Broken Hill City Council

# Broken Hill

## Urban Stormwater Master Plan

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# Document History and Status

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# 1. Introduction

The Broken Hill City Council commissioned the development of an Urban Stormwater Master Plan in 2005, to assist in the management of stormwater within the City.

This study was required to address the following key elements:

- Evaluation of the standard of the existing drainage system;
- Establishment of a level of protection appropriate for the catchment;
- Identification of locations in which the existing drainage systems are deficient;
- Preparation of an upgraded drainage system plan showing any new drainage required to achieve the desired level of protection;
- Identification of opportunities for stormwater harvesting and reuse, and the implementation of water quality improvement measures.

This report documents the findings of the Study, including a preliminary costing of the works recommended to upgrade the drainage system.

# 2. Catchment Description

## 2.1 Location & Climate

The City of Broken Hill is located in far western New South Wales, approximately 50 kilometres east of the South Australian border. Broken Hill is subject to a semi-arid climate, with the mean maximum temperature at its greatest in January at 32.7°C. An average annual rainfall of 253 mm occurs on an average of 48 days each year. Council have characterised rainfall during the summer months to be predominantly high intensity, short duration events rather than prolonged periods of precipitation.

## 2.2 Catchment Extents

The catchment situated within the boundaries of the Broken Hill City Council represents an area of approximately 180 km$^2$. This study focussed on a portion of the urban Council area (roughly 12.6 km$^2$), which has been delineated into five separate catchments based on outfall location (refer Figure 2.1):

- The Living Desert;
- Mulga Creek;
- Cemetery Creek;
- Railwaytown;
- South Broken Hill.

The forms of land use within these catchments included residential, light industrial, and commercial premises, schools, Council reserves, and sports and community facilities.

The remaining catchments constituting the urban Council area (defined as the 'City Zone' in the Council GIS database) were excluded from the study, and are displayed as 'External Catchments' on Figure 2.1. These catchments primarily included undeveloped land, and built-up areas in close proximity to the drainage system outfalls (ie. areas that Council indicated were not particularly prone to flooding). The Broken Hill Airport and mining zone along the line of lode were also excluded from the study, as they have independent stormwater drainage systems.

The existing stormwater drainage system within the urban Broken Hill catchment consists largely of overland flows along concrete kerb and gutter channels, which direct flows into the main earth channels emerging out of the City and discharge into ephemeral creeks. The only 'conventional' underground drain is the Argent Street system in the Central Business District.

## Figure 2.1 Catchment Plan

# 3. Level of Protection

In assessing any drainage system a decision has to be made as to the level of protection that is to be used in determining whether or not the system is satisfactory.

The level of protection is generally stated as an average number of years over which it would be expected that the capacity of the drainage system would be exceeded once. For example, a 5 year Average Recurrence Interval (ARI) level of protection indicates that the system capacity will be exceeded, on average, once every 5 years.

There is often a significant margin between the point at which the capacity of an underground drainage system is reached and the point at which damage is caused by flooding. Broken Hill is characterised by a lack of underground drainage systems and a reliance on roadways to provide flow paths.

The use of the road network to convey stormwater flows results in a nuisance (and in some cases temporary interruption) to traffic. The severity and frequency of the nuisance could in some locations be considered undesirable given the relative importance of the road.

To measure the performance of the road network as a stormwater flow path, it is considered appropriate to calculate the flow capacity at a ‘top of kerb’ water level. Notwithstanding this measure, judgment is still required to be exercised as to whether this flow condition is acceptable at each location (ie. with respect to the road being trafficable).

Determining an appropriate level of protection for a drainage system also requires the exercise of engineering judgement to balance the cost of the works required to achieve a particular level of protection against the benefits obtained. As capital costs rise with increased level of protection; there must be tangible benefits if a high level of protection is specified.

Current thinking on appropriate levels of protection is that for minor overland flow paths on low trafficked Council roads a relatively low level of protection is satisfactory, provided any inconvenience experienced once the capacity is exceeded is of nuisance value only and that the system is on-grade (ie. major flows are conveyed downstream to an outfall point during major storm events).

A greater level of protection can be justified for major overland flow paths, especially those which pass through private property, an easement, or a main road. In these cases flows in excess of the capacity are likely to lead to flooding of private property or to significant disruption to traffic.

Where trapped low points exist within the catchment and the only overland flow path for flows in excess of the capacity of the drainage system is through adjacent properties, a very high level of protection (100 year ARI) is desirable.  However the cost associated with achieving such a result in most of these situations can be prohibitive.  Each of these cases requires individual assessment to determine a suitable level of protection.

Based on the above and the results of the hydrological modelling that have established the current level of protection provided (refer Section 5), the target standard of protection which has been considered by this report is as follows:

- 100 year ARI protection for major overland flow paths;
- 5 year ARI underground drainage standard where a 100 year ARI standard overland flow path is available.

# 4. Hydrological Modelling

## 4.1 Introduction

Hydrological modelling was performed to determine peak flows occurring in drainage systems throughout the catchment for a range of Average Recurrence Interval (ARI) events. Comparing these peak flows against the known capacity of each system allows evaluation of existing standards, and identification of problems requiring future improvement.

## 4.2 Hydrological Model

Hydrological modelling of the catchment was undertaken using the ILSAX model “Program for Urban Stormwater Drainage Design and Analysis” (O’Loughlin, 1993). ILSAX is a computer-based rainfall-runoff routing program combining flows through a drainage network. Runoff from each subcatchment is generated by the time-area method according to the specified rainfall temporal data, the rainfall loss model and the time of concentration specific to that subcatchment. Losses are based on an initial and continuing loss model.

The ILSAX model incorporates equations for flow times and drains can be specified in ‘review’ mode to determine their existing capacity, with flows in excess of the drain capacity being directed to the appropriate downstream reach. The underground drainage in Argent St, and the culvert passing between Blende St and the intersection of Beryl St and Chloride St, were modelled in this manner.

The drains can also be specified in ‘design’ mode, which determines the size of drain that will provide a level of protection equivalent to a particular recurrence interval. The program can be configured to produce an output file which summarises each drain, reporting the maximum peak flow passing through it for a range of storm durations, for each recurrence interval investigated. This capability was used to determine the design flows for the overland flow paths in the Broken Hill catchment.

ILSAX has been used by designers for over a decade, and has been accepted as an industry standard tool in producing realistic results.

## 4.3 Catchment Data Collection

A detailed field inspection was conducted in order to verify road gutter drainage patterns and thus confirm the overland flow paths identified in Council records. This inspection also provided an opportunity to examine areas known to be flooding hotspots, and allowed the identification of potential sites for stormwater harvesting and reuse, and water treatment facilities (including the existing Gross Pollutant Traps).

The scope of the study was refined to focus on the downstream reaches of the overland flow paths identified by Council. The use of contour maps and information gathered during the field inspection enabled the delineation of subcatchments based on the areas contributing flow to these overland flow paths. The reaches were represented in the hydrological model by ‘design’ drains with the same effective grade as the actual overland flow path.

## 4.4 Rainfall Data

Rainfall data for the hydrological model were derived from Australian Rainfall & Runoff (Institution of Engineers Australia, 1987). Rainfall depths and temporal patterns were determined for Average Recurrence Intervals between 1 and 100 years and storm durations from 5 minutes to 72 hours.

## 4.5 Time of Concentration

The time of concentration is the time after the commencement of rainfall at which the whole subcatchment is contributing to flow at the downstream end, and is dependent upon the gutter slope and maximum length of gutter flow. This value was calculated for each individual subcatchment.

## 4.6 Rainfall Loss Model

Data for each subcatchment was specified individually to represent the proportion of that area that is deemed to be impervious (eg. rooves, paved areas). The remainder of the area has been assumed to be pervious (eg. grass, garden). Several sample blocks representative of residential areas within the different catchments were identified by inspection of aerial photography. The impervious area of each block was measured from aerial photography.

A field inspection of the street network indicated that very few allotments had downpipes directly connected to the street watertable. In a majority of instances downpipes discharged onto paved, gravel or pervious areas within the allotment. Despite some degree of ponding occurring within allotments, it can reasonably be expected that flows eventually reach the street water table, especially in higher ARI events.

The impervious area can be divided into that which has direct and indirect connection to the stormwater system for runoff coefficient input into ILSAX. Alternatively the time of concentration for the subcatchment can be adjusted to account for the relative proportion of directly and indirectly connected impervious area. The latter approach was adopted in this Study, and the roof to gutter component of the time of concentration was increased to 10 minutes (ie. twice that commonly adopted for a catchment that is directly connected).

Table 4.1 displays the results of the runoff coefficient analysis for the sample blocks.

**Table 4.1 Impervious Fraction for Sample Blocks**

| Catchment | Directly Connected Impervious Fraction |
|---|---|
| The Living Desert | 0.36 |
| Mulga Creek 1 | 0.42 |
| Mulga Creek 2 | 0.39 |
| Cemetery Creek | 0.34 |
| Railwaytown | 0.33 |
| South Broken Hill | 0.43 |

This enabled the runoff coefficient for each subcatchment to be accurately estimated, based on its location and the proportion of impervious area compared to the sample blocks. The runoff coefficient analysis identified a clear trend across the catchments (refer Figure 4.1).

Figure 4.1 Runoff Coefficient Trends Across Catchments

Table 4.2 indicates the ranges of values used for modelling of various land use areas throughout the catchment.

**Table 4.2 Impervious Fraction for Various Land Uses and Regions**

| Land Use | Directly Connected Impervious Fraction |
|---|---|
| Residential | 0.3 – 0.5 |
| Shopping Centres, Carparks, Roads | 0.9 |
| Commercial | 0.6 - 0.7 |
| Light Industrial | 0.7 - 0.8 |

ILSAX determines the rainfall losses from the impervious area in each subcatchment by subtracting an initial loss from the rainfall hyetograph. Throughout the modelling process for this Study, this initial loss value used for impervious areas was 1 mm. All remaining rainfall is considered to produce runoff.

