
22 Jan 78	67	Exceed Rating	0
26 Oct 78	83	Exceed Rating	0
20 Jan 79	86	Exceed Rating	0
20 Feb 79	5960	Exceed Rating	7104
27 Sep 79	1765	Exceed Rating	1843
3 Jan 80	372	Exceed Rating	0
23 Apr 80	152	Exceed Rating	0
29 Jan 81	0	Estimate	380
22 May 81	1	Good	1978
28 Nov 81	57	Exceed Rating	0
3 Apr 83	1190	Missing data	3016
13 Jan 84	7019	Missing data	20 332
26 Jan 84	0	No Data	3274
4 Nov 85	46	Exceed Rating	4285
3 Dec 86	402	Exceed Rating	0
19 Feb 87	443	Exceed Rating	296
28 Feb 87	6690	Exceed Rating	10 946
27 Aug 87	391	Exceed Rating	0
31 Mar 88	1	Good	411
25 Dec 88	1745	Exceed Rating	16 349
8 Mar 89	2469	Exceed Rating	0
13 Mar 89	35 922	Station Lost	117 095

Table 1 *Runoff events on Big John Creek*

For the top 36 high flow events shown on table 1 the assessment of the inconsistencies is as follows:

- **1973**: On 18/10/73 a daily maximum of 19 mm was recorded at Narrina which is not enough to trigger an event in the RF/RO model. On 26/10/73 Wertaloona recorded 32 mm and the other stations considerably less. In both of the above dates it is likely that high intensity storms initiated the runoff events recorded. The model could not predict the results based on the low daily rainfalls. On 30/12/73 rainfall ranged from 68 mm at Narrina to less than 32 mm at the other 3 sites. The high rainfall at one site, this well outside the catchment indicates that the rainfall was closer to the 32 mm within the catchment. Hence the model overestimated the event.

- **1975**: On 15/12/75 82 mm was recorded at Mt Serle which is out of proportion to the 20 mm recorded elsewhere. This, assuming it is a correct reading, was likely a local storm and may not have occurred over a large area of catchment.
- **1976**: Rainfall of 43 mm recorded at Narrina on 11/1/76 led to the prediction of runoff occurring. The gauging station provided only an estimate and Windy Creek at Maynards only showed a small flow. It is presumed that this event did not occur.
- **1981**: On 29/1/81 the gauging records show only an 'estimate' of flow of zero yet the model predicted 380ML. The gauging station at Maynards on Windy Creek indicated significant flow for this period. The highest rainfalls were recorded at Narrina and Mt Serle (28 mm) indicating that the bulk of the rainfall occurred to the north. It is likely that a small flow event occurred for this period. On the 22/5/81 significant rainfall occurred at Balcanoona 53 mm and Wertaloona 37 mm yet little to the west. This is likely a spatial problem with no rainfall gauges in the catchment.
- In **1983** most runoff occurred as one event (on 3/4) A rainfall of 96mm recorded at Mt Serle and missing gauging record indicates the modelled data is correct.
- In **1984** there were significant periods of missing record. High rainfall (on 13/1) occurred at Balcanoona and Narrina (132 mm and 109 mm) but it is seen that the rainfall is highly variable spatially across the catchment. On 26/1 significant rainfall events occurred at Balcanoona & Wertaloona (68 mm and 45 mm) but gauging data missing both at Big John and Windy at Maynards. Data from Aroona Dam however indicates a significant event did occur.
- In **1985** there was just one large rainfall event on 4/11 with 94 mm falling at Balcanoona and 62 mm at Wertaloona. The model responded to this rainfall but no significant flow was recorded at the gauging station. Balcanoona and Wertaloona are located west of the catchment indicating the bulk of the rainfall fell outside the catchment. The gauging station on Windy Creek at Maynards was estimated during this period. The major problem here appears that we do not have rainfall data from stations located within the catchment. This event may have occurred or the rainfall may have fallen too far to the east.
- **1986**: On 3/12/1986 only 18 and 15 mm of rain occurred on consecutive days at Mt Serle with little elsewhere near the Big John Catchment. A more significant event occurred on Windy Creek at Maynard and Leigh Creek indicating that the rainfall fell further north.
- **1988**:on 31/3/88 a peak rainfall of 40 mm occurred at Balcanoona and 31 mm at Wertaloona. These stations are west of the catchment, and far lower events occurred to the south and east. The problem once again is not having rainfall stations within the catchment. Windy Creek at Leigh Creek also did not indicate runoff occurring at this date. High rainfall was recorded over all rainfall stations in the area on 25/12/88 ranging from 130 to 67 mm. The model responded to these high rainfalls to over estimate the result. It is likely given the consistency of the rainfall across all of the gauging stations that this was a widespread and less intense event. To the north the model underestimated the flow that occurred on Windy Creek at Leigh Creek. This indicates that there may be a rating problem and Big John is recording too low.
- In **1989** the modelled flow is far greater as record ceased with the loss of the station during a massive event. The modelled data looks reasonable for this amount of rain.

In summation it can be seen that the model is consistently able to predict the occurrence of the very large runoff events. However, the accuracy of the estimate can be extremely variable. The reason for inaccuracy is most likely the varying intensity of the storms, the poor coverage of the rainfall stations to measure spatially variable rainfall and (to a lesser extent) the poor rating of the gauging station.

The model correctly identified 15 of the 17 high flow events (> 1 gigalitre), underestimating 2 events but predicting two others that were measured with some certainty as not occurring.

In the medium flow range (up to 1 gigalitre) the model was rarely able to pick the timing of the events. On ten occasions the model did not predict a flow that was recorded while conversely it predicted 4 events that didn’t occur. Given the difficulty of predicting such events it can at least be said that the model is conservative with respect to water resource availability. On the basis of these results there may be more frequent events available for water extraction than the model predicts.

The remaining 21 are low flow events estimated by the model but not recorded as occurring at the gauging station. These would have likely occurred during days of high volume but low intensity rainfall. Some of these may have occurred, as on 7 occasions flows were not recorded at the gauging station, but were estimated to be zero, probably on little evidence.

This inaccuracy is not surprising as marginal runoff producing rainfall events (approximately 30–40 mm) depend heavily on their intensity as to whether runoff will occur or not. Large catchments also have significant instream losses and while runoff may be produced in the tributaries, this is not seen at the gauging station. This is likely the case when it comes to estimate the runoff occurring at the Nepabunna station, but without flow data at Nepabunna this cannot be assumed.

Comparison of Mt McKinlay Creek and the other Gauging Station Records.

Comparison of Mt McKinlay records with those from the 4 adjacent gauging stations indicated that Mt McKinlay Creek produces less runoff. As all sites have very poor ratings it is difficult to put much faith in this finding.

Windy Creek at Leigh Creek enable the most accurate calibration, most likely due to the rainfall stations lying within the catchment. Emu Creek appears to be a more efficient catchment in producing runoff, but this may be more to do with poor rating. However, Emu Creek also flows more frequently than all of the other gauges in the vicinity, giving some credibility to this finding.

Kemp (1989) studied peak yields in these catchments (as reported in Desmier, "Rainfall Runoff Relationships in the Spencer Regions of SA") and considered that the only stream gauge with a fair rating was at Aroona Dam. Aroona Dam has a stable spillway, hence it has a more accurate theoretical rating curve.

To examine the relative accuracy of gaugings on Windy and Emu Creeks, a comparison of runoff events was made between these stations and Aroona Dam. The flows used are total flows over the number of days of an event. Figure 6 displays the relationship between these stations

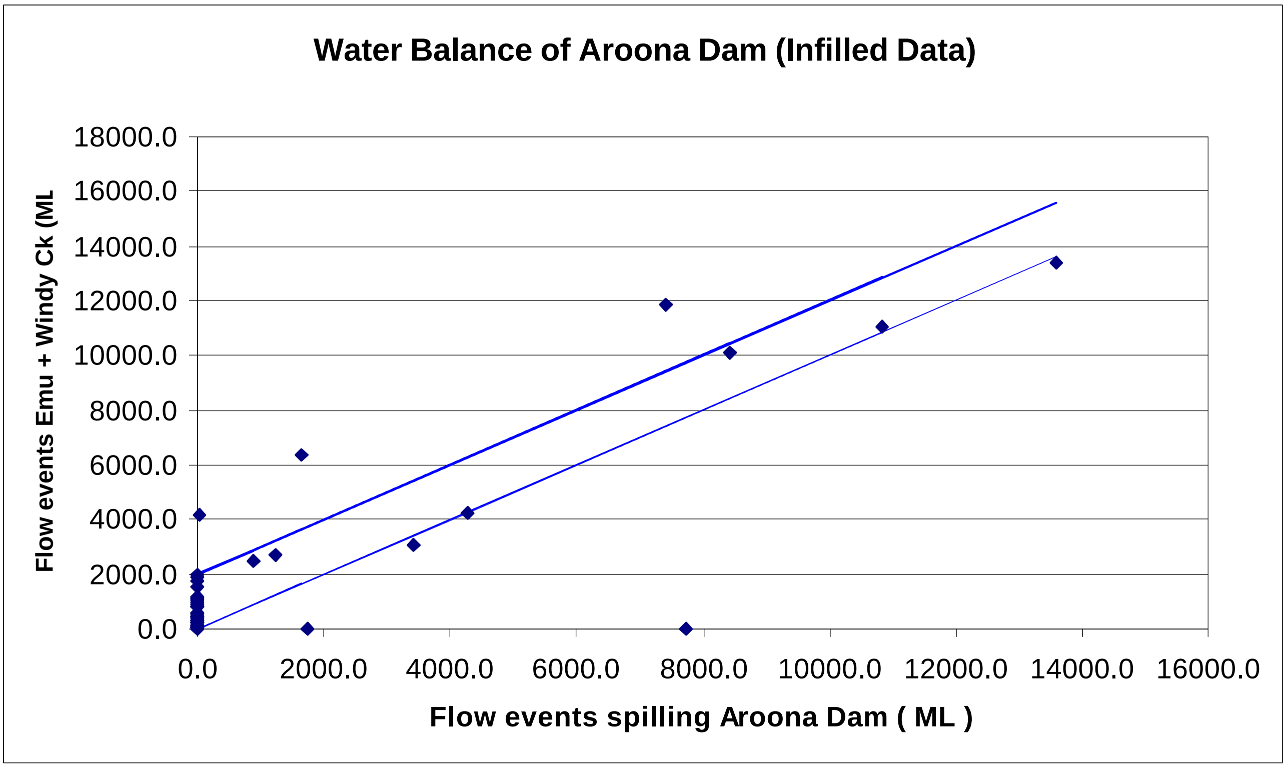


Figure 6 *Comparison of flows on Windy and Emu Creeks*

Missing data was infilled by using a simple best fit linear regression between Windy and Emu Creek gauges. The relationship used was Windy Creek flow = 1.2 times Emu Creek. As Emu Creek has only one half of the catchment area of Windy Creek (448 to 224 km^2) this provides an indication of the greater efficiency of the Emu Creek catchment.

Aroona Dam has a capacity of 2000 ML which is represented by the two parallel lines on figure 6. This is a band of uncertainty since the capacity of the dam at the start of the event is not known. Events falling above the upper of these lines indicate an overestimate of gauging at Windy and Emu Creeks. The points lying on the axis are missing data that could not be infilled.

Whilst too much cannot be drawn from the graph, it does indicate that the recorded flows in Windy and Emu Creeks are in the correct order of accuracy but it does not provide evidence on the efficiency of Emu Creek.

Placing the 'best' model parameters found for the calibration of Windy Creek into the model for Mt McKinlay Creek (using Mt McKinlay Creek rainfall) shows a 15% increases in the predicted estimate for Mt McKinlay flows. Hence the results appear to confirm that the catchments to the west of Mt McKinlay Creek catchment are more efficient at producing runoff. Nepabunna lies to the western margin of the Mt Mckinlay Creek catchment and therefore some greater efficiency might be implied for catchments in the Nepabunna area.

In Summary

The best overall calibration was achieved using the WC1 model with the following parameters:

- Median soil moisture MSM = 90
- Interception Storage IS = 40
- Catchment Distribution CD = 55
- Groundwater Discharge GWD = 0
- Soil moisture discharge = 0
- Soil Pan Factor PF = 0.68
- Fraction Groundwater loss FGL = 0
- Soil wetness multiplier SWM = 0.85
- Groundwater Recharge GW = 0
- Creek loss CL = 0.001

The model can be calibrated to provide a match to the total flow recorded over the period of gauging. However the accuracy of prediction (as a percentage of the recorded value) of individual flow events decreases as their size decreases, i.e. the worst accuracy is associated with rainfall events in the order 0f 20–40 mm which are on the threshold of producing rainfall.

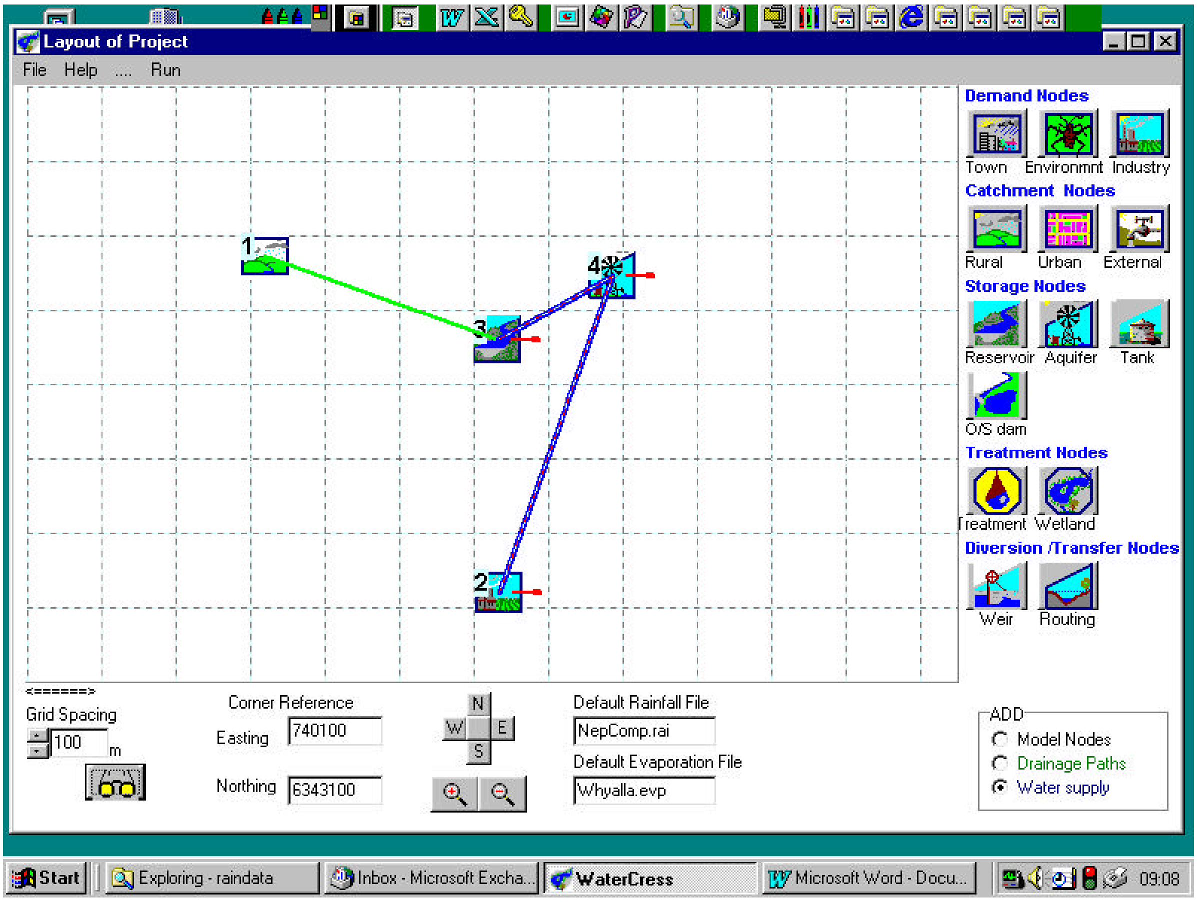
The accuracy of the model increases when applied to catchments with a better coverage of raingauges. Thus much of this low flow inaccuracy is believed to be spurious.

It is likely that significant losses occur instream. The use of 'best' calibration parameters fitted to a large catchment record for predictions of flow on a small catchment may therefore underestimate the volumes of water produced by the small catchment.

There is weak evidence to show that catchments to the west produce more runoff than those to the east. Thus runoff in the Nepabunna area may be slightly greater than that for the remainder of the Mt McKinlay Creek catchment.