This Study utilised the feature of ILSAX allowing the user to define the coefficient values that determine runoff from pervious areas. An initial loss of 35 mm and a continuing loss of 3 mm/hr from the rainfall hyetograph were used to determine the runoff from pervious areas.

## 4.7 Execution of the ILSAX Model

The ILSAX model was executed for the 1, 2, 5, 10, 20, 50 and 100 year ARI events, thus allowing the existing level of protection of the selected overland flow paths to be accurately assessed.

# 5. Existing Drainage System Performance

## 5.1 Assessment of the Capacity of the Existing Drainage System

Adelaide based surveyors Allsurv were engaged to conduct detailed engineering survey of road and open channel cross-sections and drainage infrastructure, at critical locations along the defined overland flow paths. Where underground drainage was present (ie. Argent St), drain data including type, size, grade and location were collected from Council records.

This data was supplemented with survey of the ‘north’, ‘middle’, and ‘south’ drains carried out by local surveyor Graham Howe in 1992. The section of the ‘north’ drain between the junction with the ‘middle’ drain and Bagot St was ignored in the assessment of the capacity of the existing drainage system. Field inspection indicated that this section of the ‘north’ drain appeared disused, and that the vast majority of flows from this catchment would drain to Beryl St via the downstream section of the ‘middle’ drain.

The capacities of the overland flow path cross-sections and drainage infrastructure were calculated using Manning’s Equation. The road cross-sections were deemed to be at capacity (ie. flowing full) when the height of flow reached the lowest top of kerb level. It is important to note that this criteria does not infer that the road will be trafficable at all times when flow is contained within the road profile.

Similarly the open channel cross-sections were deemed to be at capacity when the height of flow reached the top of the main channel wall (ie. flow over the floodplain or adjacent roadway was not considered). Thus there was no allowance for freeboard in the calculation of overland flow path capacities.

The capacity of each individual cross-section and drainage structure was then compared to the calculated ARI flow in the corresponding drain reach, as determined using the ILSAX model. This revealed the existing level of flood protection within the catchments. Manipulation of this data using the GIS software package MapInfo enabled maps to be created which display the standard of the existing drainage system, and these are discussed in greater detail later in this Section.

## 5.2 Calibration of ILSAX Model with Known Flooding Hotspots

The ILSAX model was initially used to investigate the performance of the existing drainage system for a range of recurrence intervals. The results indicated that flooding problems existed in many drainage systems throughout the catchment. In order to validate the accuracy of the model variables, known drainage problems and their frequency of occurrence, as indicated by Council staff, were compared with the model output. The information gathered from this process suggested problems in locations that the model also reported.

Despite comprehensive flooding records not being available, the model is considered to reasonably reflect the behaviour of the catchment, provides findings on the performance of the existing system, and provides a consistent basis on which to make recommendations on future upgrading of the drainage network.

## 5.3 Existing Drainage System Assessment

This Section describes the results of the hydrological modelling of the existing drainage system within The Living Desert, Mulga Creek, Cemetery Creek, Railwaytown, and South Broken Hill catchments. These results have been used to identify the level of protection provided to the community in each of the catchments, enabling an assessment to be made of locations where improvement to the level of protection is required. For the sake of brevity, only significant drainage systems or those that provide a relatively low level of protection are described below. Those not discussed are considered to provide a sufficient level of protection.

### 5.3.1 The Living Desert Catchment

Figure 5.1 illustrates the standard of the existing drainage system in The Living Desert catchment. The northern portion of the catchment drains to the east via Wyman St, which has been shown to provide a low level of protection (2 year ARI standard) upstream of Brazil St, and a 100 year ARI standard downstream.

The existing drainage system in the southern portion of the catchment has been shown to have greater than or equal to a 20 year ARI standard of protection, except for the low points in McCulloch St and Murton St (ie. at either end of the open channel adjacent to Fisher Ln), which have less than a 1 year ARI standard (refer Photo 1).

**Photo 1**
Looking upstream along Fisher Ln from the intersection with Murton St

It is worth noting that the use of a trafficable portion of Radium St (between Murton St and Brooks St) as a stormwater channel presents a serious safety hazard during significant rainfall events. This issue is discussed in greater detail in Section 6.

### 5.3.2 Mulga Creek Catchment

Figure 5.2 illustrates the standard of the existing drainage system in the Mulga Creek catchment. The north-western portion of the catchment drains to Iodide St (part of State Highway 8) via several overland flow paths. The analysis has highlighted several locations along these flow paths which provide a standard of protection less than 2 years. In particular, the northern ends of Chloride St and Oxide St, and Iodide St north of Wolfram St, have been shown to provide less than a 1 year ARI standard of protection.

Blende St and the twin pipe system through to Beryl St have been shown to have a 1 year ARI standard of protection, while the downstream flow path of Chloride St and Beryl Ln provide less than a 1 year ARI standard of protection.

These findings are consistent with anecdotal and photographic evidence of historical flooding problems along Iodide St (particularly at the intersection with Wolfram St, refer Photo 2), the low point in Blende St to the west of Chloride St, at the intersection of Beryl St and Chloride St, and down Beryl Lane.

**Photo 2**
Looking upstream along Wolfram St at the intersection with Iodide St

The underground drain in Argent St is able to cater for flows from the 2 year ARI event, while the overall Argent St system (ie. the combined capacity of the underground drain and road cross-section) has roughly a 10 year ARI standard of protection.

To the east of Iodide St the catchment is served by a series of open channels (both concrete lined and earthen), which have been shown to provide a 100 year ARI standard of protection. It is important to note that although the standard of this portion of the existing drainage system is high serious drainage problems still exist; primarily the discharge of flows from the concrete lined channel on Beryl St (ie. the downstream portion of the ‘middle’ drain) across Bagot St, and the use of a trafficable portion of Rhodenite St as a stormwater channel. These issues are discussed in greater detail in Section 6.

### 5.3.3 Cemetery Creek Catchment

Figure 5.3 illustrates the standard of the existing drainage system in the Cemetery Creek catchment. The catchment drains to Cemetery Creek via a large number of overland flow paths, and these are seen to provide a varied standard of protection. The eastern portion of the catchment drains via Morgan St, Jones St and Thomas St, which range from less than a 1 year ARI to a 5 year ARI standard. The analysis indicates that the eastern end of Rakow St has a 1-2 year ARI standard. The remaining overland flow paths provide in excess of a 20 year ARI standard of protection.

### 5.3.4 Railwaytown Catchment

Figure 5.4 illustrates the standard of the existing drainage system in the Railwaytown catchment. The findings indicate that the standard of protection provided by the main overland flow paths is very low, particularly in the upstream portion of the catchment. Upstream of Nicholls St the main overland flow paths are generally seen to provide less than a 1 year ARI standard of protection, with the exception of Mercury St to the west of Bismuth St.

A large number of survey cross-sections were taken at this location, and this portion of Mercury St ranges between a 1 year ARI and a 100 year ARI standard. The analysis indicates that the cross-sections which intersect property driveways provide a very low standard of protection (less than a 1 year ARI to a 2 year ARI), while typically Mercury St provides an adequate standard of protection (in excess of a 10 year ARI). This confirms Council reports of flooding and property inundation at this location.

Downstream of Nicholls St the main overland flow paths provide greater than or equal to a 2 year ARI standard of protection. However the main downstream overland flow path of Creedon St is not trafficable during higher ARI events, as seen below in Photo 3.

**Photo 3**
Looking upstream along Creedon St from the intersection with Gaffney St

### 5.3.5 South Broken Hill Catchment

Figure 5.5 illustrates the standard of the existing drainage system in the South Broken Hill catchment. The analysis indicates that the current standard of protection is generally low (1-2 year ARI). At the northern end of the catchment the open channel between Patton St and Wilson St has less than a 1 year ARI standard of protection (refer Photo 4).

**Photo 4**
Looking downstream from Patton St along the open channel

The main overland flow path for the catchment is Comstock St, which has a standard of protection ranging from less than a 1 year ARI to a 5 year ARI (a 2 year ARI standard is typical). These findings are consistent with historical observations of flooding down Comstock St. The main flow path for the eastern portion of the catchment is Duff St, which is seen to provide less than a 1 year ARI standard of protection. At the downstream end of the catchment Knox St has a standard of protection ranging from a 1 year ARI to a 10 year ARI.

Figure 5.1 Existing Drainage Standard Map for The Living Desert Catchment

Figure 5.2 Existing Drainage Standard Map for Mulga Creek Catchment

## Figure 5.3 Existing Drainage Standard Map for Cemetery Creek Catchment

## Figure 5.4 Existing Drainage Standard Map for Railwaytown Catchment

## Figure 5.5 Existing Drainage Standard Map for South Broken Hill Catchment

# 6. Stormwater Related Road Safety Issues

The field investigation identified several locations where existing stormwater management practices have serious implications on traffic and road safety. The following discussion gives a broad overview of the issues pertinent to these locations, and should not be considered to constitute a detailed road safety audit.

## 6.1 Beryl St/Bagot St Intersection

The Beryl St/Bagot St intersection is located on the major overland flow path for the Mulga Creek catchment. Due to the large contributing catchment area this overland flow path receives significant flows, even during lower ARI events. The existing concrete lined channel adjacent to Beryl St discharges across Bagot St, and along a trafficable portion of Rhodenite St. During rainfall events it becomes impossible for vehicles to traverse Bagot St, and access to properties on Rhodenite St is severely restricted.

It is undesirable for a trafficable road to perform the dual function of a stormwater channel. Aside from drainage and road upgrades, overall road safety could be improved at this location by imposing speed restrictions, installing warning signs/hazard boards, and clearly defining traffic lanes and the floodway (white lines).

## 6.2 Radium St

The portion of Radium St to the east of Murton St forms the downstream section of the main overland flow path for The Living Desert catchment. The road levels on Radium St are significantly lower than those on adjacent roads, and the road profile at this location also incorporates a low flow channel between the Murton St and Brooks St culvert crossings. The capacity of the low flow channel is exceeded even during lower ARI events, causing the inundation of the trafficable portion of Radium St.

Of particular concern at this location is the use of Radium St (including the low flow channel) as a parking lane for the adjacent school. Road safety could be improved at this location by installing warning signs/hazard boards, clearly defining traffic lanes and the low flow channel (white lines), establishing the road as a ‘no parking’ zone, and upgrading the safety rails at the culvert crossings (the existing rails are considered dangerous in the event of a collision).

## 6.3 Other Locations

Photographic evidence indicates that several roundabouts throughout Broken Hill are prone to inundation during rainfall events (eg.Gypsum St/Wills St and Beryl St/Chloride St), as seen in Photo 5.

**Photo 5**
Inundation of roundabout at the intersection of Wills St and Gypsum St

It is also evident that roads which form the downstream overland flow paths in each catchment are prone to complete inundation (eg.Iodide St, Wolfram St, Creedon St, Comstock St). Road safety could be improved at these locations through the provision of depth marker posts.

# 7. Proposed Drainage Master Strategy

## 7.1 Introduction

The cost of establishing a ‘conventional’ 5 year ARI underground drainage network along existing major overland flow paths throughout the five catchments was estimated to be in excess of $20 million. Given the budget constraints of Council it was necessary to refine the scale of proposed works to target specific flooding hotspots within each catchment.

The drainage schemes formulated using this approach adopt a combination of underground drainage, above ground detention storages, and existing overland flow paths to minimise the extent and frequency of inundation of private property, and improve the trafficability of roads during storm events.

The target design standard for each proposed upgrade scheme was site specific, based on constraints identified during field assessment and inspection of the available survey data. It was necessary to preserve the existing ground levels along the overland flow paths to ensure that flows are not impeded during high ARI events. This imposed design constraints relating to the maximum possible drain size (based on the cover available), and the storage volume available for detention.

The alignments of the proposed drains have been selected using the general criteria of maximising the use of existing overland flow paths with sufficient capacity (thus minimising cost), provision of interception drains to alleviate flows in existing overland flow paths with poor capacity, minimising the length of relatively high road flows, and use of public rather than private land. The design pipe size for the main drain elements are presented in Appendix A.

## 7.2 Drainage Upgrade Priorities

The priorities for the works identified have been set according to the following:

- The significance in reduction of flood damages and nuisance as a result of constructing the proposed works;
- The requirements for a proposed drain to be constructed, so that other proposed drains feeding into this drain can subsequently be constructed;
- The availability of Council land required for the upgrade;
- The timing of associated works (such as road upgrades and development of land divisions);
- Funding availability;
- Construction cost.

Application of the above criteria produced a preferred priority hierarchy for staging the construction of these projects.

Large scale drainage upgrades associated with significant State and Regional Roads are considered to be of a high priority. This is due to the fact that these drainage upgrades will improve road trafficability, and are required to facilitate upgrades further upstream in the catchment. Drainage upgrades at locations identified by Council to be flooding hotspots are also considered to be high priority works, as are locations where the frequency of nuisance flooding or the risk of flooding to adjacent properties is likely to be high. The high priority works have been assigned rankings corresponding to the proposed order of construction. Estimates of construction cost have been provided for the high priority works.

Drainage upgrades in low speed, low trafficked side streets throughout the catchment, which are dependent upon the upgrade of downstream systems, have been deemed low priority works. The low priority works have been assigned rankings corresponding to the proposed order of construction. Estimates of construction cost have also been provided for the low priority works.

## 7.3 Indicative Cost Estimates

Costings for the proposed drainage systems have been prepared. The estimated costs for the proposed drainage works include an allowance for construction costs, design, alteration of existing services, and contingencies. A regional index has been applied to account for the additional costs associated with construction work in Broken Hill (ie. increased transport costs for materials etc). The costs do not include GST.

These estimates are prepared for general information only, and it is recommended that an appropriately qualified quantity surveyor be consulted to provide detailed advice regarding construction costs should more definitive estimates be required. It is expected that further development of the design concepts would be required to refine the estimates further.

## 7.4 The Living Desert Catchment Drainage System Upgrades

The proposed drainage upgrades for The Living Desert catchment are shown in Figure 7.1.

### 7.4.1 Low Priority Upgrades

***McCulloch St System***

It is proposed to construct underground drainage along McCulloch St between O'Neill St and McGowen St, and at the downstream end of adjoining roads. The underground system is to discharge into the detention basin proposed to be constructed on the Council reserve to the east of McCulloch St. The outfall from the basin is to be located at Murton St. Based on contours and limited survey of existing ground levels a concept design of the proposed detention basin was carried out.

The maximum storage level in the detention basin was set at 293.80 mAHD to match the natural invert level of the adjacent open channel. Thus the existing overland flow path can be utilised to direct flows in excess of the proposed underground drainage system capacity into the basin, and receive flows in excess of the basin capacity via overtopping. The invert level of the basin was set at 292.80 mAHD such that adequate fall could be achieved for the outfall drain to Murton St. Assuming batter slopes of 1:6 and a storage height of 1 metre, it was estimated that 4000 $m^3$ of storage volume is available at the site.

This basin configuration limits the discharge in the 20 year ARI event to a flow which can be contained within the low flow channel on the northern side of Radium St (1.5 $m^3/s$). This allows Radium St to remain trafficable during storms up to the 20 year ARI event, and alleviates the need to construct an underground outfall along Murton St and Radium St through to The Living Desert. This is a highly desirable outcome given that the construction of such an outfall presents significant financial and engineering constraints; primarily those associated with the drastic changes required to the existing road and ground levels.

It is worth noting that the above works can be completed in a staged process. The construction of the detention basin and outfall is to take place first, thereby affording a greater level of protection for the downstream reaches in the short term. Flows from the upstream catchment can be directed into the basin via the existing overland flow paths until the completion of the upstream underground drainage.

| | |
|---|---|
| **20 year ARI basin, 1 year ARI underground drain:** | $920,000 |
| **20 year ARI basin, 2 year ARI underground drain:** | $990,000 |

Figure 7.1 The Living Desert Catchment Drainage System Upgrades

## 7.5 Mulga Creek Catchment Drainage System Upgrades

The proposed drainage upgrades for the Mulga Creek catchment are shown in Figure 7.2.

### 7.5.1 High Priority Upgrades

***Bagot St Culvert Crossing***

The existing concrete lined channel adjacent to Beryl St discharges across Bagot St into Rhodenite St, resulting in significant disruption to traffic. In order for the intersection of Bagot St and Beryl St to remain trafficable during lower ARI events, it is proposed to construct a culvert crossing at this location to provide a 5 year ARI standard of protection.

It is expected that existing road levels at the intersection will need to be altered to facilitate culvert construction, however the existing overland flow path will be preserved to allow flows in excess of the 5 year ARI to overtop and spill across Bagot St.

The proposed culvert crossing does not improve access to the properties on the northern side of the concrete lined channel downstream of Bagot St. It is not considered economically or technically feasible to extend the culverts in a north-easterly direction along Rhodenite St.

| | |
|---|---|
| **Culvert Crossing Length:** | 30 metres |
| **5 year ARI design:** | $540,000 |

***Kaolin St System***

It is proposed to construct underground drainage at the intersection of Kaolin St with both Beryl St and Blende St. The underground system is to discharge into the detention basin proposed to be constructed on the Council owned land to the east of Kaolin St between Beryl St and Blende St. The outfall from the basin is to be located on Beryl St. Based on contours of existing ground levels a concept design of the proposed detention basin was carried out.

The maximum storage level in the detention basin was set at 307.50 mAHD to match the existing street water table on Kaolin St, which enables flows in excess of the proposed underground drainage system capacity to follow the existing overland flow paths into the basin. The invert level of the basin was set at 306.50 mAHD to match the street water table at the proposed outfall to Beryl St, enabling flows to be discharged directly to the existing downstream overland flow path. Note that the aforementioned basin invert and obvert levels were based on contour information only, as there was no survey available for this particular site.

Assuming batter slopes of 1:6 and a storage height of 1 metre, it was estimated that 4000 $m^3$ of storage volume is available at the site. This basin configuration limits the discharge in the 50 year ARI event to the 5 year ARI flow, and thereby affords a greater level of protection for the downstream reaches.

| | |
|---|---|
| **50 year ARI basin, 2 year ARI underground drain:** | $600,000 |
| **50 year ARI basin, 5 year ARI underground drain:** | $630,000 |

***Iodide St System***

Iodide St forms part of State Highway 8, and constitutes the major overland flow path for the western portion of the Mulga Creek catchment. Iodide St grades to a low point at the intersection with Wolfram St, and ultimately discharges to the Beryl St drain via the concrete lined channel through the Council reserve (the ‘middle’ drain).

Council staff have reported significant disruption to traffic along Iodide St, even in low ARI events. Hence it is proposed to construct underground drainage along Iodide St between Chapple St and Beryl St, in order to reduce the magnitude of overland flow and thereby improve trafficability.

| | |
|---|---|
| **2 year ARI underground drain:** | $1,390,000 |
| **5 year ARI underground drain:** | $1,520,000 |

***Mulga Creek Catchment Wetland***

Refer Section 8.6.2

### 7.5.2 Low Priority Upgrades

***Beryl St System***

An investigation into the possible construction of a detention basin on the Council reserve abutting the intersection of Beryl St and Chloride St was undertaken. The analysis indicated that the small volume of detention storage potentially available had no significant flow mitigation benefit for downstream reaches.

Therefore it is proposed to construct underground drainage between the intersection of Chloride St with Beryl St and Blende St (a low point), and the proposed Iodide St system at the intersection of Iodide St and Cobalt St. The proposed system will also accept flows from the existing twin pipe system between Blende St and Beryl St.