In so far as it is best to be conservative in respect to water supply designs, it is recommended that the WC-1 model with the parameters given above is used to predict flows in the Nepabunna area.

APPENDIX 2 STREAM FLOW DATA AND RAINFALL – RUNOFF MODEL CALIBRATION



Water System Depiction

All major components of a water system are shown in the 'menu' which is shown on the right hand of the screen. These can be selected and brought onto the screen and assembled as a system of nodes to represent the desired system. The nodes are linked by flow paths. A green link is a drainage path. The amount of flow passing along a drainage path is unlimited and uncontrolled. A blue link is a pipeline. The amount passing along a pipe is limited by the pipe capacity and is controlled by operational rules. By clicking onto any node or pipeline a series of data entry windows are presented into which all details governing the operation of the node are entered.

Node 1 represents the 30 km sq catchment draining to the bore site 3 km west of Nepabunna. A 115 year record of continuous daily rainfalls has been assembled for the catchment. A rainfall runoff model converts the records of rainfall to a continuous estimate of daily runoff. A salinity-runoff relation assigns a salinity level to each daily flow estimate.

Node 3 represents an on-stream dam which receives the inflow from the catchment. The main variables affecting the performance of the dam within the system shown are its maximum capacity, its area-volume relation and the supply rate taken from the dam. Most attention in the investigation is directed towards the relation between the maximum capacity of the dam, the supply and the amount and salinity of the water supplied.

Node 4 represents the aquifer. The main variables are the initial and maximum capacity of the aquifer, the salinity of the native water, the leakage rate of recharged water and the rate at which transfer can take place into and out of the aquifer. Limits may be set on the maximum salinity of water transferred.

Node 2 represents the demand. This is set at a constant annual rate, but may be distributed according to a seasonal pattern. The main purpose of the system modelling is to determine the reliability of the assembled system under different assumptions of component details in providing supply to satisfy the demand.

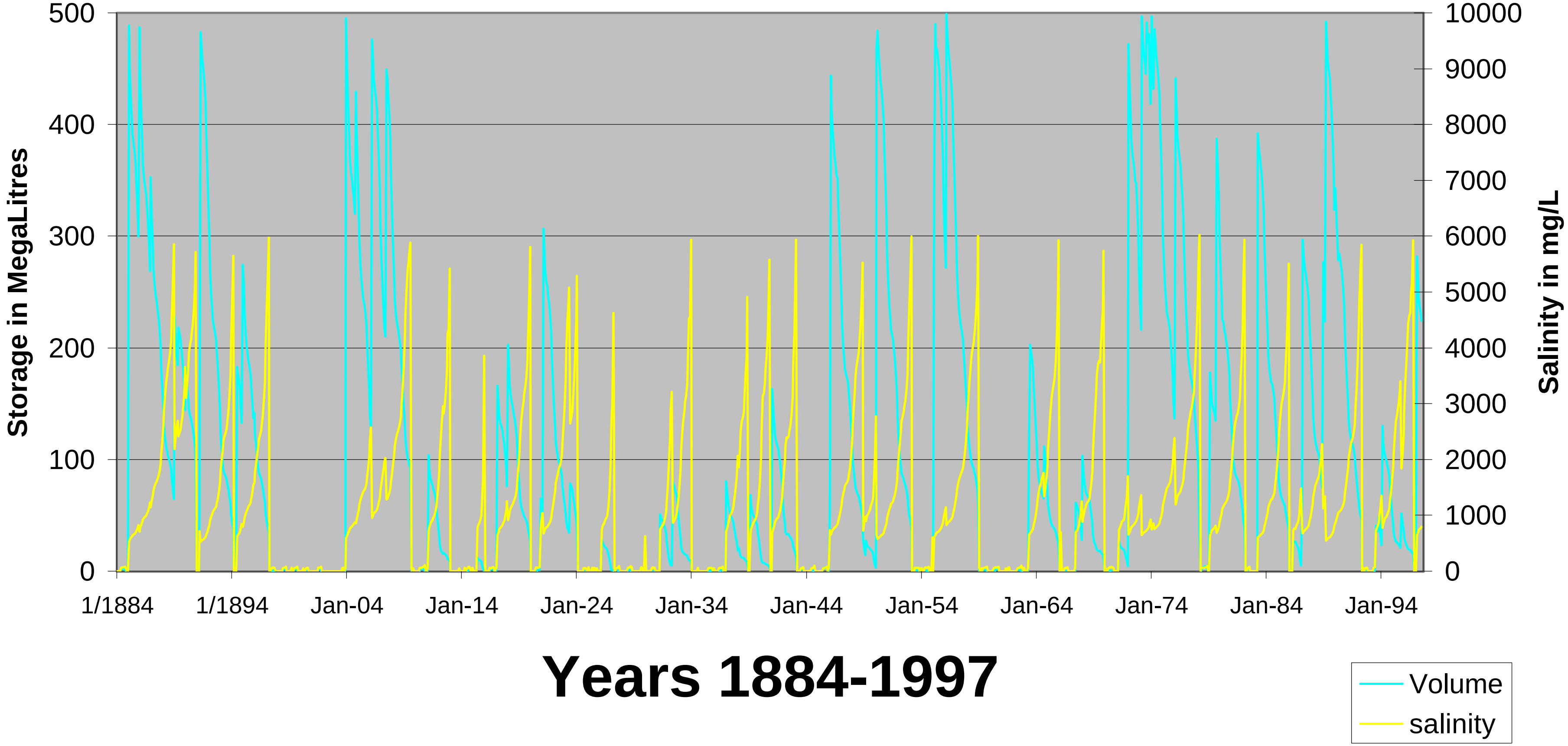


Figure A1 *Simulated storage and salinity of a surface dam over 115 years. The dam is drained when the salinity reaches 6000mg/L.*

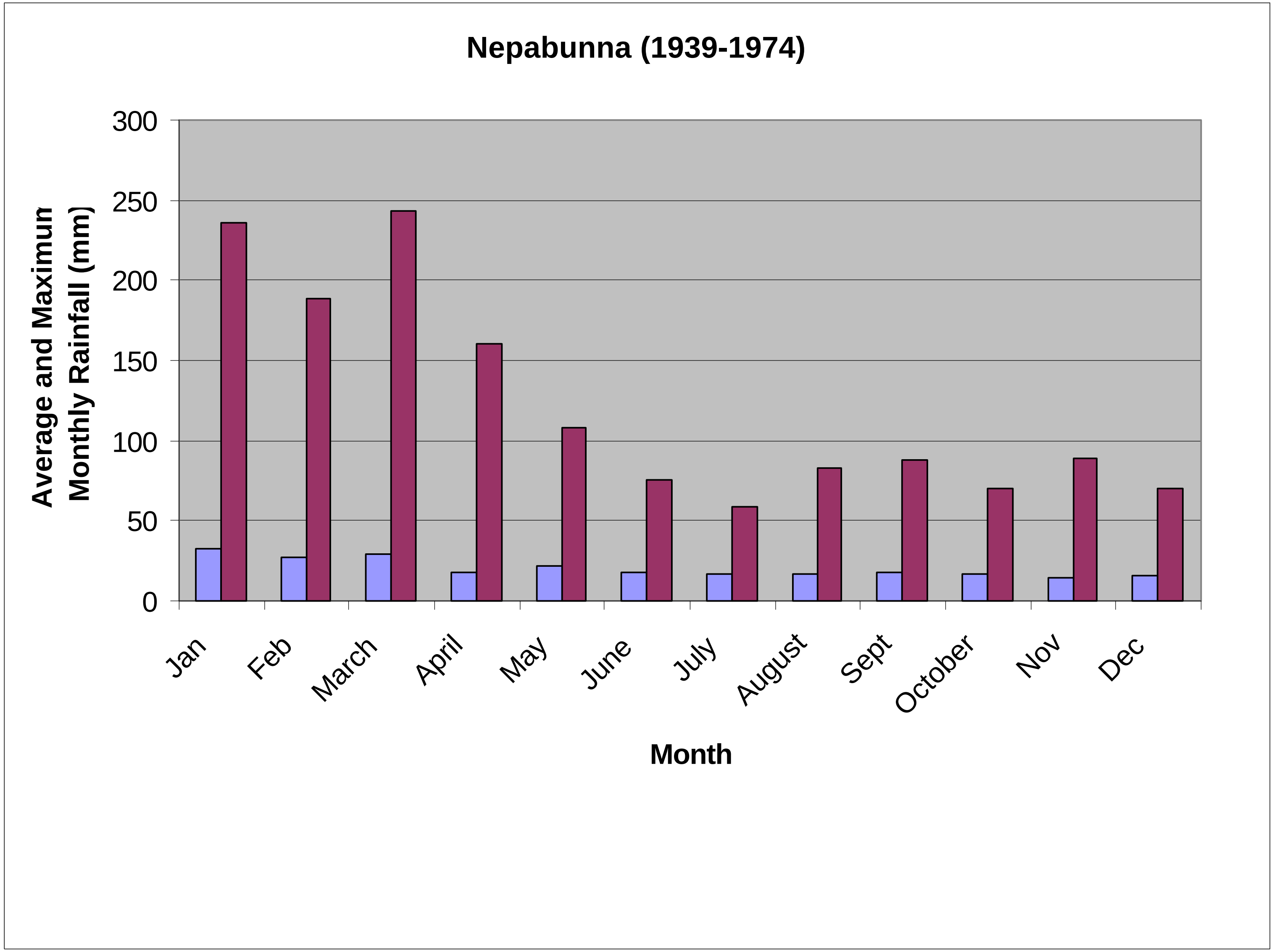


Figure A2 *Nepabunna Average and Maximum monthly rainfalls.*

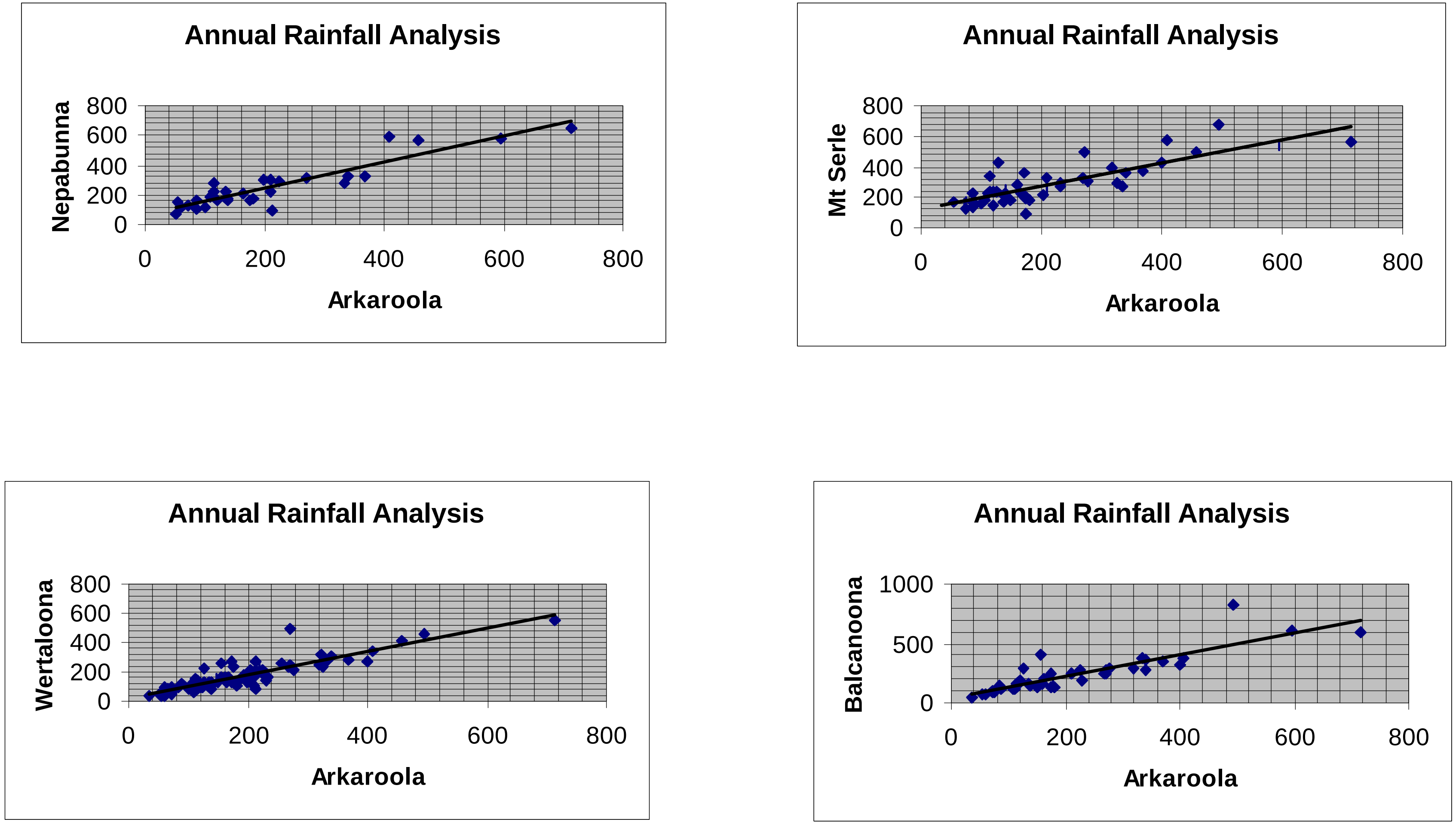


Figure A3 *Comparison of Annual Rainfall with Arkaroola*

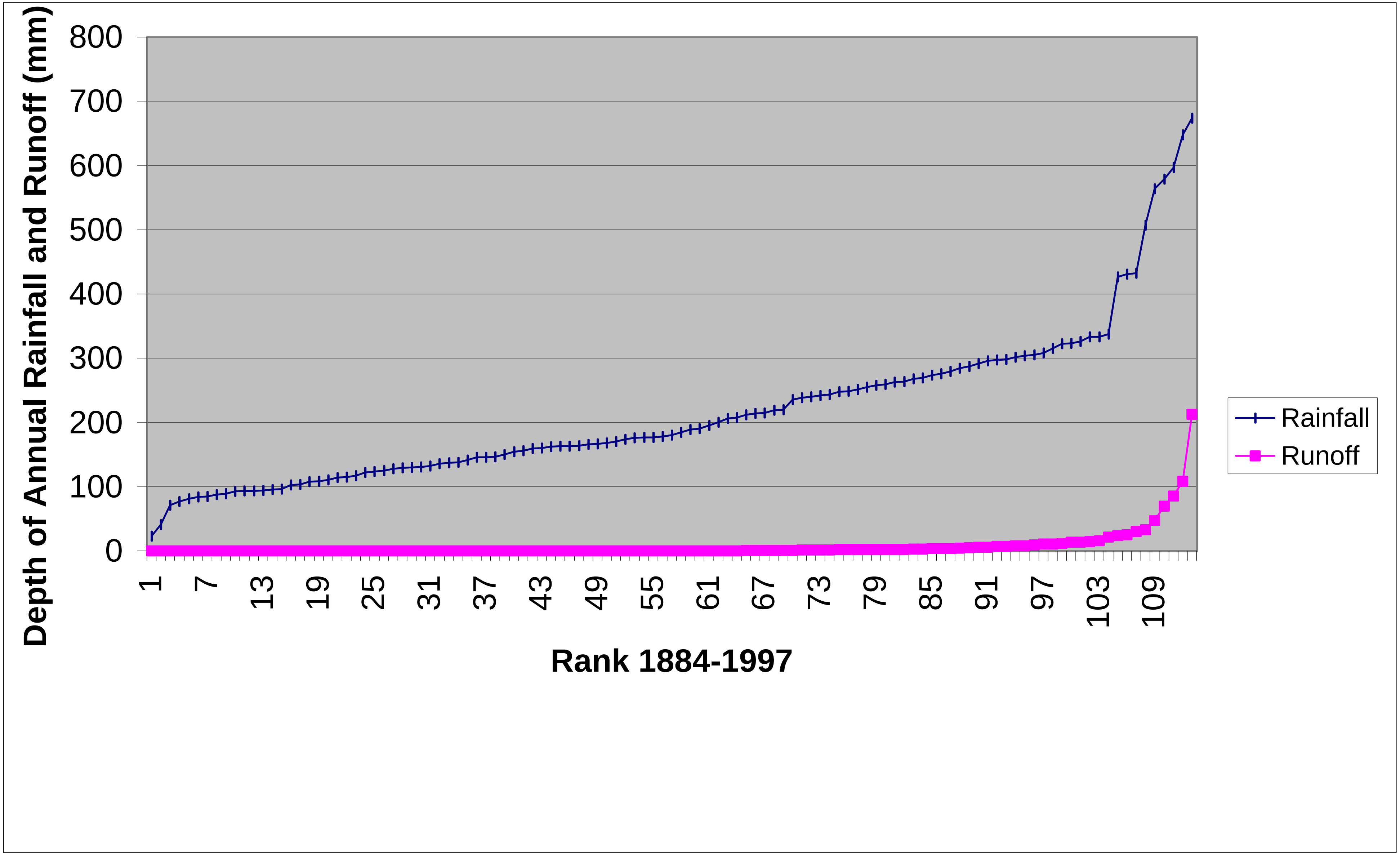


Figure A4 Ranked Annual Rainfall and Predicted Runoff for the Nepabunna area.