This will serve to reduce the large overland flows that currently disrupt traffic at the intersection of Chloride St and Beryl St, and along Beryl Ln, and also minimise ponding at the low point in Blende St.

| | |
|---|---|
| **2 year ARI underground drain:** | $1,090,000 |
| **5 year ARI underground drain:** | $1,200,000 |

### *Iodide St System Extension*

The analysis of the existing drainage system reported a low level of protection along the overland flow paths to the north-west of Iodide St. It is proposed to extend the Iodide St system along Chapple St, Oxide St, Williams St, Chloride St, and Williams Ln. The construction of underground drainage along these reaches will serve to increase flood protection for adjacent properties and maintain trafficability, by reducing the magnitude of overland flows. It is important to note that these works are able to be completed as a staged process, by extending the Iodide St system up the catchment as the Council works program allows. In this manner it may be possible to complete the drainage works concurrently with road upgrades.

| | |
|---|---|
| **2 year ARI underground drain:** | $2,310,000 |
| **5 year ARI underground drain:** | $2,560,000 |

### *Morgan St System*

An investigation into the possible construction of a detention basin on the Council owned allotment at the intersection of Sulphide St and Morgan St was undertaken. The proposed basin is to receive runoff from the north-western extents of the Mulga Creek catchment (approximately 26 ha) via the major overland flow paths of Morgan Ln and Sulphide St, and will afford a greater standard of flooding protection to properties at the downstream end of Morgan Ln.

Given that there is no underground drainage proposed for this upstream portion of the catchment, flows are to be directed into the basin via existing overland flow paths. The basin is to have no formal outfall, and will discharge via infiltration and evaporation until the maximum storage level is exceeded and overtopping occurs. During the detailed design process it is suggested to assess the feasibility of overtopping directly to Morgan St (rather than the downstream portion of Morgan Ln, which has a very flat grade and is prone to flooding).

Based on contours of existing ground levels a concept design of the proposed detention basin was carried out. Allowing for a buffer zone of 2 metres around the allotment boundary (40 x 100 metres), and assuming batter slopes of 1:6 and a storage height of 1 metre, it was estimated that 2600 $m^3$ of storage volume is available at the site.

Thus the proposed basin is able to detain flows up to the 2 year ARI event (flows in excess of the 2 year ARI event will overtop and continue down the catchment unmitigated).

| | |
|---|---|
| **2 year ARI basin:** | $150,000 |

## Figure 7.2 Mulga Creek Catchment Drainage System Upgrades

## 7.6 Cemetery Creek Catchment Drainage System Upgrades

The proposed drainage upgrades for the Cemetery Creek catchment are shown in Figure 7.3.

### 7.6.1 Low Priority Upgrades

***Rakow St System***

It is proposed to construct underground drainage along the portion of Rakow St between Nicholls St and Harvy St (part of State Highway 8). This is expected to improve trafficability by significantly reducing flows within the road profile, and also serves to minimise flows entering Rakow St from adjoining roads.

| | |
|---|---|
| **2 year ARI underground drain:** | $720,000 |
| **5 year ARI underground drain:** | $790,000 |

***Thomas St System***

It is proposed to construct underground drainage in the north-eastern portion of the Cemetery Creek catchment. This will serve to reduce flows within the road profile and improve the standard of protection for properties abutting Morgan St, Jones St, Thomas St, and their adjoining roads and laneways.

| | |
|---|---|
| **2 year ARI underground drain:** | $1,600,000 |
| **5 year ARI underground drain:** | $1,760,000 |

## Figure 7.3 Cemetery Creek Catchment Drainage System Upgrades

## 7.7 Railwaytown Catchment Drainage System Upgrades

The proposed drainage upgrades for the Railwaytown catchment are shown in Figure 7.4.

### 7.7.1 High Priority Upgrades

***Mercury St System***

It is proposed to construct underground drainage along Mercury St between Bismuth St and Galena St, and at the intersections with adjoining roads. The underground system is to discharge into the detention basin proposed to be constructed on the Council owned land to the south of the intersection of Mercury St and Graphite St. The outfall from the basin is to be located on Cornish St. It is understood that the site is contaminated land and as such we have assumed that an impermeable lining will be required for the basin.

The proposed drainage works will reduce excessive overland flows along Mercury St, and afford a greater level of protection for downstream reaches. It may also be possible to connect the proposed underground drainage directly to the existing supermarket outfall at the intersection of Mercury St and Graphite St. Based on contours and limited survey of existing ground levels a concept design of the proposed detention basin was carried out.

The maximum storage level in the detention basin was set at 308.10 mAHD to match the existing street water table on Mercury St, which enables flows in excess of the proposed underground drainage system capacity to follow the existing overland flow paths into the basin. The invert level of the basin was set at 307.10 mAHD to match the street water table at the proposed outfall to Cornish St, enabling flows to be discharged directly to the existing downstream overland flow path.

Assuming batter slopes of 1:6 and a storage height of 1 metre, it was estimated that 8000 m$^3$ of storage volume is available at the site. This basin configuration limits the discharge in the 100 year ARI event to the 5 year ARI flow.

| | |
|---|---|
| **100 year ARI basin, 2 year ARI underground drain:** | $770,000 |
| **100 year ARI basin, 5 year ARI underground drain:** | $800,000 |

***Wills St System***

It is proposed to construct a detention basin on the Council land at the intersection of Wills St and Nicholls St, and underground drainage to convey flows from Wills St (including the intersection with Nicholls St) into this basin. The outfall from the basin is to be located at the intersection of Burke St and Nicholls St.

In order to maintain the existing oval for sporting and recreational use, it is proposed to utilise part of Nicholls St for the basin area. This will involve reducing the road width on Nicholls St between Wills St and Burke St to approximately half of its current width, allowing through traffic and access to properties fronting Nicholls St to be maintained. Based on contours and limited survey of existing ground levels a concept design of the proposed detention basin was carried out.

The maximum storage level in the detention basin was set at 300.40 mAHD to match the existing street water table on Wills St, which enables flows in excess of the proposed underground drainage system capacity to follow the existing overland flow paths into the basin. The invert level of the basin was set at 299.40 mAHD to match the street water table at the proposed outfall to the Burke St/Nicholls St intersection, enabling flows to be discharged directly to the existing downstream overland flow path.

Assuming batter slopes of 1:6 and a storage height of 1 metre, it was estimated that 2000 $m^3$ of storage volume is available at the site. When operating in series with the proposed Mercury St basin, this basin configuration limits the discharge in the 20 year ARI event to the 5 year ARI flow, and thereby affords a greater level of protection for the downstream reaches. It also provides greater flood protection to adjacent properties. When operating in isolation (ie. without the presence of the proposed Mercury St basin) this basin configuration limits the discharge in the 10 year ARI event to the 5 year ARI flow.

| | |
|---|---|
| **20 year ARI basin, 2 year ARI underground drain:** | $460,000 |
| **20 year ARI basin, 5 year ARI underground drain:** | $490,000 |

### *Mercury St Road Reconstruction*

The portion of Mercury St to the west of Bismuth St has a history of flooding problems, with properties on both the northern and southern sides of the road frequently experiencing inundation during significant rainfall events.

Visual inspection and survey of the site indicated that this portion of Mercury St has a very flat longitudinal grade (less than 0.25%) towards Bismuth St. The crown of road levels were shown to be high relative to the water table and kerb levels, resulting in a low capacity in the kerb and gutter channel. Furthermore the adjacent property levels were often shown to be lower than the kerb levels, particularly at driveway entrances.

A combination of the aforementioned factors results in the properties on this portion of Mercury St being extremely susceptible to inundation, even during lower ARI events. A HEC-RAS backwater curve model was compiled and executed for this reach, which indicated that there is no significant backwater effect acting to increase the magnitude and/or frequency of flooding at this location.

It is not considered cost effective to construct underground drainage at this location, given that it lies within the upstream portion of the catchment, necessitating a very long outfall drain. Due to the comparatively small contributing catchment size, it is expected that a sufficient level of flood protection can be achieved for adjacent properties by reconstructing the streetscape. It is therefore recommended that the road profile be altered by lowering the crown of road level and raising top of kerb and driveway entrance levels, in order to increase the capacity of the kerb and gutter system.

| | |
|---|---|
| **Road Reconstruction and Streetscaping** | $370,000 |

### 7.7.2 Low Priority Upgrades

***Wills St System Extension***

Extending the proposed underground drainage system along Wills St to the intersection with Gypsum St will serve to increase flood protection for adjacent properties and maintain trafficability, by reducing the magnitude of overland flows.

| | |
|---|---|
| **2 year ARI underground drain:** | $440,000 |
| **5 year ARI underground drain:** | $490,000 |

Figure 7.4 Railwaytown Catchment Drainage System Upgrades

## 7.8 South Broken Hill Catchment Drainage System Upgrades

The proposed drainage upgrades for the South Broken Hill catchment are shown in Figure 7.5.

### 7.8.1 Low Priority Upgrades

***Patton St System***

It is proposed to construct a detention basin on the Council reserve on Wilson St (between South St and Comstock St), and underground drainage to convey flows from the intersection of South St and Patton St into this basin. Based on contours and limited survey of existing ground levels a concept design of the proposed detention basin was carried out.

The maximum storage level in the detention basin was set at 298.85 mAHD to match the natural channel invert at the southern (downstream) end of the laneway adjoining Patton St, which forms the entry point to the basin. This enables flows in excess of the proposed underground drainage system capacity to follow the existing overland flow path into the basin. There appears to be a lack of cover available in the aforementioned laneway, which limits the feasible drain size at this location. Thus the standard of the proposed system is restricted to a 1-2 year ARI.

The invert level of the basin was set at 298.00 mAHD to match the street water table on Wilson St, enabling flows to be discharged directly to the existing downstream overland flow path. This alleviates the costs associated with constructing a formal underground outfall to the southern end of Comstock St. Assuming batter slopes of 1:6 and a storage height of 0.85 metres, it was estimated that 3600 $m^3$ of storage volume is available at the site.

This basin configuration limits the discharge in the 20 year ARI event to the 2 year ARI flow, and thereby affords a greater level of protection for the downstream reaches. It is worth noting that the above works can be completed in a staged process. The construction of the detention basin and outfall is to take place first, and flows from the upstream catchment can be directed into the basin via the existing overland flow paths until the completion of the upstream underground drainage.

| | |
|---|---|
| **20 year ARI basin, 1 year ARI underground drain:** | $490,000 |
| **20 year ARI basin, 2 year ARI underground drain:** | $510,000 |

Initial investigations into the possible construction of a detention basin in the southern portion of the catchment were abandoned due to a lack of suitable land being available.

Figure 7.5 South Broken Hill Catchment Drainage System Upgrades

## 7.9 Proposed Drainage System Upgrades Summary

It is recommended that the proposed drainage upgrades defined as 'high priority' in this report are placed within a short to medium term works schedule. These schemes are expected to improve the standard of protection at existing flooding hotspots, improve the safety and trafficability of significant roads, and maximise the stormwater reuse potential within Broken Hill.

Table 7.1 includes a summary of costs for the high priority drainage upgrade schemes (assuming the higher ARI option for underground drainage is adopted ie. 5 year ARI):

**Table 7.1 High Priority Drainage Upgrades**

| Rank | Proposed Drainage Upgrade Scheme | Cost |
|---|---|---|
| 1 | Mulga Creek Catchment Wetland | $1,200,000 |
| 2 | Mercury St Road Reconstruction | $370,000 |
| 3 | Iodide St System | $1,520,000 |
| 4 | Bagot St Culvert Crossing | $540,000 |
| 5 | Mercury St System | $800,000 |
| 6 | Kaolin St System | $630,000 |
| 7 | Wills St System | $490,000 |
|  | **Total** | **$5,550,000** |

It is recommended that the proposed drainage upgrades defined as 'low priority' in this report be undertaken as part of a long term works schedule. These schemes are expected to confer flood mitigation benefits (ie. reduction in nuisance flooding) along other major overland flow paths.

Table 7.2 includes a summary of costs for the low priority drainage upgrade schemes (assuming the higher ARI option for underground drainage is adopted ie. 2 or 5 year ARI):

**Table 7.2 Low Priority Drainage Upgrades**

| Rank | Proposed Drainage Upgrade Scheme | Cost |
|---|---|---|
| 1 | Morgan St System | $150,000 |
| 2 | Patton St System | $510,000 |
| 3 | McCulloch St System | $990,000 |
| 4 | Beryl St System | $1,200,000 |
| 5 | Wills St System Extension | $490,000 |
| 6 | Rakow St System | $790,000 |
| 7 | Thomas St System | $1,760,000 |
| 8 | Iodide St System Extension | $2,560,000 |
|  | **Total** | **$8,450,000** |

## 7.10 Funding Opportunities

It is highly likely that some of the proposed drainage schemes will be eligible for subsidy through State or Federal Government initiatives. Under the Australian Government Water Fund large-scale projects may be eligible for funding through the Water Smart Australia Scheme, while small-scale projects may be eligible for funding through the Community Water Grants Scheme. Both of the aforementioned schemes are designed to support projects targeting water savings/efficiency, water reuse/recycling, and improvements to surface and groundwater health.

As part of their investment strategy the Lower Murray-Darling Catchment Management Authority have dedicated a budget of $0.5 million to stormwater works within the Broken Hill catchment. In order to qualify for funding the proposed works must be targeted to reduce the level of discharge and improve water quality of major urban stormwater drainage outfalls that discharge directly to rivers and waterways. Several of the aforementioned schemes satisfy these criteria, in particular the Mulga Creek Catchment Wetland and ASR related projects (refer Section 8).

Funds are also available for the proposed drainage works under the Floodplain Management Program, an ongoing initiative run by the NSW Department of Infrastructure, Planning and Natural Resources (DIPNR). The funds assist Councils in flood mitigation projects such as the construction of underground drainage and detention basins, and voluntary purchase and/or raising of houses in areas at high flood risk. Generally the NSW Government provides two thirds of the funding, while Council is expected to make up the balance. The funds are accessible to Council through an annual application process, however under special conditions applications submitted outside of the structured deadlines are considered.

The NSW Government has also indicated that grants are available for local Councils to develop stormwater harvesting and reuse projects such as capturing rainwater for use on sporting fields and golf courses.

A cost-sharing relationship is likely to be feasible with the NSW Roads and Traffic Authority (RTA) for proposed drainage works associated with the upgrade of State Roads (such as Iodide St and Rakow St which form part of State Highway 8) and Regional Roads (such as Gypsum St). Generally drainage infrastructure can be constructed concurrently with pavement rehabilitation works and the sealing and/or widening of road shoulders.

State Roads are capitalised as RTA assets, and therefore it is the responsibility of the RTA to fund and determine priorities for their upgrade. Regional Roads are capitalised as Council assets, and therefore it is the responsibility of Council to fund, prioritise, and carry out upgrade works. However works involving Regional Roads may qualify for funding under the REPAIR (Repair and Improvement of Regional Roads) program run by the RTA. The program provides for a State Government contribution of 50% of the project cost, with projects selected on a merit basis across each RTA Region.

Representatives from the aforementioned agencies were contacted as part of this Study to be informed of the nature and scope of the proposed drainage upgrades for Broken Hill, and to notify them of Council's intent to apply for funding in the near future. Thus a network of contacts has been developed to facilitate the funding application process for Council.

## 7.11 Managing Runoff from Future Development

Council staff have indicated that the redevelopment of individual sites is expected to be limited, however the development of group dwellings and community facilities (eg. retirement villages and nursing homes) occurs from time to time.

Therefore the net effect of redevelopment of individual sites is not expected to significantly impact upon the stormwater drainage system. Larger scale developments are likely to result in a greater proportion of impervious area being discharged to the street drainage system, with a greater efficiency. This has the impact of increasing peak flows and consequently reducing the level of protection provided by the existing drainage network.

It is recommended that new developments be required to provide on-site detention to reduce peak flows to a desirable maximum limit. It is expected that the detention requirement would limit the peak runoff from the site, such that the existing drainage system can provide an adequate level of protection. This limit could be based on a condition requiring the post-development flow to match the pre-development flow.

### 7.11.1 Principles of Development Control

Council requires 'generic' requirements to be included in the Development Plan, for use in ongoing assessment of development applications. The following criteria are suitable for this purpose:

- Development and associated works within the Council area must not adversely affect the level of floodwaters on adjoining properties;
- Development must establish a building floor level for the site to minimise the risk and hazard of inundation:
  - Where the proposed development site is not located in close proximity to a major overland flow path or creek, floor levels must be a minimum of 300mm above the adjacent top of kerb level
  - Where the proposed development site is adjacent to a major overland flow path or creek as defined by this report, floor levels must be a minimum of 300 mm above the 100 year ARI flood level;
- Development must include Water Sensitive Urban Design (WSUD) features allowing for the retention and re-use of stormwater, and in particular:

  - the collection, storage and reuse of runoff from rooves
  - treatment of runoff from paved and carpark areas using swales or other appropriate devices;
- Where new development will result in an increase in impervious site coverage, on-site stormwater detention structures/techniques must be provided to limit the post-development peak discharge rate of stormwater from the site (including roof and ground surface runoff) to the pre-development peak flow rate during both the 5 year and 100 year ARI events;
- New development should ensure that all roof areas are directly connected to the street water table, and where practicable should not contribute ground surface runoff to adjoining properties;
- Development should, where possible, minimise impervious ground surfaces and direct runoff to landscaped areas, soakage trenches, or possible aquifer recharge.

## 7.12 Formalisation of Drainage Easements

Field inspection identified numerous locations throughout the catchment where stormwater runoff is directed through private property via informal drainage easements (eg. Lane St, Buck St, Thomas Ln etc).

It is generally considered undesirable for Council to utilise private property for stormwater management without establishing a formal easement. This is largely due to Council's legal responsibilities, and issues related to a possible lack of cooperation from the property owner with respect to site access and drainage upgrades. Hence a course of action has been formulated to facilitate in the formalisation of drainage easements throughout the catchment, as outlined below:

- Preparation of a drainage easement inventory for the catchment, including the location and other data specific to each easement;
- Development of a ranking system to prioritise the drainage easements to be formalised (taking into consideration contributing catchment size and the relative importance of the existing site specific flooding issue);
- Notification of property owners whose land includes an unofficial easement;
- Undertake a process of public consultation regarding drainage easement issues;
- Negotiate with landowners for voluntary formalisation of easements where possible;
- Seek possible sources of funding to support the strategic acquisition of land for easements where necessary;
- Set target quotas for the number of easements to be formalised within a fixed timeframe;
- Commit to a long term plan to formalise all easements within the catchment by incorporating the associated costs into the annual Council budget.

## 7.13 Asset Management Plan

Asset management is an approach to develop and maintain infrastructure to ensure that:

- Asset requirements and asset management strategies are driven by defined service levels and performance standards;
- Scarce financial resources are properly allocated and managed to optimise investment in infrastructure;
- A long term lifecycle approach is taken when determining asset operations, maintenance, renewal and development strategies.

The first stage in the development of an asset management plan involves the collection of data relating to the standard attributes of each type of infrastructure. This data is then compiled into a database that can used to determine valuations of existing assets, and the prioritisation of maintenance and upgrade/replacement works to meet the target performance standard.

Asset management planning is a continuous process whereby data is regularly updated, and performance objectives are measured over both short and long term timeframes.

### 7.13.1 Existing Drainage Asset Inventory

Council are in the process of establishing an inventory of information relating to the existing stormwater drainage infrastructure. The inventory is formatted in an Excel spreadsheet and includes the following attributes; size, length, depth, slope, material, age, existing condition, and life expectancy, as well as photographs and CCTV footage.