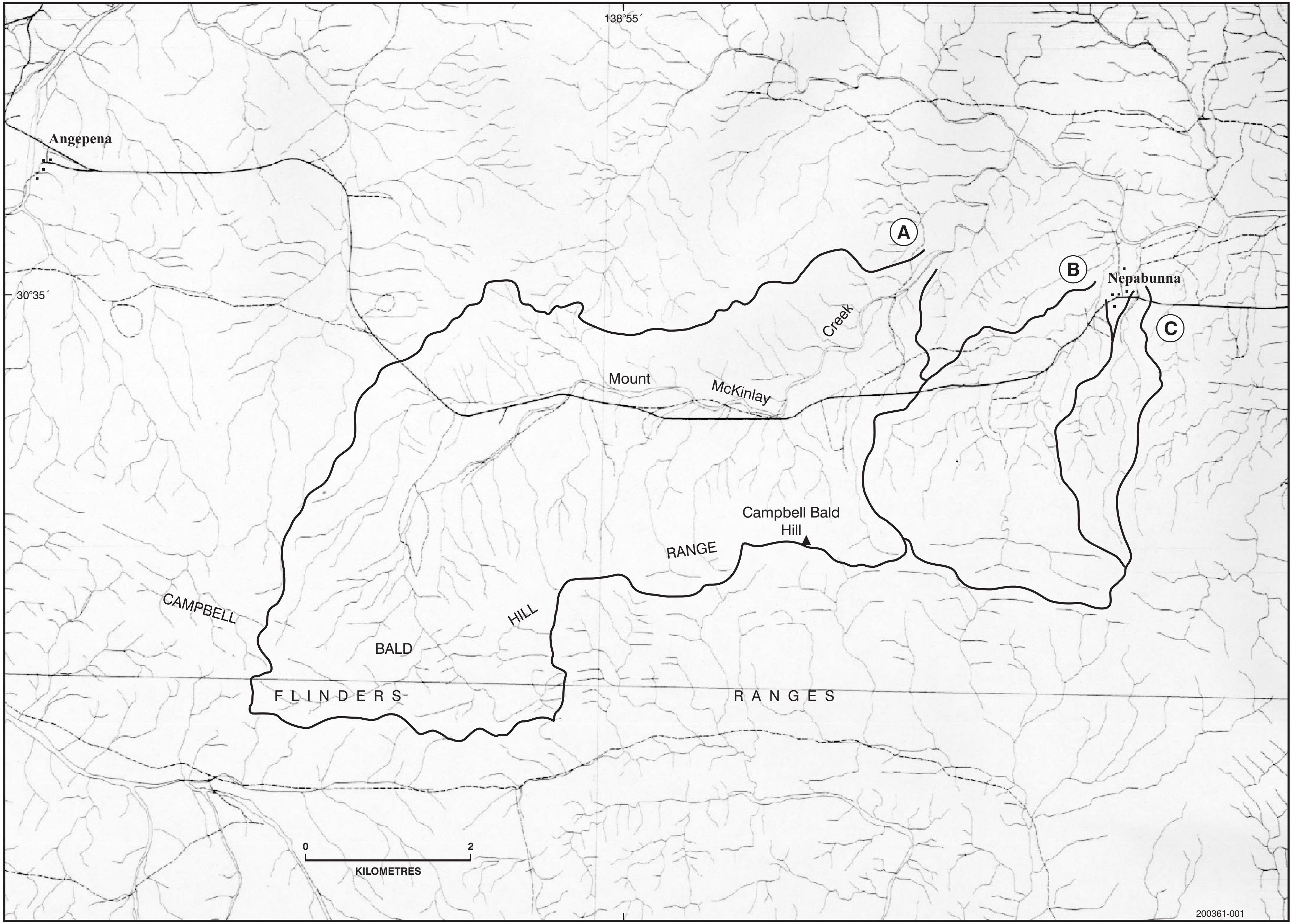


Figure A5 *Locality plan showing topography, recommended capture sites and catchment areas.*

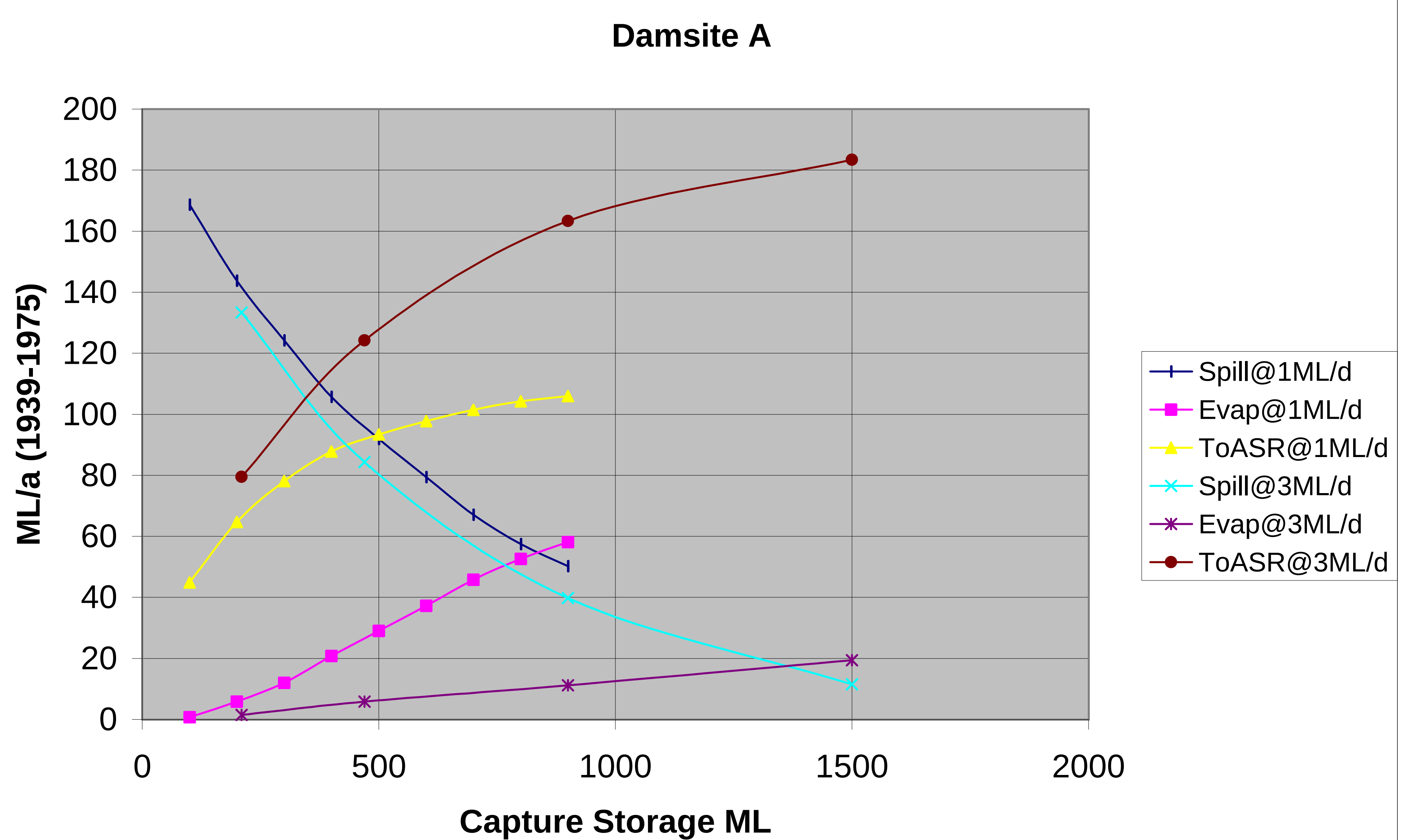


Figure A6 *Captured Water Disposal for Damsite A.*

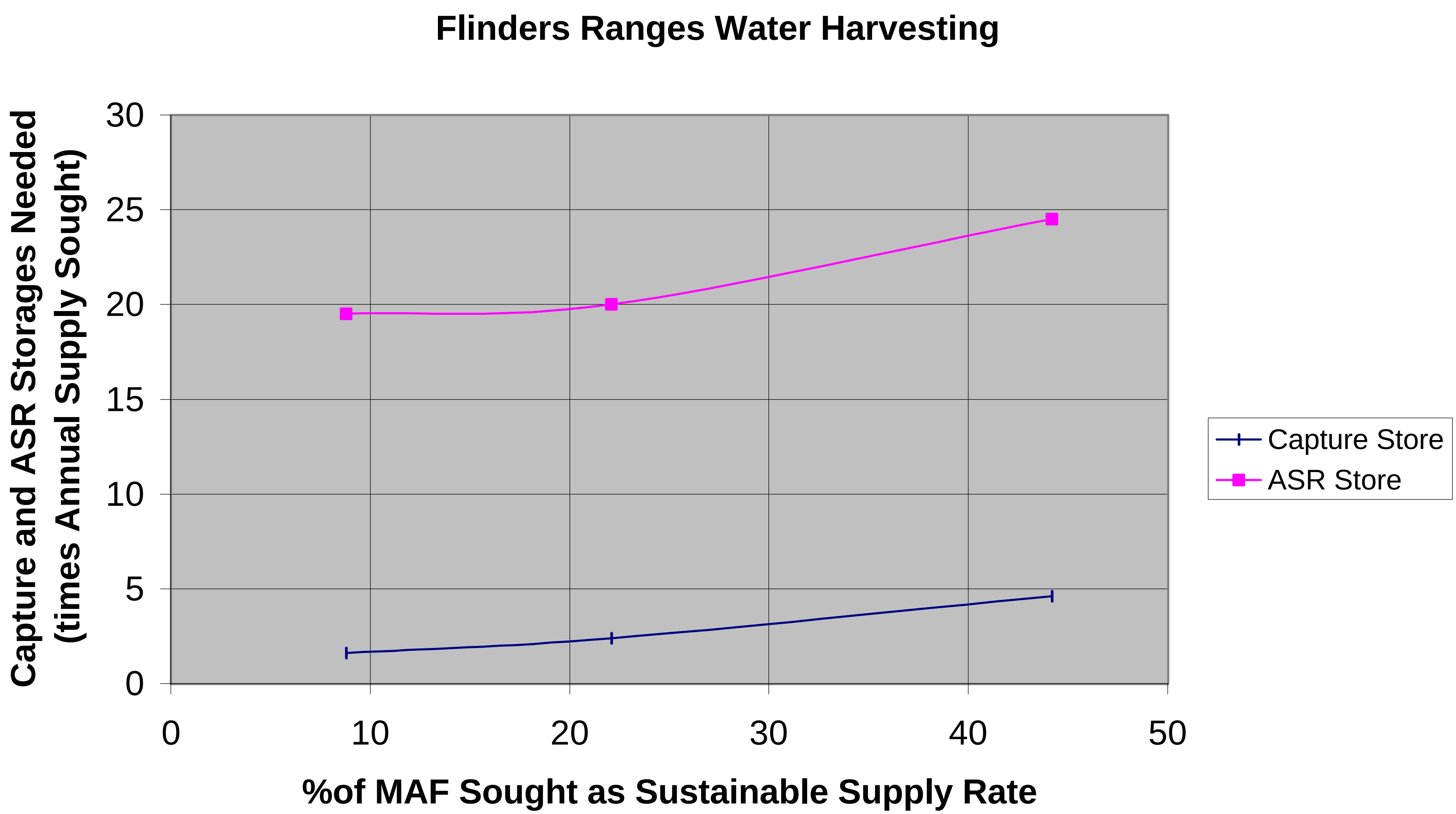


Figure A7 *Flinders Ranges Water Storages required.*

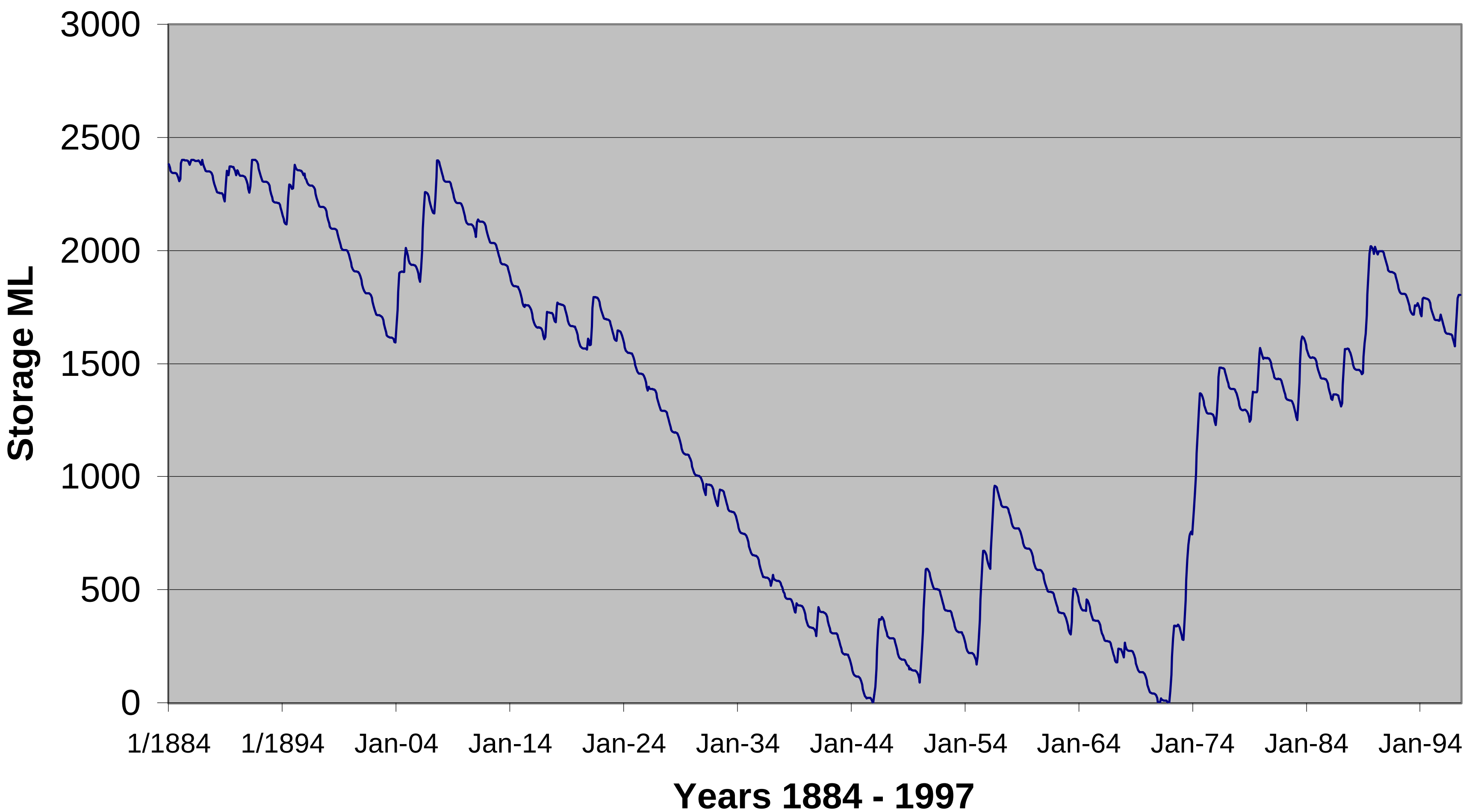
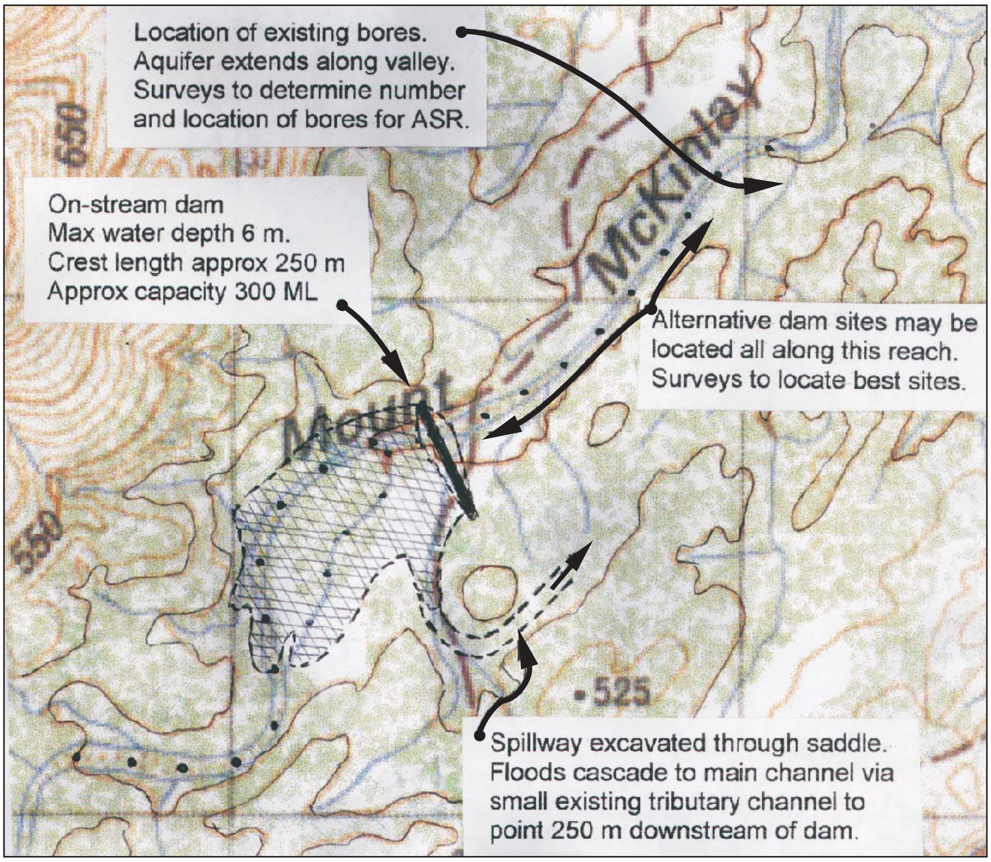


Figure A8 *Long Term Storage required to supply 100 ML/a every year.*

(a)



(b)

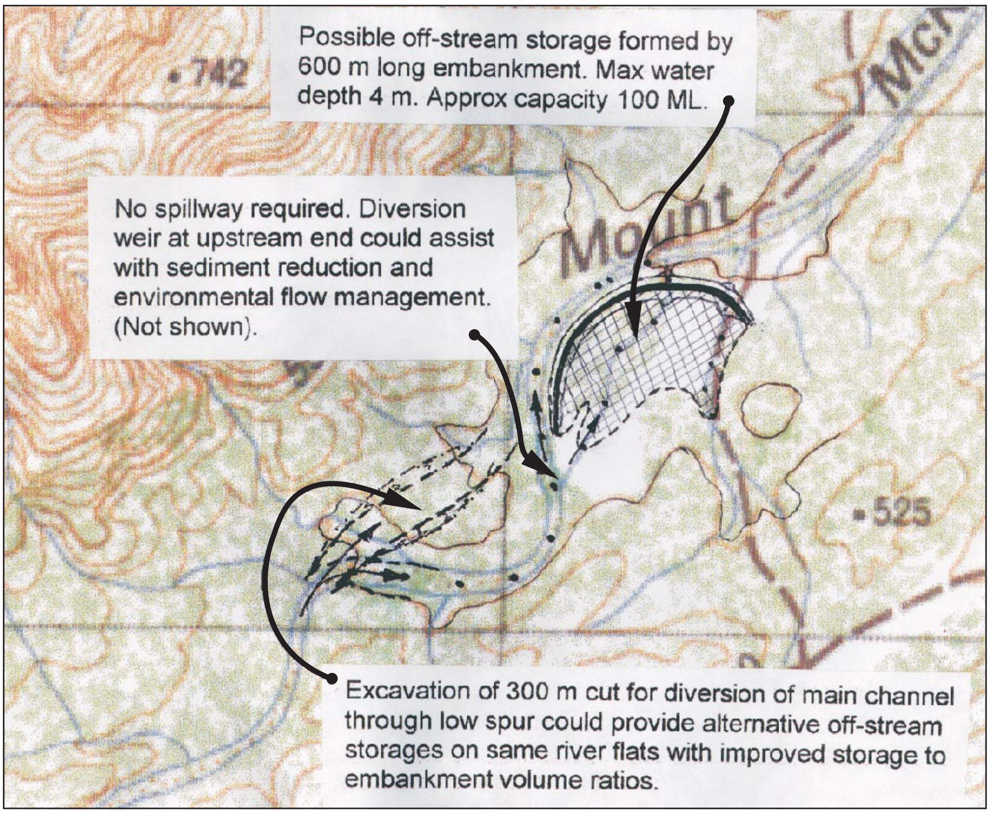


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Figure A9 *Suggested on and off stream dams and spillways Site A.*

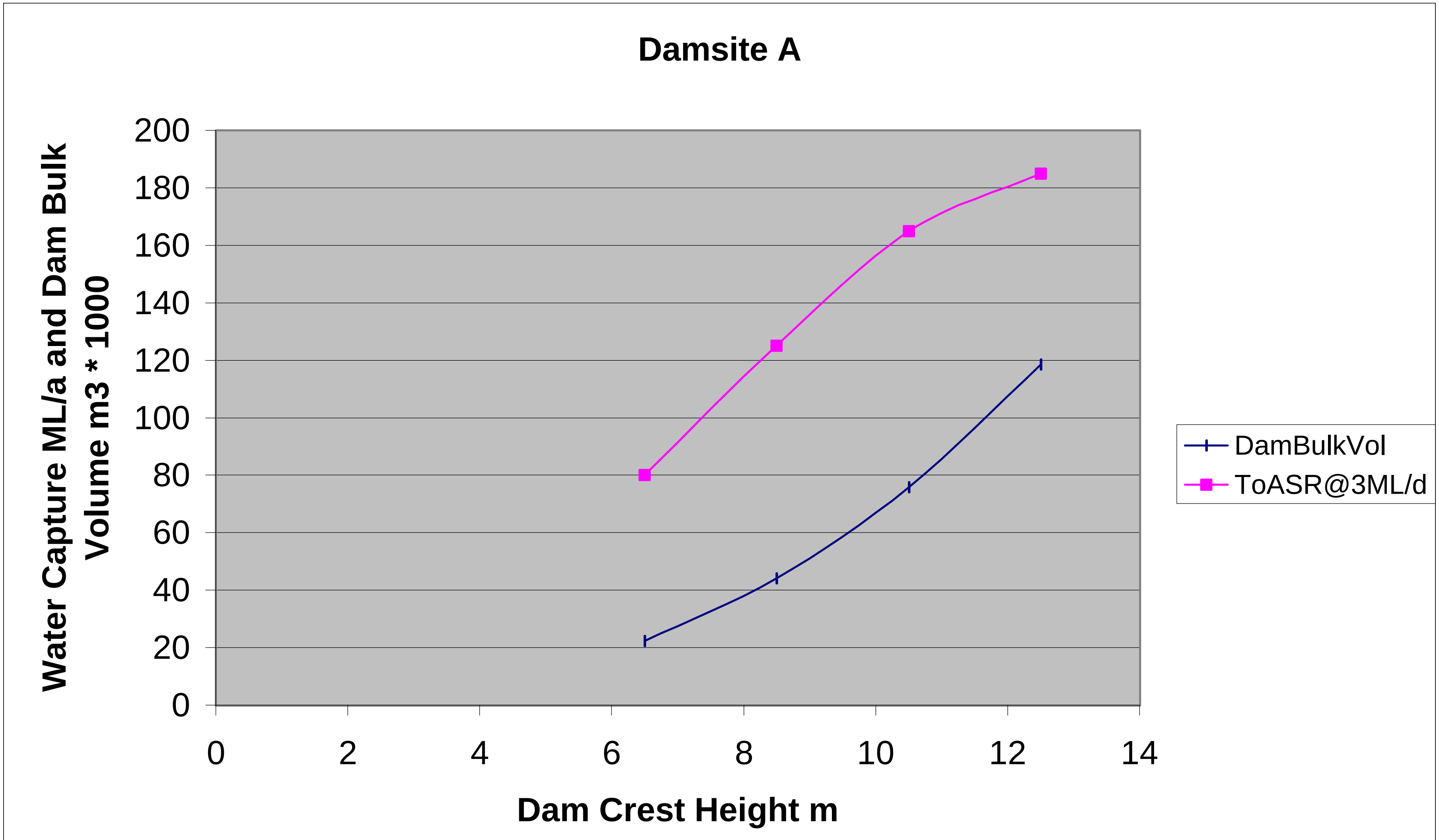


Figure A10 *Damsite A dam bulk and Capture Storage*

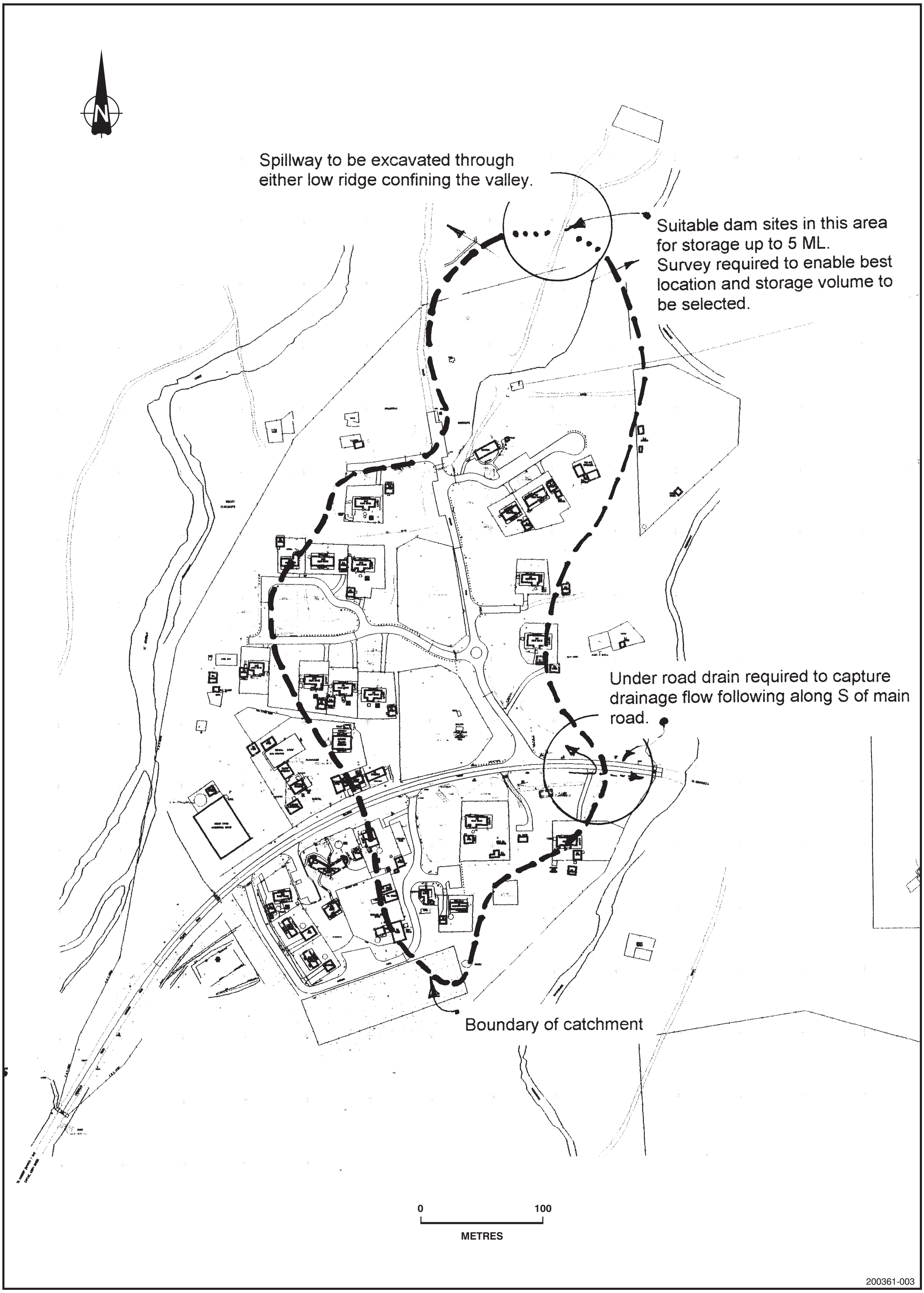


Figure A11 Catchment area and dam location Site B.