Spatial coordinates are also an important component of the inventory, as they facilitate the transition of the information into Council's GIS database. Should the exact location of drainage assets not be known, this information should be recorded using a GPS unit. This also enables invert and adjacent watertable levels to be recorded.

It is understood that to date, a majority of the assets recorded in the inventory are underground drains (ie. culverts and pipes). The scope of the inventory should be extended to include important information relating to open channels, major overland flow paths, creeks, and easements (as discussed in detail in Section 7.12).

It is considered that this database essentially forms the first stage in the development of a drainage asset management plan.

### 7.13.2 Council Asset Management System

We recommend that Council continue by compiling data for other assets, such as; roads, footpaths, kerb and gutter, bridges, signs, traffic devices, buildings, parks etc. Data can be compiled by exploring the existing GIS database, digitising hardcopy plans, and field collection. It is important that the structure and format of these datasets be defined in a logical and user friendly manner, and that information on all relevant asset attributes be recorded.

As a MapInfo strategic partner we have compiled a comprehensive range of ‘asset attribute sets’ suitable for compilation by local government agencies. We would be happy to assist Council in refining the nature of the data required for each asset.

Once the data has been assimilated it is critical that a process of valuation be implemented in order for Council to comply with AAS 27. Current stormwater asset data capture methods are considered to provide a basic level of compliance, however there are clear advantages in storing the data in a format that:

- Can be maintained relatively easily (ie. even by new staff);
- Can be incorporated into a wider system;
- Is convenient to update and provide cost summaries.

Due to the high volumes of data associated with asset management, the use of specialised computer software is considered necessary if the information is to be used to its maximum benefit.

There are many software packages commercially available for this purpose, such as Conquest, which is generally compatible with Council’s existing GIS database (or indeed can be tailored to suit). These software packages have the capability to provide asset valuations and prioritise maintenance and upgrade/replacement works within user defined constraints.

We are able to assist Council in the implementation of such software, the use of which will assist Council in better understanding the extent and condition of their asset base, and guide future decision making and funding opportunities.

## 7.14 Pollutant Control Measures

It is widely accepted that a large proportion of pollution in stormwater runoff originates from commercial and industrial areas, and arterial roads. The main types of pollutants typical in stormwater discharge include:

- rubbish;
- suspended solids;
- oil and grease;
- nutrients (eg. ammonia, nitrate, phosphorus);

- heavy metals (eg. copper, lead, zinc).

Pollutant control measures are designed to minimise the transfer of pollutants to receiving waters. It is considered that a 'treatment-train' approach is most effective in improving the quality of stormwater discharge. This involves adopting a series of water quality improvement measures aimed to provide continuing treatment to stormwater as it flows through the catchment. When implementing the 'treatment-train' approach it is considered best practice to install pollution control devices (primary treatment) upstream of wetlands and grassed swales (secondary treatment).

This arrangement is recommended to be adopted throughout the Broken Hill catchments. It is proposed to install pollutant control devices (such as Gross Pollutant Traps and Trash Racks) upstream of the detention basins. All proposed detention basins are on-line, in that they form part of the designated overland flow paths through the catchments. Grassed swales are to be constructed through the basins, in order to provide filtration and water treatment during low flow events, which comprise a majority of rainfall events.

### 7.14.1 Pollution Control Devices

The size and cost of a pollutant control device increases with the size of the catchment area that it services, as larger catchments generate higher peak flows. It is usually considered cost prohibitive to install a single pollutant control device to service a large catchment, and thus the installation of several smaller devices throughout the catchment is preferable. Increasing the number of pollutant control devices within the catchment also provides a higher level of redundancy, whereby improvements to water quality are still made even if one or more units are not operating correctly at any given time.

There are currently three CDS Gross Pollutant Traps operating within the downstream extents of the Mulga Creek catchment; Argent St, Wolfram St, and Lane St.

The principle considerations when selecting a stormwater pollutant control device are functionality and whole of life cost. Functionality relates to the types of contaminants targeted by the device, the rate of pollutant capture, the design life, and the performance of the device under various flow regimes and site conditions. The whole of life cost takes into account the capital cost, as well as the ongoing monitoring, cleaning, maintenance and operational costs associated with the device.

It is considered that any of the devices listed below would be suitable for use in the Broken Hill catchment:

- CDS GPT;
- Rocla CleansAll GPT;

- Humes Humeceptor;
- Trash Rack Structures.

Suitable locations for the installation of such devices include:

- *The Living Desert Catchment* - the proposed McCulloch St System upstream of the detention basin;
- *Cemetery Creek Catchment* - the downstream end of the proposed Rakow St and Thomas St Systems (ie. prior to discharge to Cemetery Creek);
- *Railwaytown Catchment* - the proposed Mercury St System upstream of the detention basin;
- *South Broken Hill Catchment* - the proposed Patton St System upstream of the detention basin.

### 7.14.2 Siltation of Underground Drainage

Council staff have indicated that the deposition and accumulation of silt within existing underground drainage infrastructure incurs significant clean-up costs and hinders drain functionality. It is not economically feasible to construct silt traps or similar devices on all proposed drainage systems. In order for the proposed drainage upgrades to be 'self-cleaning' a minimum desirable drain grade of 1% is to be adopted, where physically possible.

# 8. Aquifer Storage & Recovery Investigation

The objective of this investigation was to assess the potential for groundwater to be used as a resource in the area, either independently or conjunctively with Aquifer Storage and Recovery (ASR) techniques. This section of the report summarises in detail the hydrogeology of the area and the prospect of groundwater in the immediate vicinity of Broken Hill.

The following briefing on geology and hydrogeology of the area was based on the most recent information interpreted from geological maps, hydrogeological data, previous reports and data from existing wells in the area. Regions and/or major features that exhibit the greatest potential have been highlighted for future in depth investigation. A full report is included in Appendix B.

## 8.1 Geology

The geology of the Broken Hill area includes; Tertiary and Quaternary sediments (undifferentiated Cainozoic deposits), and prograde metamorphic rocks. In the Broken Hill township the Quaternary and Tertiary sediments form a thin veneer overlying the metamorphic rock. They consist mainly of sand, gravel, clay and silt and form a minor aquifer except along the main drainage where the alluvial sediments become saturated with the occasional runoff from the creeks and form an aquifer with limited potential. Generally it is difficult to separate the Quaternary from the Tertiary sediments in the township area.

The prograde metamorphic rocks consist of basic granulite and amphibolite, metasediment, quartzo-felspathic gneiss, composite gneiss and mimetite, leucocratic K-feldspar rich and plagioclase rich quartzo-felspathic rocks, and calc-silicate rocks.

Dykes of granite, pegmatite, dolerite, and hornblendite intruded after the prograde metamorphic even and were altered by retrograde metamorphism. The area is traversed by several major retrograde schist zones and an abundance of minor schist zones.

The aquifers within the hard rock can be classified as compartment, strip, or fracture aquifers.

Major folds in the Broken Hill sheet area are first and second generation structures. Folds contain high fracture zones, particularly along their axis, and are an important feature in groundwater storage and movement.

The stratigraphy of the Broken Hill sheet area comprises metamorphic rocks, is host for the Broken Hill Main Lode, and generally contains various amounts of Broken Hill type "lode" rocks (eg. quartz-gahnite).

Structural analysis in the Broken Hill mines area recognised two main styles of deformation; folds with axial plane schistosity and a lineation parallel to their axes, and folds with the same schistosity and which themselves lack axial plane schistosity.

The majority of lineaments are orientated in sets trending north-north-west to north-west or north-north-east. The trend of these lineament sets corresponds with the trend of sets of retrograde schist of schist zones.

The Retrograde schist zones are localised and can be referred to as 'crush zones', 'shear zones' or 'faults'. Some degree of transcurrent displacement is evident in some schist zones. This includes the Globe-Vauxhal, Stephens Creek, and limestone schist zones trending north-easterly to easterly.

## 8.2 Hydrogeology

Previous work by C.M. Jewell & Associates (2004) and Corkery (2005), and the NSW Department of Infrastructure, Planning and Natural Resources (DIPNR) database shows relatively little use of groundwater in the Broken Hill area. There are no known municipal or potable domestic abstractions, however a number of boreholes, wells and mine shafts provide useful groundwater information for stock and non-potable domestic use.

The total depth of the wells varies from 10 to 131 metres, with yield varying between 0.1 and 10 L/s. Generally yield averages 1.5 L/s which is considered very low for the intended purpose. Observations made during mining operations have also provided a reasonable indication of groundwater occurrence in the Willyama Supergroup rocks.

### 8.2.1 Willyama Supergroup Rocks

These rocks transmit water only through secondary features such as fractures and joints. The main lode horizons tend to be significantly more fractured than the surrounding rocks and therefore function as low-permeability compartment aquifers.

A few water boreholes are known to have been constructed into the Willyama Supergroup rocks, and these generally provide low to moderate yields of water. Yields range between 20 to 500 $m^3$/day (0.2 to 6 L/s).

Experience over many years of mining in the Broken Hill area confirms these findings. Most mines have made some water, with locally higher and sometimes usable inflows. The largest groundwater inflows have occurred in the southern mining leases.

Similarly, the rock quarry to the south-west of Broken Hill experiences low volume groundwater seepage inflows from a depth of 15 metres along the southern and south-western faces. Recharge to the Willyama rocks may occur by direct infiltration of rainfall, but the volume is low.

Reports from mining operations revealed that the lode horizon in the southern leases has a well developed fracture system oriented parallel to the main rock contacts (striking north-south and dipping to the west at about 70 degrees). Reports suggest the presence of a brecciate hanging-wall aquifer between the lode horizon and the granite-gneissic country rock.

Aquifer transmissivity ranges between 20 to 80 $m^2$/day, which is considerably low. There is no documentation of draw down, the type of the aquifer, and whether or not recovery tests were carried out. Therefore this information must be treated with great caution. It has been assumed that the aquifer is unconfined, and a value of specific yield can be assumed to be in the range 0.001 to 0.01.