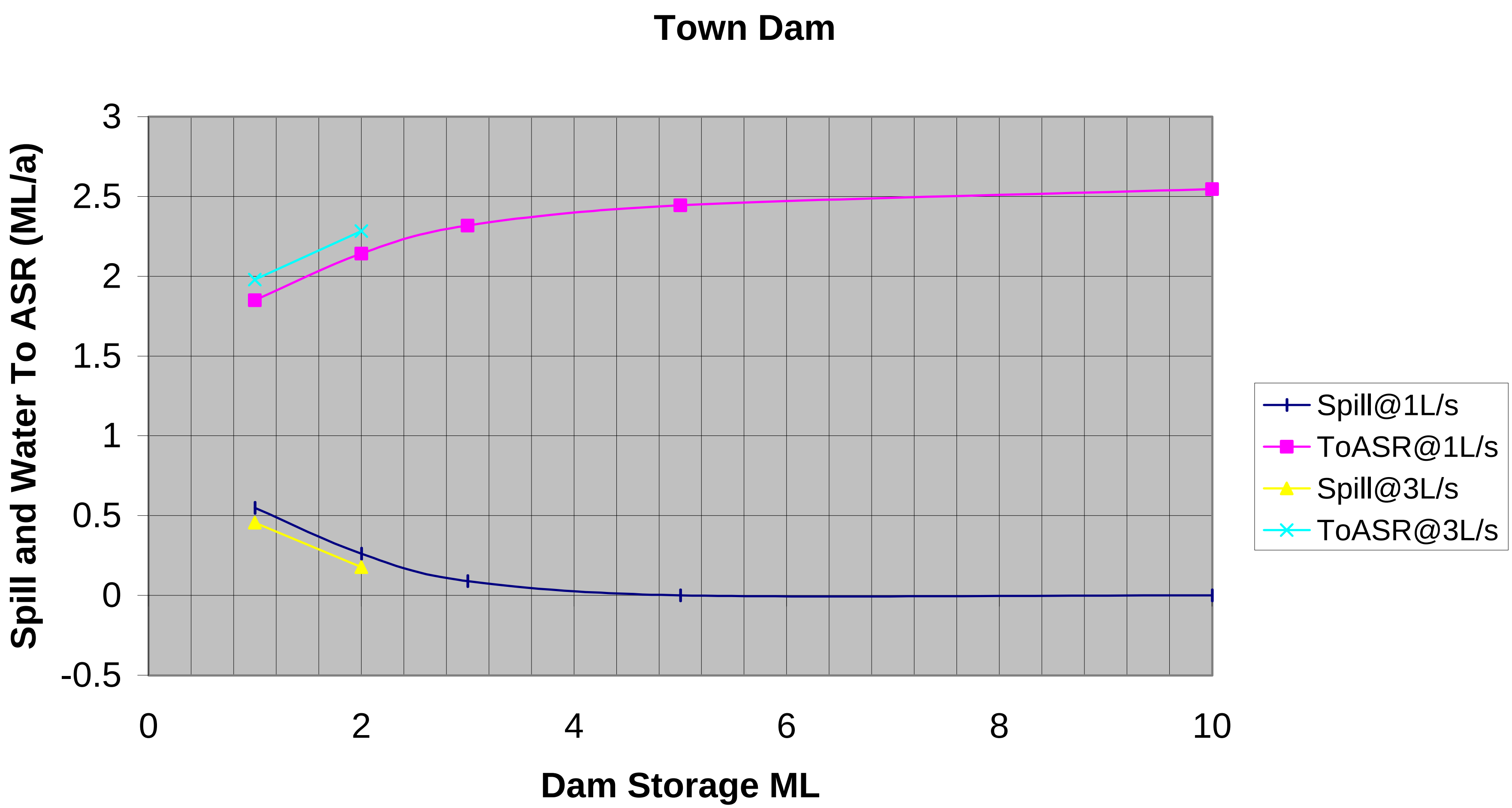


Figure A12 *Town Dam Water Capture Disposal.*

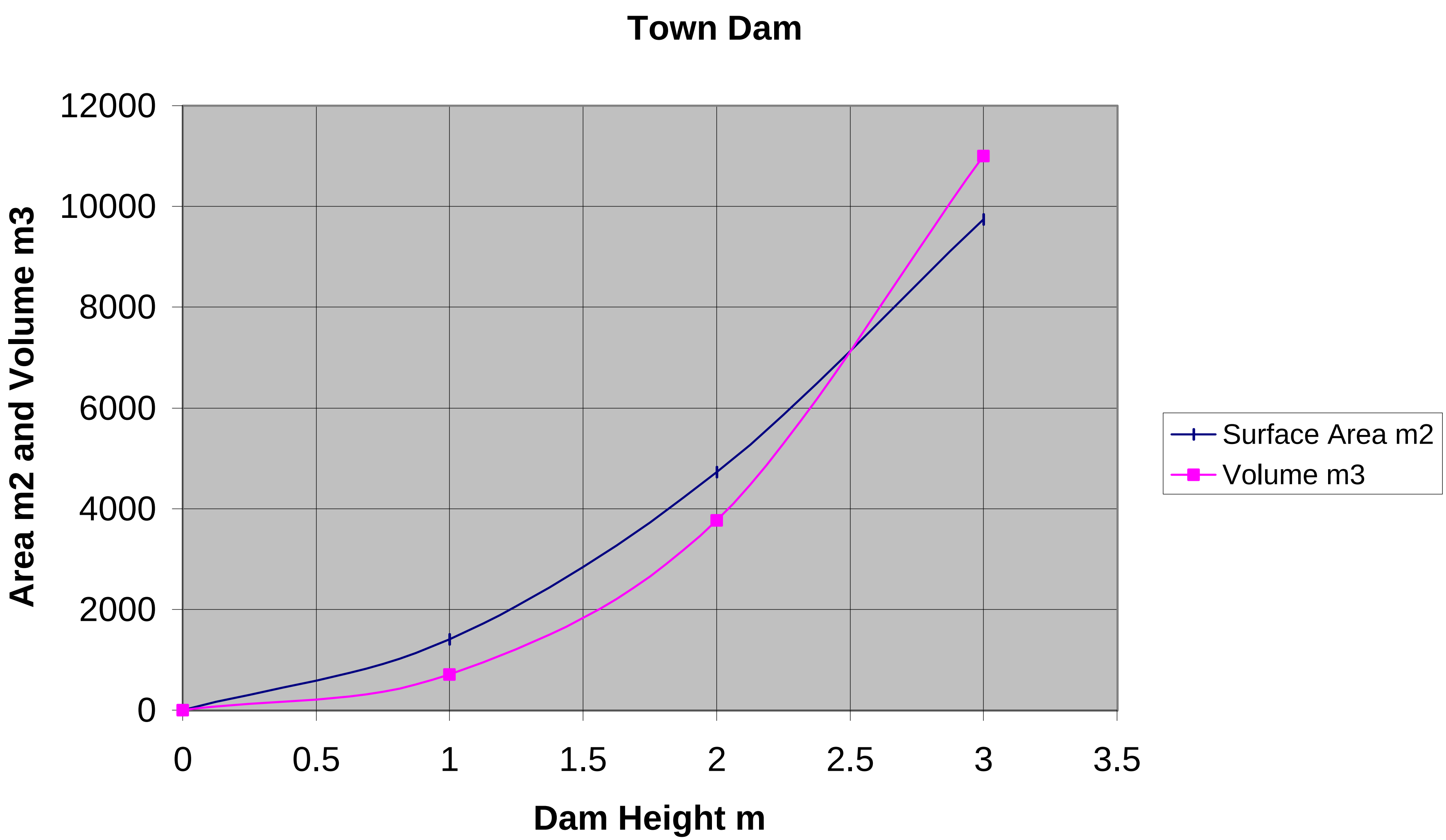


Figure A13 *Town Damsite Dam Bulk and Capture Storage*

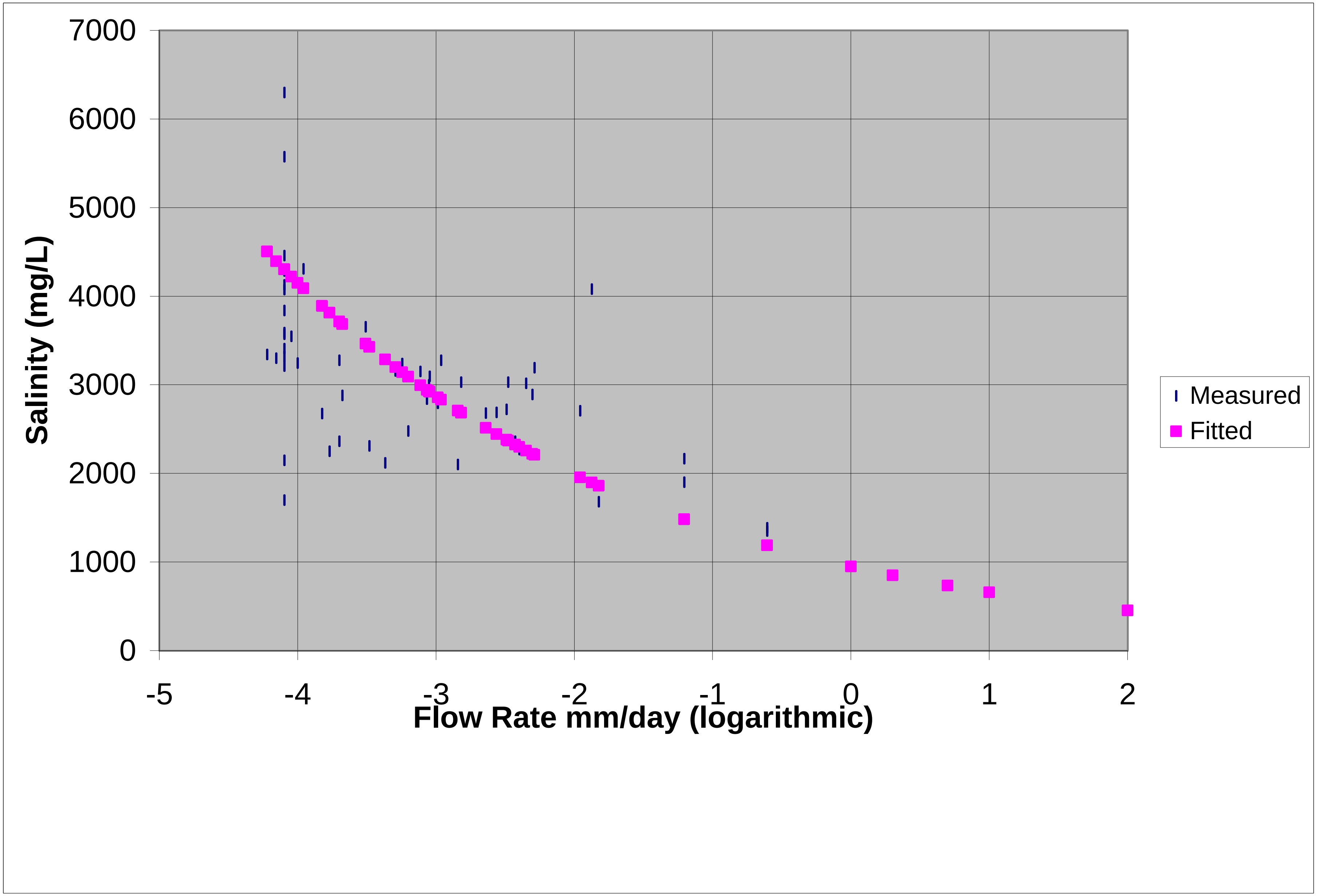


Figure A14 Analysis of Flow and Salinity in Big John Creek

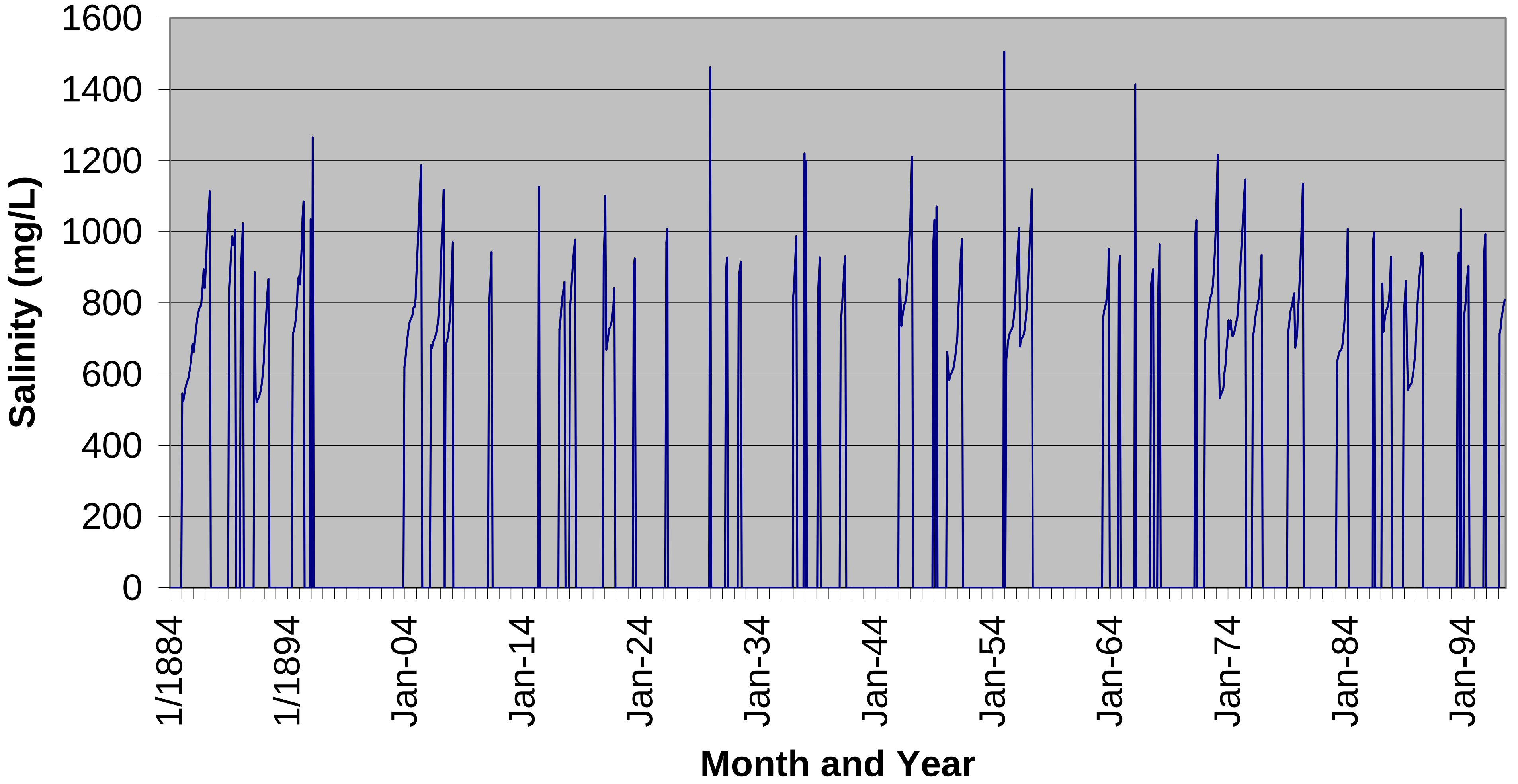


Figure A15 *Salinity of water transferred to ASR at Damsite A.*

Appendix B

Assessment of the potential for ASR in the Nepabunna Community

N.Z. Gerges

APPENDIX B

ASSESSMENT OF POTENTIAL OF ASR IN NEPABUNNA ABORIGINAL COMMUNITY

N.Z.GERGES & K. OSEI-BONSU
DEPARTMENT OF WATER RESOURCES

GEOLOGY

The geology of the area comprises metasediments of the Adelaide Geosyncline, and is fully described in Read (1981) (Figure B1). Of particular interest for this study is the Wilkawillina Limestone Formation, which underlies much of the immediate surrounds of Nepabunna and comprises mainly limestone and siltstone. It can be divided into 2 units, the lower one being the more interesting hydrogeologically and consisting of clean limestone inter-bedded with silty limestone, nodular limestone and calcareous siltstone. The upper unit consists of calcareous and silty limestone. Dips are quite variable, but are characteristically steep, of the order of 70–80^{o}.

Most outcrop is characterised by minor faults/joints and steep-dipping eastward trending cleavages. The cleavages are parallel to the axial plane of folding. Some of the joints appear to be filled with calcite. The spacing of the joints is several meters in the Wilkawillina Limestone but closer in the Nepabunna Siltstone.

The dominant joint set has near north-south strike with near vertical dip.

HYDROGEOLOGY

The site lies in an area of fractured rock aquifers. A number of the water supply wells that have been drilled indicate that there is localised occurrence of groundwater in fractures in the limestone and siltstone rocks. Borehole and pumping data suggest the presence of linear or strip aquifers of limited areal extent.

Generally the aquifers are heterogeneous in nature due to variations in lithology and structure over a short distance. While the steeply dipping joint system could provide both vertical and horizontal permeability, the former is evidently favoured by this orientation of jointing. Thus vertical drillholes are prone to missing the resulting aquifers, but are likely to have excellent yields when the permeable zone is intersected. Yields from these aquifers range between 0.2 l/sec and 22.7 l/sec, the latter being obtained from a well tapping a fractured rock aquifer in massive limestone beds of the lower unit of the Wikawillina Limestone some 2 km west of Nepabunna.

Groundwater occurs under confined conditions, the depth to standing water levels ranging between 16 and 38 m. The observed decline in water levels in production well 101 indicates that the aquifers contain a limited supply of water, the reasons being limited recharge to the aquifers and limited areal extent of the fracture zones.

While local structural conditions are more important than lithology in the search for adequate supplies of groundwater, the fractured limestone tends to give much higher yields than the siltstone. The aquifers are believed to be made up of a system of minor faults.

The salinity of groundwater from these fractured rock aquifers varies from 1200 mg/l to 3000 mg/l. Better quality water is also found close to areas where recharge from creeks occurs.

AQUIFER PROPERTIES

In the 1980's two sites (A and B, Figure B2) were explored for a water supply for the Nepabunna Community. Site A, located some 2km west of Nepabunna in the valley of Mount McKinlay Creek, is focussed on a steeply dipping massive limestone unit of the Wilkawillina Limestone Formation, while Site B, around Nepabunna itself, is in the Nepabunna Siltstone. Eight wells were drilled, four at site A (Unit numbers 96, 149, 165 and 209) and four at site B (Unit Numbers 77, 95, 97 and 101). From the limited information available wells 149 and 209 at site A provided a comparatively higher yield than any of the other wells at either site, but unfortunately there is no pumping information for these or any of the wells at site A. For this reason the properties of the aquifers can only be determined for wells at site B.

Pump Test Analyses

Figures B3, B4 and B5 are log-log plots of drawdown versus time for pumping and observation wells 97, 101 and 77, located in site B. Rather than resulting in Theis curves the plots of test data from wells in the area (site B) exhibit straight-line fits on log-log plots, showing that the flow in the aquifers is not radial. Semi-log plots of these data as drawdown, s, versus time, t, are curvilinear. This is a certain indication that these are fractured rock aquifers with discrete boundaries.

A summary of the analyses of these wells follows.

Well 97

Figure B3 is a plot of drawdown versus time for pumping well 97, using a pumping rate of 158kl/day (1.8L/s). This indicates a linear flow regime during later parts of pumping. (Linear flow conditions are indicative of flow lines that do not diverge away from a pumping well, ie, that the cross-sectional area of flow does not increase with distance from the pumping well). Linear flow conditions occur in aquifers with parallel no-flow boundaries or aquifers with high conductivity vertical fractures.

The following equations best describe the behaviour of the well during the pumping test.

$s = 0.0050Q\ t^{0.273}$ for $0 \le t \le 300$ (1)

$s = 0.00139Q\ t^{0.5}$ for $300 \le t \le 700$ (linear flow regime) (2)

Where
s (m) is the drawdown in the pumping well
Q (m^3/d) is the pumping rate and
t (minutes) is the time since pumping began.

The slope of the line for the time interval $300 \le t \le 700$ gives an estimate of $L(TS)^{0.5}$,

Where
L is the length of the fracture
T is the transmissivity parallel to the direction of flow, and
S is the storage coefficient.

From equation 2, $\underline{L(TS)^{0.5} = 719 m^2/day^{1/2}}$

The term $L(TS)^{0.5}$ is known as the generalised hydraulic diffusivity of the fractured zone. Where L is known and S can be reasonably estimated, T can be determined.