Results of a pumping test carried out at a Rising Sun shaft (C.M. Jewell & Associates, 2004) have indicated that with pumping rates of 520 $m^3$/day (6 L/sec) the draw down was about 20 metres, while the White Leads shaft yielded less than 100 $m^3$/day for three months with about 57 metres draw down. Exploration boreholes in the southern leases have yielded up to 500 $m^3$/day. Groundwater is pumped from the former Daydream mine, north of Broken Hill, for non-potable domestic and garden irrigation use.

Borehole records from the Willyama rocks show that water quality is generally ‘good’ to brackish and mostly suitable for irrigation and stock, with a calcium-magnesium sulphate content that is relatively high.

Groundwater sampling carried out from exploration boreholes in the southern leases indicated total dissolved solids generally in the range of 8000 to 10,000 mg/L, with sulphate typically 1500 to 2800 mg/L. A sample of water obtained from the quarry sump was sampled and showed salinity of 7000 mg/L, with sulphate of 1160 mg/L and chloride of 2310 mg/L.

### 8.2.2 Quaternary Aquifers

Types of quaternary aquifers present in the City of Broken Hill and immediate surroundings include:

- Colluvial sediments
    - contain thin aquifer
    - uneconomic for either extraction or injection

- Alluvial sediments
  - located primarily along water courses with the coarser material confined to areas in the immediate surrounding of creeks
  - water quality is generally good

- Sand dunes aquifer
  - this type of aquifer is scarce in the Broken Hill area but is located in the surrounding region
  - contains groundwater at its base
  - salinity levels unknown
  - yield unknown but is expected to be negligible for the intended purpose

Overall, information on the Quaternary aquifers in close proximity to the Broken Hill township is scarce. However, available information suggests that they are thin and unreliable for the intended purpose.

### 8.2.3 Quaternary & Tertiary Aquifers in the Darling Floodplain and Lake Menindee Area

These aquifers are located at a distance of approximately 90 to 100 kilometres south-west of Broken Hill, but were included in this study because of their significance in implementing ASR related to the fact that the Broken Hill water supply is partly supplemented from Lake Menindee. The lake is located within the Murray-Darling Basin.

The regional hydrogeology in the Darling Floodplain and Menindee Lake area can be summarised as follows;

*Alluvial Aquifer (includes Blanchetown Clay)*
Generally unconsolidated fine to medium sands interbedded with clays, and silty and sandy in parts. Recharged from rivet and rainfall. The supply from this aquifer is considered negligible for the intended purpose.

*Pliocene and Aquifer Parilla Sand*
Partly cemented shelly limestone with sand matrix and unconsolidated fine to coarse sand. This aquifer generally has high salinity for the intended purpose, however groundwater salinity of 1000 mg/L or less is recorded around the Darling river area suggesting a direct recharge from the River.

*Murray Group Limestone Aquifer*
Consolidated, highly fossiliferous fine to coarse limestone.

*Lower Renmark Group Aquifer*
Unconsolidated carbonaceous sands, silt and clay.

Tertiary aquifers are considered reliable for performing ASR. The aquifer can be used for purifying the injected water and extracting for the intended purpose.

## 8.3 Potential for ASR in the Broken Hill Area

Several lineaments, folds and schist zones intersect the Broken Hill area. The location of potential sites to obtain groundwater and implement ASR correspond mainly to major structures in the Willyama Supergroup (refer Figure 8.1) and can be summarized below:

- South Base Monocline;
- Globe Vauxhall City Schist Zone;
- Hanging Wall Synform;
- Broken Hill Synform;
- Broken Hill Antiform.

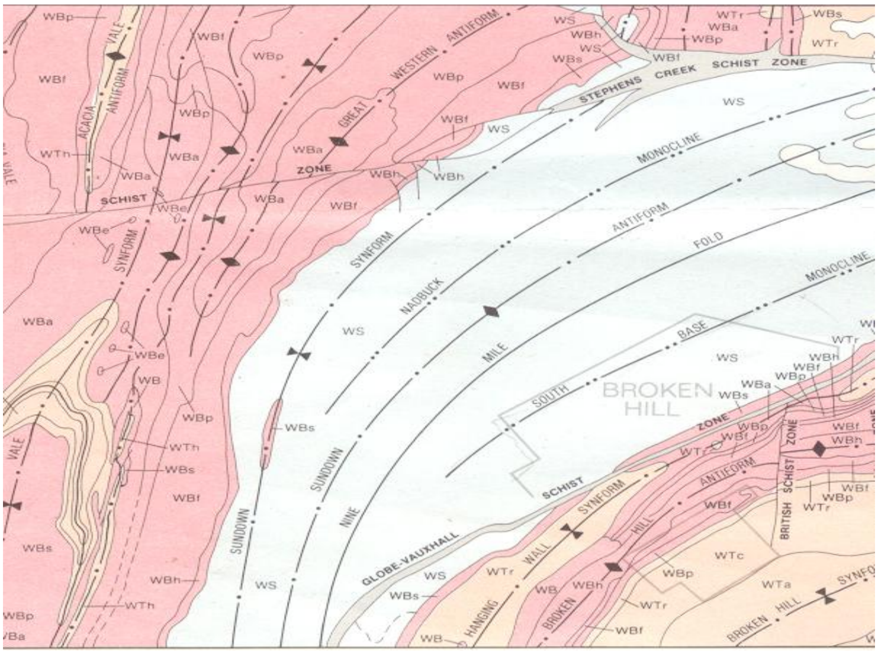

**Figure 8.1 Major Geological Structures in the Willyama Supergroup**

The South Base Monocline is presented on the geological map of Broken Hill, but unfortunately it has not been described in detail in any geological references. However, this monocline (F3) is open upright folds of lithological layering and first and second degree deformation structures. The axial plans strike north to north-east and dip subvertically. This structure may contain major fractures which make it a potential target aquifer.

The Globe-Vauxhall City Schist Zone attains a maximum width of 100 metres and dips to the south-east at 60 to 70 degrees. This schist zone crops out very poorly in the Broken Hill city area and to the west of the township, but is well exposed to the east of Broken Hill. This structural feature contains fractures and if the fractures intersect the water table, it may represent a potential source of groundwater supply.

The Hanging Wall Synform is a relatively tight, overturned, south-west plunging second generation ($F_2$) fold whose axial plane dips steeply north-west. This synform is well defined by lithological symmetry.

The Broken Hill Synform is an open, overturned, south plunging second generation ($F_2$) anticline whose axial plane dips north-west and passes through the core of outcrop of the "Alma" augen gneiss.

The Broken Hill Antiform is inferred to exist between the Broken Hill Synform and the Hanging wall Synform. This antiform is a tight inverted syncline that dips steeply to the north-west and plunges to the south-west.

## 8.4 Potential for ASR in the Surrounding Region

The existing infrastructure for pumping surface water from Menindee to Broken Hill represents a good opportunity to implement ASR techniques. The presence of a thick Tertiary aquifer near Lake Menindee represents a good opportunity for injecting surplus winter water into one well, to be extracted later from another well. This concept relies in essence on a suitable aquifer to accept and filter untreated River Murray water to a level suitable for domestic and potable purposes. This technology has been successfully applied at the township of Clayton to improve the quality of River Murray water supplied for domestic and potable purposes in towns located in the lower River Murray.

## 8.5 Proposed Field Investigations

It is recommended to drill exploration wells to total depth of 150 metres using a small hummer drilling technique, and if the exploration well is successful then it can be converted into an injection/extraction well. The proposed location of the investigation well is in the vicinity of the proposed Mulga Creek Catchment Wetland in the City of Broken Hill (refer Section 8.6 for more detail), which is located in close proximity to the Globe Vauxhall City Schist Zone and south of the South Base Monocline. Field investigations are also proposed for a suitable location in close proximity to Lake Menindee.

A careful study of local geology and site mapping by a hydrogeologist is required prior to drilling. Drilling supervision by a qualified hydrogeologist is important for the success of the project. Pump testing must be conducted according to specification from a hydrogeologist.

Indicative costing for drilling an exploration well in the fracture rock aquifer to a total depth of 150 metres is approximately $40,000 to $50,000, excluding water courtage and hydrogeological supervision, and also providing that overburden does not exceed 10 metres. This costing is serving as an indication only, and the actual costs could be considerably higher depending on the availability of drilling rigs and operators.

The pump testing consists of three stages; injections of 100 minute duration, long term draw down tests and long term recovery tests. An indicative cost to carry out this exercise is approximately $14,000. The long term recovery test is important in the fracture rock environment as it will indicate the type of aquifer (either strip or compartment aquifer) and give some indication of the boundary condition of the aquifer. Drilling permits are required to carry out such works in NSW, and an EPA licence may be required for injection.

If drilling and pump testing prove to be successful a conceptual scheme design is required which identifies the following parameters:

- Type of aquifer;
- Boundary condition;
- Injection rates and potential, and this includes impact of clogging and treatment;
- Recovery efficiency of injected water;
- EPA Licence if required.

The second stage of the conceptual design process requires the following:

- Injection/extraction well design (includes number of wells, if applicable);
- Injection pumps and delivery pipeline;
- Process control;

Monitoring requirements.

## 8.6 Other Stormwater Harvesting & Reuse Opportunities

Stormwater harvesting and reuse systems can vary from small scale storage tanks and cells (either above or below ground), to large scale wetlands and retention basins.

While storage tanks and cells are likely to prove beneficial in supplementing the water demands of residential, commercial and industrial properties, they are unlikely to be feasible in catering for higher demands such as those associated with irrigation. For such demands to be met a much larger storage facility, capable of capturing and storing large volumes of stormwater from the intense and sporadic storm events characteristic of local rainfall, need to be employed (eg. wetlands, dams).

It should be noted that wetlands do not confer any significant flow mitigation benefits, owing to the fact that the long-term storage of water prevents the use of the storage volume for detention and controlled discharge.

### 8.6.1 Identification of Potential Sites

Council staff have indicated that stormwater harvesting and reuse techniques have been adopted with some success by the nursery adjacent to Cemetery Creek. A field inspection was undertaken to identify other locations throughout the catchment that are amenable to the implementation of stormwater harvesting and reuse practices. The following criteria were adopted to determine the stormwater reuse potential of each site:

- Size of upstream catchment and likely stormwater yield;
- Demand for harvested water;
- Proximity to irrigated land (eg. Council reserves, sports ground);
- Site ownership (ie. Council land or potential for land acquisition);
- Geographical constraints (site levels as they relate to the drainage network).

Ideally a stormwater harvesting site should be located on Council owned land that is in close proximity to the demand site (to limit costs and technical difficulties), yield sufficient water to satisfy a majority of the demand, maintain continuity of supply (even during warmer, drier months), and not impede the functionality of the adjacent drainage network.

Many potential sites throughout the catchment had already been designated for use as detention basins. Although it is possible for a basin to provide both detention and retention storage (ie. the outfall level can be set at a given height above the invert level of the basin), the physical constraints limited the amount of storage available in many instances and thus the entire volume was required for flow mitigation purposes.

### 8.6.2 Mulga Creek Catchment Wetland

The vacant Council land north of the intersection of Bathurst St and Brazil St demonstrated the desirable characteristics outlined above, and warranted further investigation. The site is located at the downstream end of the large Mulga Creek catchment (approx 400 ha), adjacent to the main open earth channel discharging to Mulga Creek, and is in close proximity to the Beryl St Soccer Ovals and Jubilee Oval (refer Figure 8.2).

Based on contours and limited survey of existing ground levels a concept design of the proposed wetland was carried out. It was proposed to establish minor bunding just downstream of the junction of the two open earth channels, to direct all flows from these channels into the wetland.

The maximum storage level in the wetland was set at 288.50 mAHD to match the natural channel invert at the northern (downstream) end, thus allowing flows to continue in a north-easterly direction toward Mulga Creek once the capacity of the wetland is reached. The invert level of the wetland was set at 286.00 mAHD. Assuming batter slopes of 1:6 and a storage height of 2.5 metres, it was estimated that 10,000 $m^3$ of storage volume is available at the site.

Below is the approximate cost of establishing the proposed wetland, and associated pump stations, rising mains and irrigation infrastructure to serve both the Jubilee Oval and Beryl St Soccer Ovals. This cost could be reduced substantially if the existing irrigation infrastructure at these sites is compatible with the proposed pump and rising main system. Such details would need to be refined at the detailed design stage.

| | |
|---|---|
| **Wetland, Pump Stations, Rising Mains, Irrigation:** | $1,200,000 |

## Figure 8.2 Mulga Creek Catchment Wetland

### 8.6.3 Water Balance Modelling

A water balance model was used to simulate the water level in the wetland and estimate the potential water yield from the catchment over the period of available rainfall records.

The water balance model requires inputs relating to the catchment area, basin height-storage characteristics, catchment specific rainfall and evaporation, seepage, and inputs from adjacent storages. It also takes into account the area requiring irrigation (assuming a grassed area with low demand during winter and peak demand during summer), which in this instance was the Beryl St Soccer Ovals (6.8 ha) and Jubilee Oval (3.7 ha).

The model produces a time-series record of water levels and spill volumes from each of the ponds.

#### 8.6.3.1 Rainfall Data

Daily rainfall data (mm/day) was obtained from the Bureau of Meteorology for a gauge at Broken Hill, for the period 1889-2004.

#### 8.6.3.2 Results

The preliminary results indicate that over the rainfall record approximately 55% of the irrigation demand was met when both sites were being irrigated. Roughly 85% of the irrigation demand was met when only the Jubilee Oval was irrigated, and roughly 70% of the irrigation demand was met when only the Beryl St Soccer Ovals were irrigated.

A greater proportion of the irrigation demand could be met by increasing the storage volume of the wetland and/or decreasing the area to be irrigated.

Graphical representations of the water level in the proposed wetland (when only the Jubilee Oval is irrigated) have been generated for individual years of the rainfall record characterised as being ‘wet’, ‘average’, and ‘dry’.

Figure 8.3 shows the water level in the proposed wetland throughout 1993, a ‘wet’ year during which 435 mm of rainfall was recorded; 1998, an ‘average’ year during which 263 mm of rainfall was recorded; and 2002, a ‘dry’ year during which 81 mm of rainfall was recorded. These years were generally preceded by years with similar rainfall conditions (ie. ‘wet’ was preceded by ‘wet’, ‘average’ was preceded by ‘average’, ‘dry’ was preceded by ‘dry’).

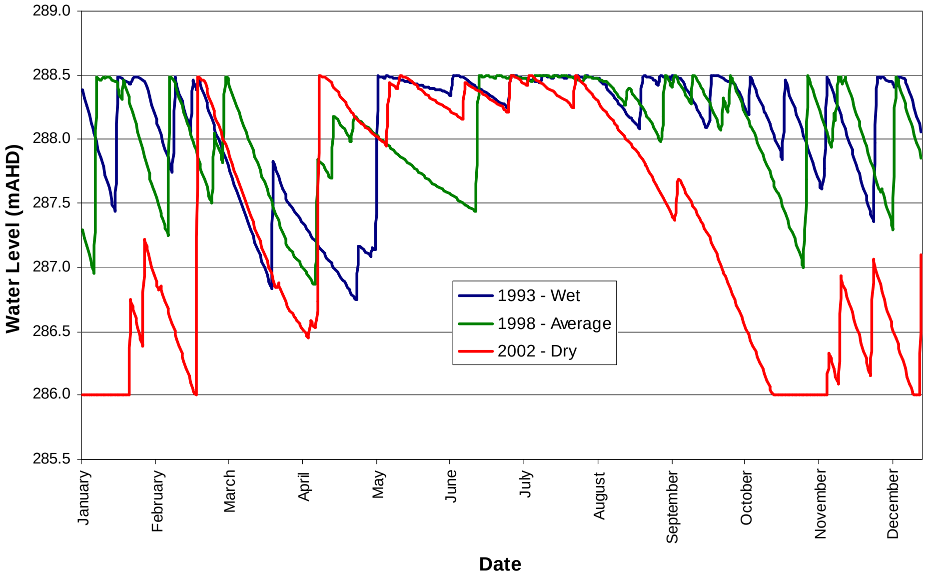

**Figure 8.3 Water Level in the Proposed Wetland for Different Rainfall Conditions (irrigation of Jubilee Oval only)**

It is not uncommon for the proposed wetland to empty completely about once a year over the summer months, particularly during ‘dry’ and ‘average’ years. However the modelling indicates that the proposed wetland is generally able to maintain security of supply for the remainder of the year, as evidenced above.

Based on these promising preliminary results it is recommended that a detailed concept be developed for this site. Detailed engineering survey of the site will need to be undertaken, to accurately determine the storage volume available. The water balance model would also need to be refined, in order to confirm that the stormwater yield is able to meet irrigation demands.

# 9. Summary

Hydrological modelling has been undertaken for The Living Desert, Mulga Creek, Cemetery Creek, Railwaytown, and South Broken Hill catchments within the urban extents of Broken Hill. Generally a relatively low standard of protection is provided by the existing overland flow paths and drainage infrastructure throughout the above catchments.

Council have identified several locations throughout the catchment known to be flood prone, and the modelling carried out in this study highlighted deficiencies in the existing drainage system at these locations. The results of the analysis of the existing drainage system were also vindicated by anecdotal and photographic evidence of historical flood events. Field inspection also identified several locations where existing stormwater management practices have serious implications on traffic and road safety.

The majority of upgrade works devised in this study have been designed to mitigate the magnitude of overland flows through the construction of detention basins on areas of Council greenspace, and constructing underground drainage where appropriate. The benefits of these drainage upgrades include increased flood protection for adjacent and downstream properties, and improved trafficability of roads during storm events.

The application of Aquifer Storage and Recovery (ASR) techniques in Broken Hill, and the harvesting and reuse of stormwater within the Mulga Creek catchment, have been investigated. The feasibility of water quality improvement measures such as wetlands, grassed swales and Gross Pollutant Traps (GPTs) have also been scoped. Further design work will be required to develop the concepts to the implementation stage.

The required works have been ranked according to their relative priority (low or high). Budget constraints and other factors are expected to dictate the order in which high priority works proceed. Indicative costs have also been provided for general information, and assessment by an appropriately qualified quantity surveyor is recommended should more detailed cost estimates be required. The total cost of the high priority upgrades was estimated to be $5,550,000, and the total cost of the low priority upgrades was estimated to be $8,450,000.

The large number of works recommended in this study bestow upon Council significant design and construction costs. This not only necessitates a staged approach to the upgrade process, but also the sourcing of available funding from

State and Federal Government initiatives.  It is expected that several of the proposed upgrade schemes will prove eligible for subsidy, and as such a guide to funding availability has been formulated.

A ‘generic’ set of principles of development control has been formulated for inclusion in Council’s Development Plan, for use in ongoing assessment of development applications.  Overall the master plan itself provides a summary of the strategic issues, opportunities and objectives for the management of stormwater in each catchment.

# 10. References

Corkery, R.W. & Co. Pty Ltd in LEMP (2005) “Broken Hill Landfill Site”, Report No.438/02.

Institution of Engineers Australia (1987) “Australian Rainfall & Runoff”.

Jewell, C.M. & Associates Pty Ltd (2004) “Broken Hill Groundwater Resources Study for John Wilson and Partners Pty Ltd”, Report No. J0958.1R-rev2.

O’Loughlin, G. (1993) “The ILSAX Program for Urban Stormwater Drainage Design and Analysis”, University of Technology, Sydney.

# Appendix A

# Proposed Drain Upgrades Summary

# Appendix B

# Aquifer Storage & Recovery Investigation Full Report