Wells 101 and 77

Well 101 was pumped at a rate of 171.7kl/day (2.0L/s) for more than 700 minutes while Well 77, 8.6 m away, was used as an observation well. The drawdown in the pumped and observation wells was monitored (Figures B4, B5, B6 and B7).

Figure B4 is a log-log plot of the drawdown in the pumping well (101) versus time, t. A straight line can be fitted to this data, which is best described by the following equation relating drawdown, s, and time, t.

$s = 0.00395Q\, t^{0.21}$ for $10 \leq t \leq 700$ (3)

Figure B5 is a log-log plot of drawdown from the observation well (77) against time. Again, a straight line can be fitted to the data giving the following relationship between drawdown, s, and time, t.

$s = 0.000792Q\, t^{0.3846}$ for $10 \leq t \leq 700$ (4)

Figure B6 is a log-log plot of the observation well drawdown data versus t/r^2.

Figure B7 is a Cooper-Jacob semi-log plot of time-drawdown data from the observation well. The plot falls on two semi-log straight-line segments. The drawdown increased on a semi-log linear trend from about t=20 minutes until a break in the slope which is evident at t=150 minutes. The slope of the first semi-log straight line is 0.6m per log cycle while that of the second segment is 1.11m per log cycle. This apparent near doubling of the semi-log slope suggests a linear-type boundary in the vicinity of the observation well.

The transmissivity, estimated by using the Cooper-Jacob method from the slope of the first segment of Figure B7, is <u>52 m^2/day</u>. The data provided an estimate of storage coefficient of about <u>0.00443</u>.

The value of calculated transmissivity and storage coefficient from the slope of second segment of Figure B7 is <u>28 m^2/day</u> and <u>0.00658</u> respectively.

Potential Aquifer Yield

The determination of the long-term yield of the aquifer from these short-term pumping tests is risky because of uncertainty regarding the extent of the fractures and the possible presence of discharge boundaries; these would become evident with long-term observation of drawdown. Moreover, extensive step testing is needed to determine the optimum pumping rate for the test without adversely de-watering the wells.

With these reservations, this short term pumping test may be used to <u>estimate</u> the long-term yield of the wells.

A simple way of making a baseline estimate of the potential long-term yield of the wells is to apply equations (2) and (3) to calculate the continuous discharge that would not lead to de-watering of the aquifers in a specified length of time.

Equation 2 can be rewritten in the form:

$Q = s_{av}/(at^x)$ (5)

Where
s_{av} is the available drawdown in m,
t is the time in minutes,
Q is the pumping rate in kl/day, and
a and x are derived from the linear fit of the pump test data (Equations 3 and 4).

The available drawdown, s_{av}, is derived from both well efficiency and the maximum drawdown until the aquifer goes unconfined. In calculating the potential yield, allowance is made for well efficiency and a ‘free-board’ to prevent the aquifer from becoming unconfined.

Well 97

Available drawdown to the top of aquifer penetrated is 15m. Assuming 30% well losses, equation 2 can be rewritten in the form of equation (5) as:

$Q = (10.5)/(0.00139\, t^{0.5})$ (6)

A plot of Q against t is shown in Figure B8, which is based on equation (6) and can be used in estimating the potential yield of this well for a given pumping duration. This shows a yield of $20m^3$/day for 100 days and is asymptotic to $10m^3$/day.

Well 101

Available drawdown to top of aquifer penetrated by well 101 is 21 m. Accounting for 30% well loss equation (3) can be rewritten in the form of equation (5) as follows.

$Q = 14.7/(0.00395\, t^{0.21})$ (7)

This relationship is shown graphically in Figure B9, and can be used to estimate yield for a given duration of pumping. The yield for 100 days is $300m^3$/day and is asymptotic to $250m^3$/day.

Alternatively, the potential maximum yield of the wells completed in the fractured rock aquifer can be estimated from the Theis non-equilibrium equation, which can be written as:

$Q=(12.566Ts)/W(u)$ (8)

where
W(u) is Theis well function that is available from published tables for dimensionless parameter

$U=(r^2S)/(4Tt)$ (9)

where
r radial distance, (m),
S storage coefficient, (dimensionless)
t time, (days)
T transmissivity, m^2/day)
s drawdown (m) and
Q pumping rate, (m^3/day)

Using r=0.05 m, T=28.31 m^2/day, S=0.00658 and assuming a pumping period of 210 days, it can be shown that $u=6.9x10^{-10}$, and from published tables of W(u) versus u, W(u)=20.5027.

Static water level is about 21 m above the top of the aquifer penetrated by Well 101. Assuming 70% of the drawdown to account for well efficiency and a “free-board” to prevent the aquifer from becoming unconfined, the maximum available drawdown would be 14.7m. Inserting these values into equation 8, the potential yield of the aquifer can be estimated to be

$Q = (12.57x28.31x14.7)/(20.5027) = \underline{255\ m^3/day}$. (3L/s)

Using the value of T=52.377 m^2/d, S=0.00443 and a pumping period of 210 days it can be shown that u=2.5x10^{-10} and W(u)=21.5323. Using a maximum available drawdown of 14.7 m, the potential yield of the well is estimated to be

Q = (12.57x52.377x14.7)/(21.5323) = 449 m^3/d. (5.2L/s)

An average potential yield of 352 m^3/day (4.1L/s) can therefore be obtained for Well 101.

Using equation (7), the potential yield for Well 101 for uninterrupted pumping period of 120 days is 263 m^3/day. This value is comparable to the value of 255 m^3/day obtained from application of Theis non-equilibrium equation using the aquifer properties determined from the second straight-line segment of Figure B7.

From this comparison it can be said that the effective transmissivity and storage coefficient of the aquifer at site B are 28 m^2/day and 0.00658, respectively.

Conclusions on Aquifer Properties

Equations to predict drawdown in individual wells at site B have been established. The derived equations are based on 700 minutes of pumping of each well.

Well 101

Based on Theis non-equilibrium equation the estimated potential maximum yield for a 210 days period of uninterrupted pumping ranges between 255.25 m^3/day and 449.34 m^3/day (3.0 to 5.2L/s).

Using the general equation, (equation 7), describing the relationship between drawdown, (s), and time, (t), a yield of 290 m^3/day (3.4L/s) for 210 days of uninterrupted pumping was obtained.

The optimum yield for 210 days of continuous pumping is about 250 m^3/day (2.9L/s).

Well 97

Using the general equation a yield of 10m^3/day for 210 days of uninterrupted pumping was obtained.

Well 209

Air lifting results indicated that this well, which is located at site A, is capable of similar yields to Well 101 and might be the preferred candidate for ASR. It should be pump tested to allow the determination of the aquifer properties, the establishment of the time-drawdown relationship and determination of the long-term yield.

GENERAL COMMENTS ON AQUIFER STORAGE AND RECOVERY

The Aquifer storage and recovery scheme involves injection when surface water is available and extraction during dry periods. The obvious benefit of this is that it provides a water resource management tool that will help maintain groundwater systems for current and future development. This is particularly valuable where relatively impermeable layers between the surface and the aquifer inhibit natural aquifer recharge, or where rainfall is rare but heavy, encouraging major losses by runoff.

A second benefit may occur where native groundwater is of a lower quality than required. The quality of the stored water may be improved locally by the injection of better quality water.

Several issues must be addressed, including the turbidity and chemical/bacteriological characteristics of the injected water and their impact on the aquifer. There is the possibility of chemical reactions that could affect well efficiency, such as clogging of the aquifer pores. Such reactions might also affect water quality. There may be a need for pre-treatment of water prior to injection. There is also the question of the capacity of the aquifer to store additional water and the associated build-up of a pressure head.

Advantages of a fractured rock aquifer for ASR

- Not usually susceptible to clogging
- More likely to be a compartment aquifer, in which case it can theoretically be completely dewatered and then injected with high quality water which will be retained for later use
- The aquifer material is not likely to react with the injected water unless it is calcareous. This is not usually the case in South Australia.

Disadvantages of a fractured rock aquifer for ASR

- Injected water may not be recoverable, since it may move too quickly down the aquifer system. Some injected water is likely to be lost unless the aquifer is confined, and such losses will affect the final water equation.
- Recovery may result in a cycle of low to high salinity during pumping. This may be a result of storing and releasing from fractures followed by delayed release from pores.

TARGET AQUIFERS FOR STORAGE AND RECOVERY IN THE NEPABUNNA AREA

Analysis of previous work suggests that the potential for ASR in the immediate vicinity of Nepabanna is controlled primarily by the availability of surface runoff in close proximity to a major recognisable aquifer.

It is evident that three known aquifer systems present a favourable opportunity for ASR:

- The inter-bedded massive limestone bands (fractured limestone aquifer) within the Wilkawillina Limestone (wells 209 and 149) provide the best opportunities for water supply and injection potential.

- The intersected massive cavities or fractures in the Parachilna Formation between 63 and 81m below ground during the drilling of Well 96 deserve more consideration. They may have the potential for disposal of large volumes of surface water that can be harvested later. Available information suggests that these formations may be relatively fractured and may be present west and south of Nepabunna area. However, this well is unfortunately not available for immediate testing as it was dry and therefore was backfilled.

- The tested and partially evaluated aquifer at the Nepabunna Community (Wells 77, 97 and 101).

Furthermore, there is substantial potential for fractures along major structures, although the number of wells penetrating them is still very small (Wells 223, 224, 225 and 226). These major structures, particularly the inferred faults located southwest and south of Nepabunna, deserve further consideration and more detailed investigation. In the case of the fault located southwest of Nepabunna, investigation for the storage of a large volume of water could be initiated 500 m southwest of well 224 (5 km west of Nepabunna), which is drilled in the Parachilna Formation. It is anticipated that combinations of major structures and cavities could provide long term sustainable injection/extraction rates. However, at this particular location the surface water catchment area is most likely to be small and consequently runoff would be limited.

RECOMMENDED WORK PROGRAMS

The following work programs target the fractured rock (limestone) aquifer for water supply and injection. They comprise 3 stages, with the implementation of each stage subject to the results from the previous stage.

Prior to the commencement of any further drilling in this area, it is recommended that investigations be undertaken to examine the potential extraction (Stage 1) and injection (Stage 2) in existing wells completed in the aquifers. This includes monitoring for recovered water chemical changes, chemical clogging and the build up of a pressure head in the injection wells.

The work programs include well discharge testing and injection testing, which are described below.

Well discharge testing.

A 3-stage step drawdown test, followed by a 72-hour constant discharge test and a 48-hour recovery test, is recommended for each well to determine:

- the well yield-drawdown relationship and well efficiency.
- the sustainable yield.
- the nature of the fractured rock aquifer, and
- the long term aquifer sustainable yield

which will:

- form the baseline required for monitoring any subsequent clogging, and
- assess the radius of influence from the well.

Injection testing

Injection testing is required to confirm the sustainability of the injection and to develop operational guidelines. The intention is to simulate, as closely as possible, the anticipated conditions for long term injection.

This involves conducting 3-stage injection tests and a constant injection test for several days, followed by a recovery efficiency test.

Following satisfactory well discharge and injection investigations, further wells could be confidently drilled on the site, followed by only minimal testing.

SITE A

Although both wells 209 and 149 are prospective targets to supply the required quantity of groundwater, only well 149 has so far been utilised as a water supply. Neither well has been pump tested. Of the wells drilled to date in the Nepabunna area, these offer the best prospect for long term extraction and injection sustainability.

Injection testing is stage 2 of this project, and will only be carried out if the discharge testing results are satisfactory. However, there is a problem with a supply of feedstock for the injection testing, if required, and the easiest solution will be to utilise the water from discharge testing. This, in turn, requires a storage facility for this water, amounting to approximately 5ML, which can be fulfilled by building a test dam to this capacity. Such a dam would hold the water for a week or two while testing (discharge, recovery and injection) is carried out and would also serve as a pilot operation for the larger dam facility required for harvesting rainwater. There is also the possibility that the construction work could be incorporated into final dam structure.

Discharge tests are recommended for both wells 149 and 209. If the results are satisfactory, with high sustainable yields being proved, then injection testing is recommended for well 209. This will be sufficient to trial the characteristics of the aquifer, but will not endanger the water supply from well 149 which is currently needed for the community.

Preparatory Work – Dam Building

A dam is to be built, possibly at the off-stream storage site some 1 km upstream of the wells but subject to final surveys. The off-stream site is preferred because this avoids any complications with rainfall before or during the testing period. It is possible, and preferable, that a site closer to the wells might be used.

Estimated cost $10 000

Stage 1 – Discharge Testing

(A) Well 209

Conduct 3 step drawdown tests, each for a duration of 100 minutes (1.2 l/s, 7 l/sec and 15 l/sec).
Conduct a constant discharge test for at least 3 days at the maximum yield indicated by the step drawdown tests (optimally 15 l/sec). The discharge water will be stored in the dam.
Monitor the drawdown in well 149.
Monitor the effectiveness of the dam – leakage.

Estimated cost

Test pumping	
Mobilisation (once only)	$6135
Operations 4 days x 24hrs x $85	$8160
Geophysical logging	
Mobilisation (once only)	$1900
Operations	$500
Water chemical analysis	$1200
Hydrogeological supervision and reporting	$2160
Total costs for stage 1	*$20 055*

(B) Well 149

Conduct 3 step drawdown tests, each for a duration of 100 minutes (1.2 l/s, 7 l/sec and 15 l/sec).
Conduct a constant discharge test for at least 3 days at the maximum yield indicated by the step drawdown tests (approximately 15 l/sec). The discharge water will be stored in the dam.
Monitor the drawdown in well 209.
Monitor the effectiveness of the dam.

Estimated cost

Test pumping operations	$8160
Geophysical logging	$500
Water chemical analysis	$1200
Hydrogeological supervision and reporting	$2160
Total costs for stage 1	*$12 020*

Stage 2 - Assessment of Pressure Head Build-up during Injection

(A) Injection Testing for Well 209

Conduct a field injection experiment using Well 209 as an injection well and previously extracted water from surface storage as injection water. This water may have to have a tracer element added to permit subsequent differentiation from native groundwater.

The work at this site involves the following:
Conduct constant injection testing at the maximum possible pumping rate (possibly 10 l/sec).
Monitor the pressure build up in Well 209.
Monitor chemical changes over residence period of time for Well 209.
Conduct recovery efficiency testing for Well 209.
Monitor the pressure build up in Well 149 and the movement of water into it from the injection well.
Assess the clogging potential.

Estimated cost

Injection operation	$8160
Water chemical analysis	$1200
Hydrogeological supervision and reporting	$2160
Total for this stage	*$11 520*

SITE B

Stage 1 - Injection Testing for Well 101

Conduct a field injection experiment using Well 101 as an injection well and Well 149 as a source of injection water.

The work at this site involves the following:
Conduct a constant injection test at 1.2 l/sec for 2–4 days using supply from Well 149 and existing infrastructures.
Monitor the pressure build up in Well 101.
Monitor the pressure build up in Wells 77, 95 and 97 and the movement of water into them from the injection well.
Monitor chemical changes over residence period of time for Well 101 (collect water samples).
Conduct recovery efficiency testing for Well 101.
Monitor drawdown in extraction Well 149.
Assess the clogging potential.

Estimated cost

Injection operation	$8160
Water chemical analysis	$1200
Hydrogeological supervision and reporting	$2160
Total for this stage	*$11 520*

Stage 3 – Drilling and Testing Program for a New Water Supply and Injection Well

The need for any new wells is totally dependent on the results of the above tests. Should they be required, the drilling locations and testing procedures will be likewise heavily influenced by the current results and are therefore not discussed further at this time.

CONCLUSIONS

Based on current information, ASR may be feasible at any of a number of sites in the Nepabunna region. The main constraint is the presence of a suitable aquifer, since all known aquifers in the area are of the fractured rock type and are of small extent and difficult to find. Aquifers have already been located at two sites, one at the Nepabunna townsite and the other 2 km west of the community, and the next stage of the investigation should be focussed on these sites.

At the Nepabunna townsite, wells have been pump tested and these results are analysed here. Injection testing of these wells is recommended.

At the site 2 km west, the wells are potentially of much greater capacity than the townsite wells, but have not been pump tested. The recommendations here include both pump testing and injection testing.

Other work, including drilling additional wells at these sites or in other locations where currently unknown aquifers might be intersected, is so dependent on the results of this testing that it would be effort ill spent to consider it at this stage.

APPENDIX

EXAMPLES OF ASR SCHEMES IN FRACTURED ROCK AQUIFERS

(A) Compartment aquifer - Scotch College:

At Scotch College School, Mitcham, irrigation demands led to the development of an ASR scheme using a well completed at a depth of 150 m in a steeply dipping quartzite aquifer with a groundwater salinity of 2300 mg/L. The hydraulic response of the aquifer under pumping suggests that it behaves as a compartment which can be pumped dry during a single summer irrigation period, thus allowing for injection. Due to the location adjacent to the ranges and a major faulted zone, it is theorised that the school is sited above a large block of dislodged basement material surrounded by an impermeable boundary. An ASR scheme was implemented by injecting low salinity (200–600 mg/L) water from the adjacent Brownhill Creek into a well completed at a depth of 48 m, and extracting it from a nearby 150 m deep well. Injection volumes are unmetered, but between 20 and 50 ML/year has been extracted since 1989 during summer irrigation. The salinity of the extracted water increases during irrigation from an initially low value of 1000 mg/L to 1600 mg/L at the end of irrigation.

(B) Strip aquifer - Northfield:

At Northfield housing estate in Adelaide an ASR scheme was developed to reduce storm water outflow and provide an irrigation water supply for parklands. This scheme involves first piping storm water to a wetland detention basin. After treatment it is then gravity injected into a saline fractured rock aquifer.

Hydrogeological investigations were conducted by drilling an 80 m investigation/injection well. The well was completed open hole in the confined fractured rock aquifer intersected at 44 m (slate). A highly fractured quartzite was intersected at 68 m with a SWL of 14 m and groundwater salinity of 2700 mg/L.

Several aquifer/well tests and injection tests using mains water (salinity 360 mg/L) at 22 L/s, (total volume 5 ML) indicated a transmissivity of 70 m^2/day. The long-term yield was calculated at 24 L/s and the gravity drainage rate as 13 L/s.

Following the gravity injection of the small volume of 10 ML of storm water during 1994 some extraction occurred. During repeated pumping periods of 6 hours water quality initially started at a value of 800 mg/L and rose to 1100 mg/L. This trend suggests that low salinity water is initally being withdrawn from the major fracture system, and after some time additional water is being withdrawn from minor fractures and the primary porosity, which yield more saline groundwater. This problem is expected to be reduced when much larger volumes of storm water are injected. During 1995 40 ML were injected followed by 50 ML in 1996 and a similar injection rates during the period between 1997 and 1999.

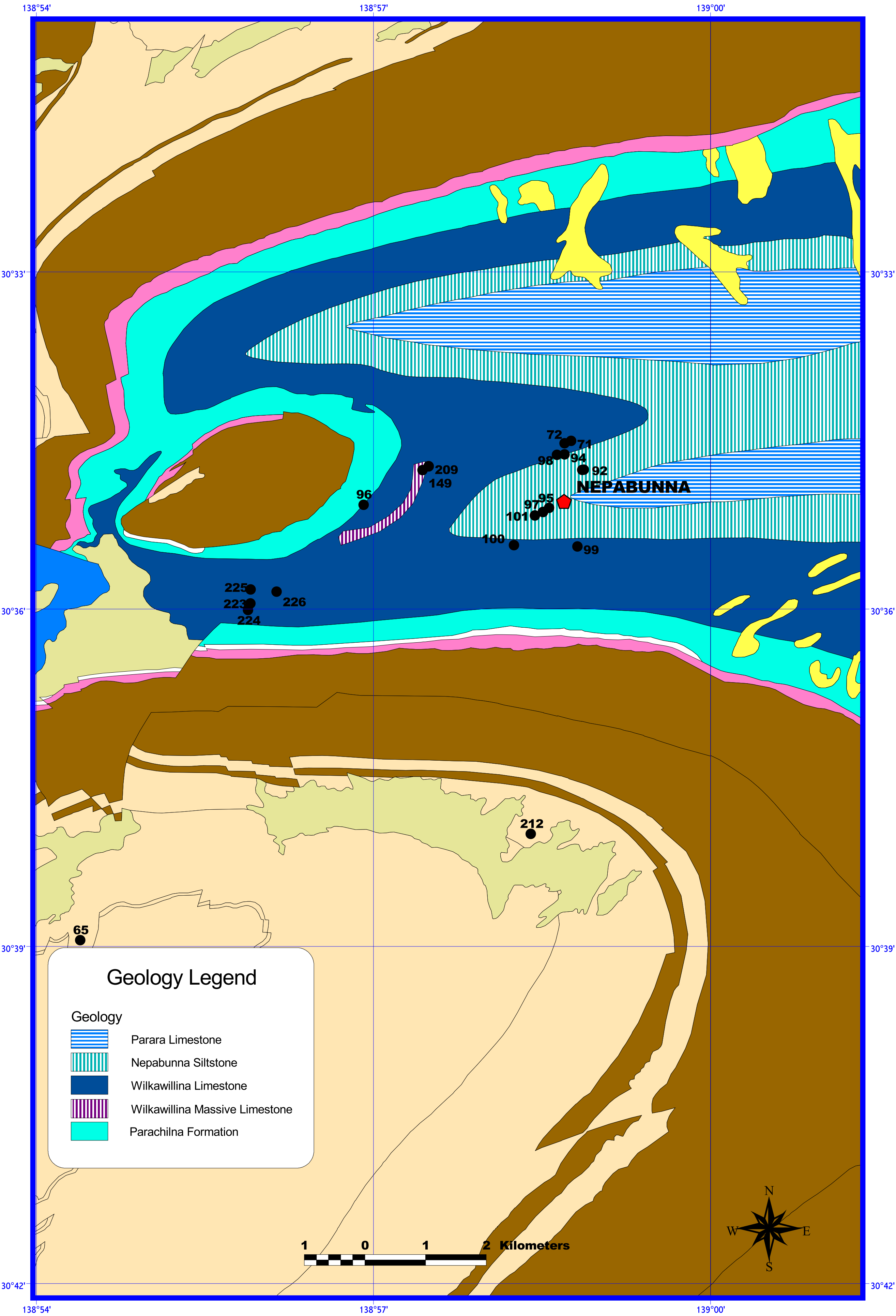


Figure B1

Nepabunna 1:100k Geology

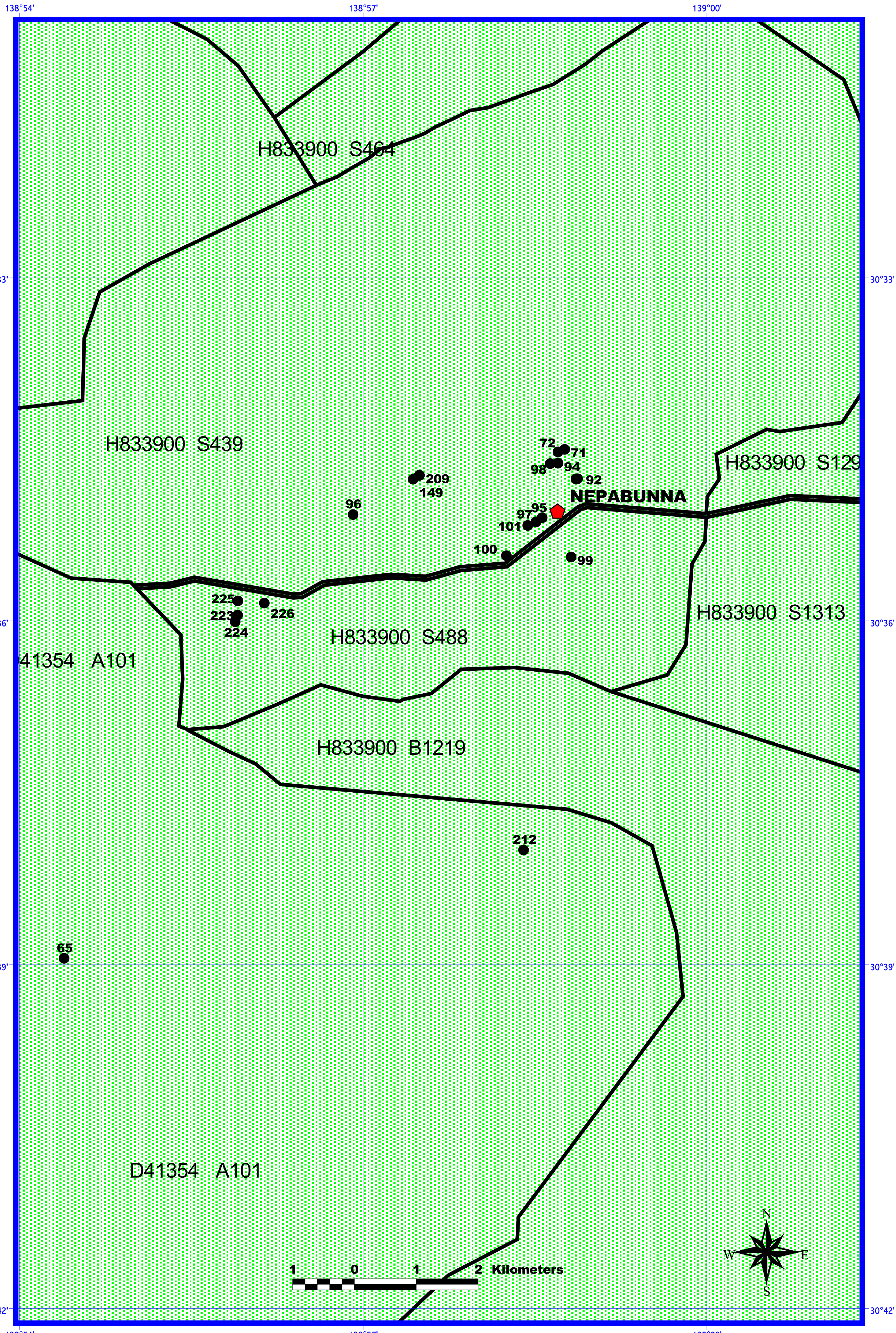


Figure B2

Nepabunna
Bore Location Map

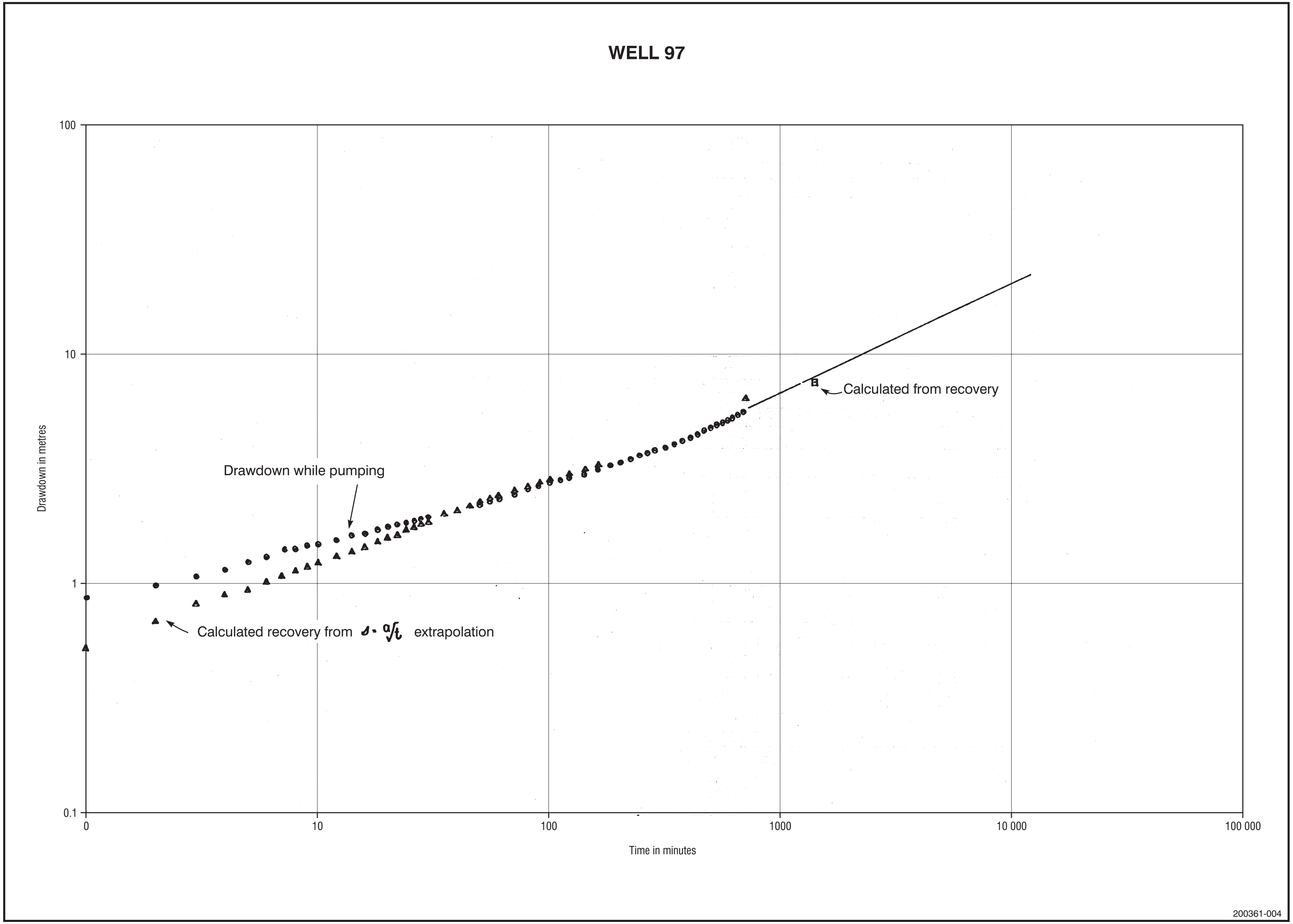


Figure B3 *Pump test results well 97.*

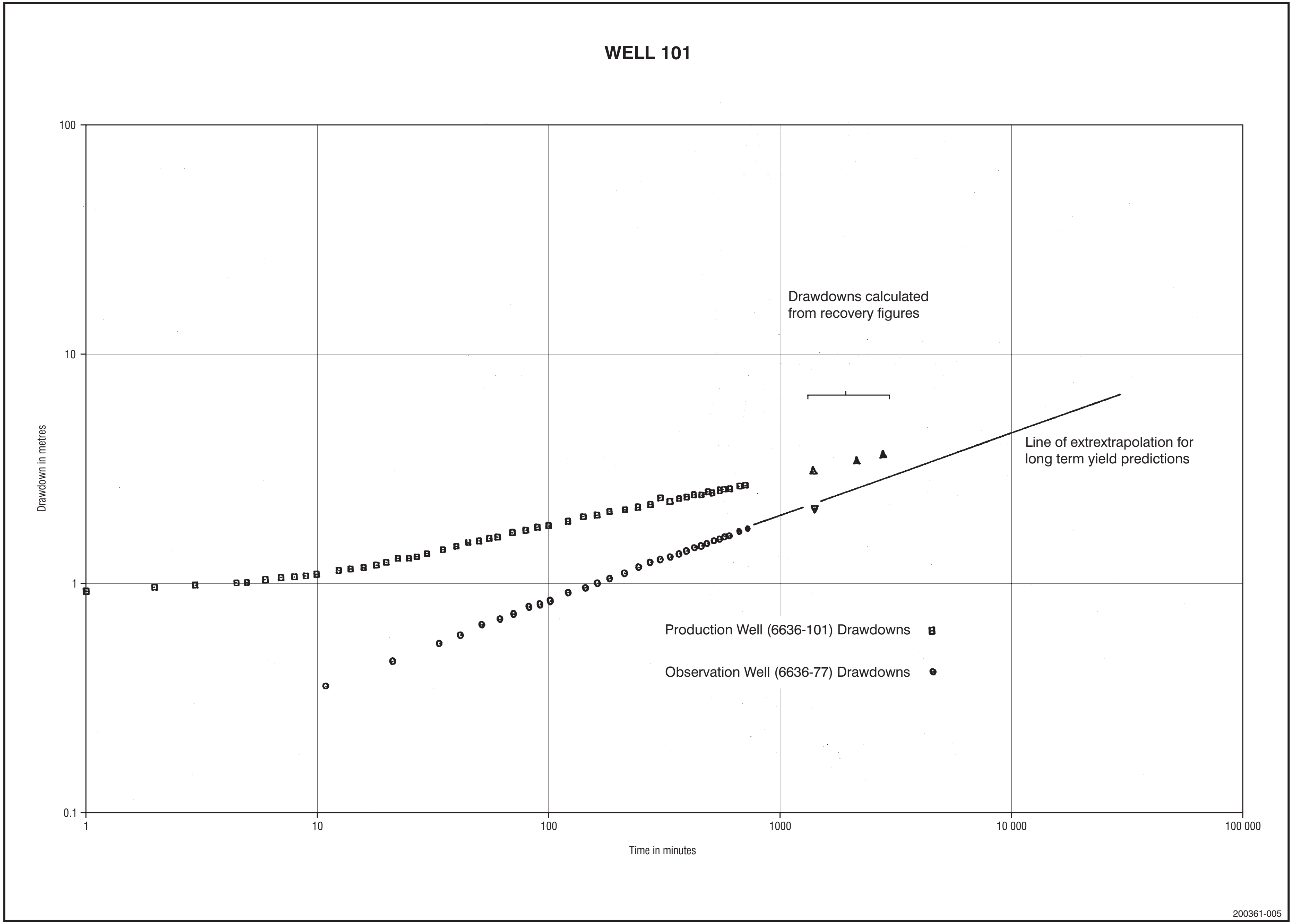


Figure B4 *Pump test results well 101.*

Figure B5. Drawdown observed in well 77 while pumping well 101 8.6 metres away

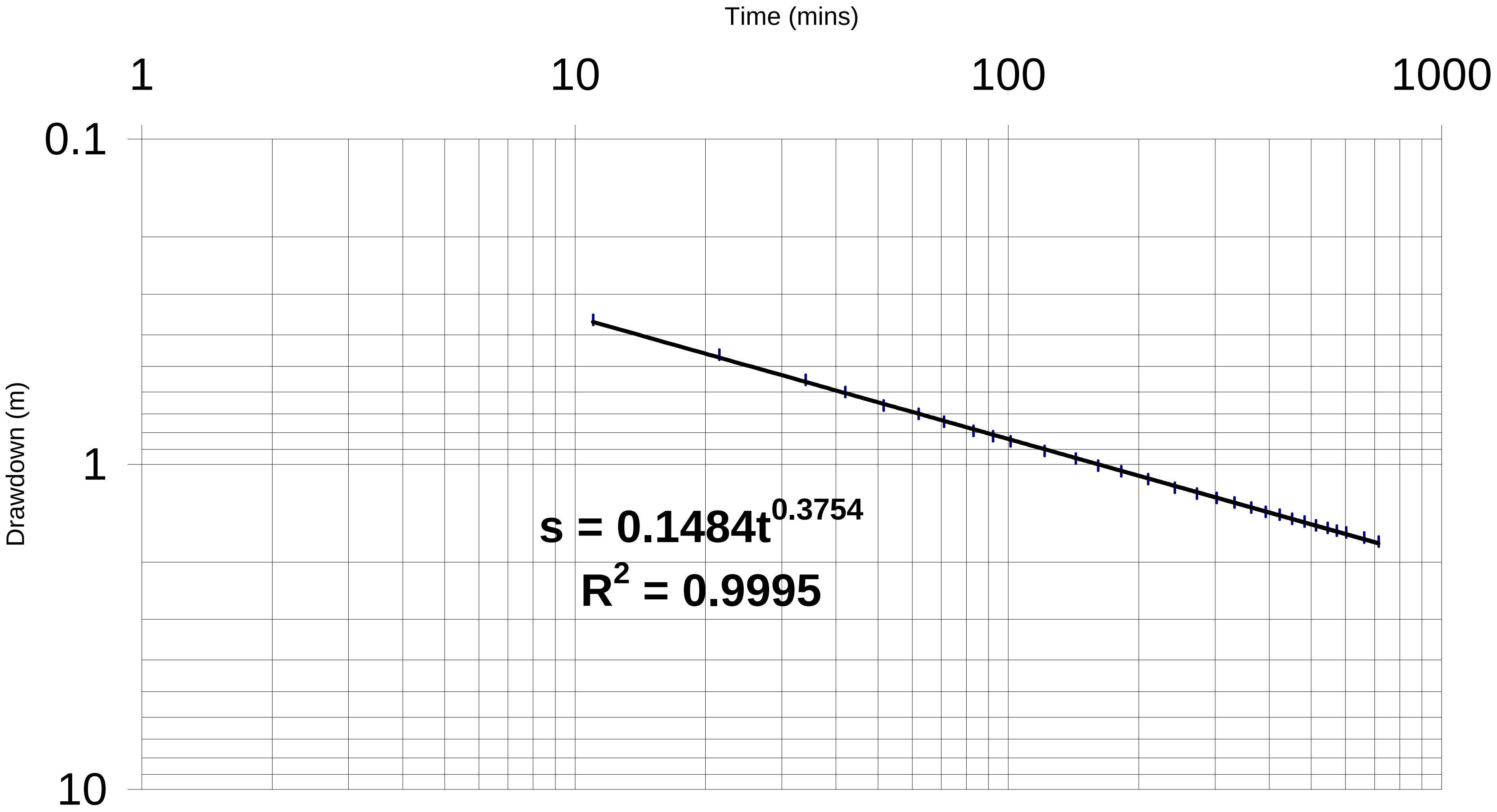


Figure B6. Drawdown observed in well 77 while pumping well 101 8.6m away.

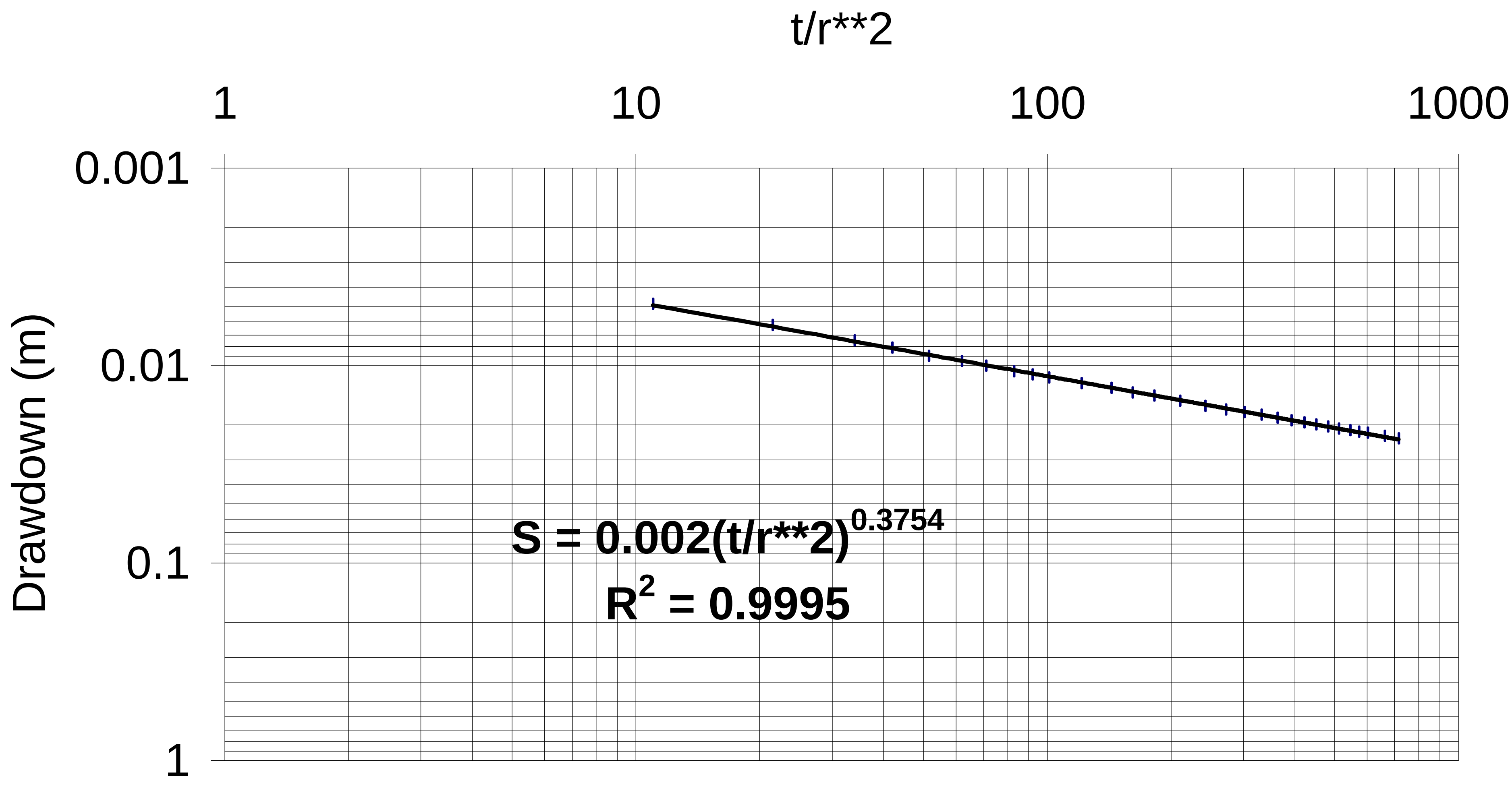
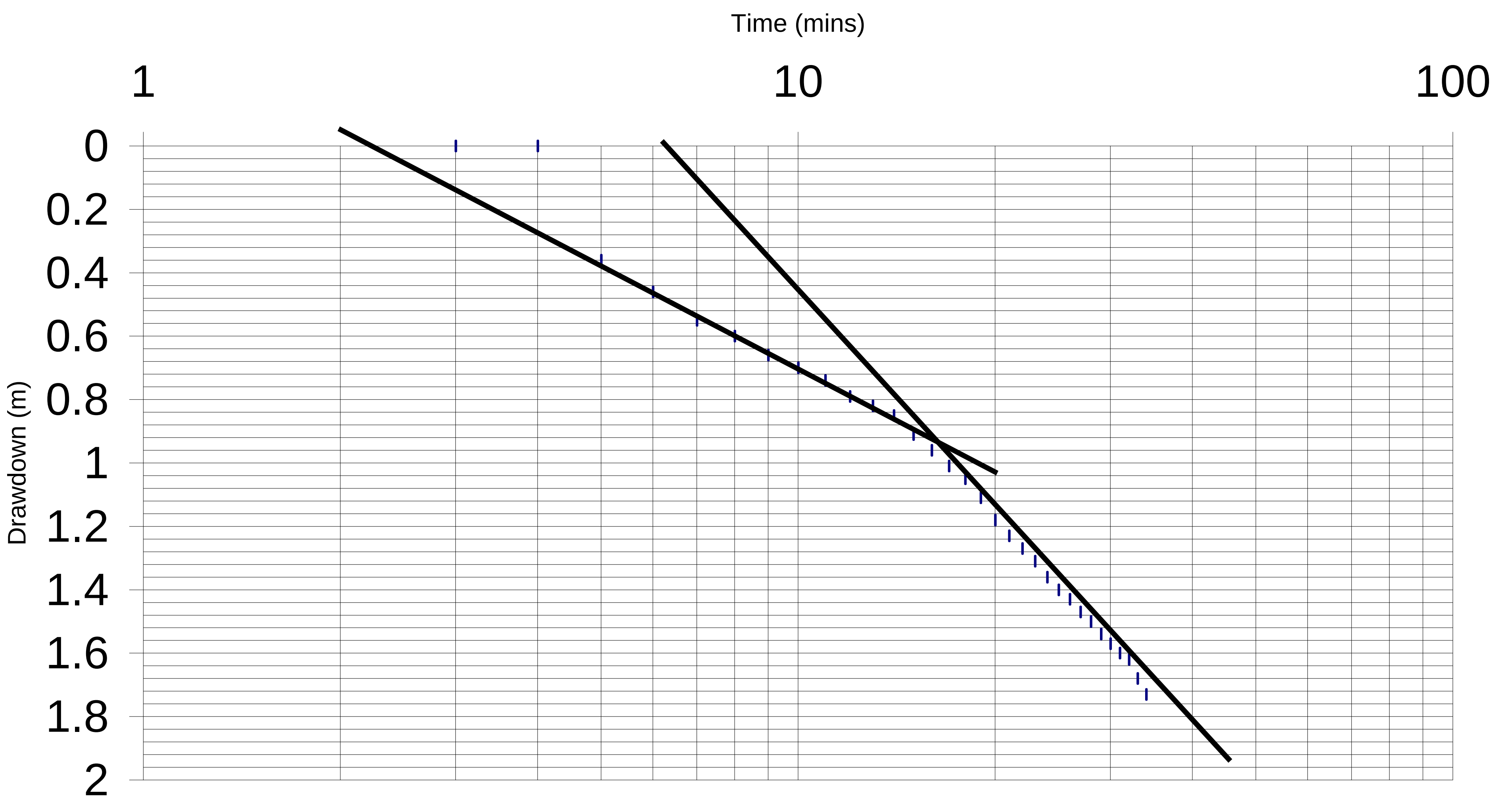


Figure B7. Drawdown observed in well 77 while pumping well 101 8.6 metres away



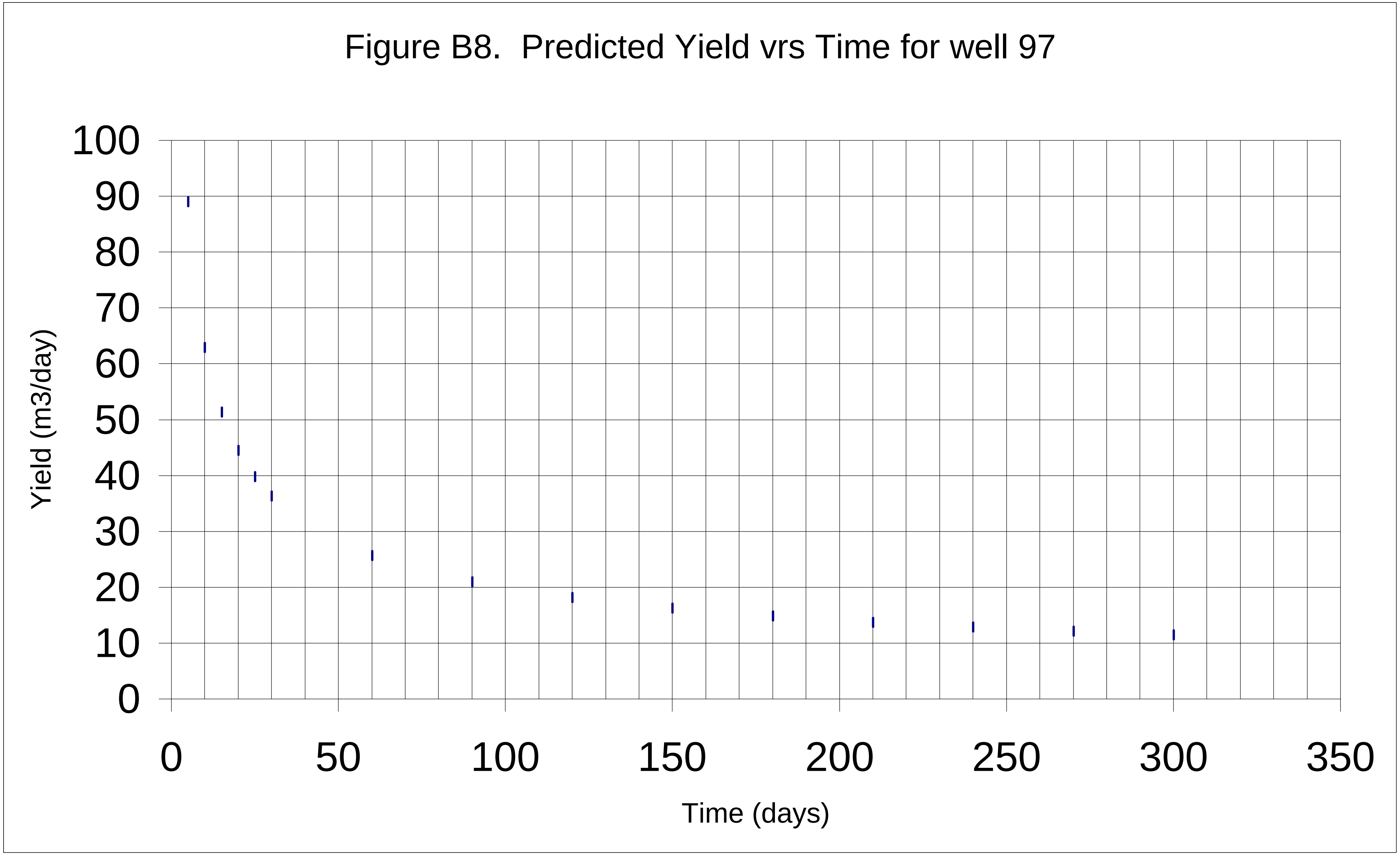


Figure B8. Predicted Yield vrs Time for well 97
Yield (m3/day)
0
10
20
30
40
50
60
70
80
90
100
0
50
100
150
200
250
300
350
Time (days)

Figure B9. Predicted Yield vrs Time for well 101