PROJECT 6
REPORT 12/1979

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INTRODUCTION

This report examines the overall hydrological behaviour, including sedimentation of the proposed recreational dam on the Todd River at the Old Telegraph Station. Part A examines dam behaviour, flow reductions downstream of the dam and associated flood attenuation in Alice Springs. Sediment inflow into the dam and downstream scour effects are examined in Part B.

The report was prepared under the supervision of:

Mr. Andy Macqueen, Senior Engineer, Water Investigations, Alice Springs.
& Project Leader

Mr. Ralph Ash, ex Senior Engineer, Hydrographic Section, Darwin

Mr. Ron Reinhard, Senior Engineer, Analysis and Computation, Darwin

All Australian Height Datum (AHD) levels in this hydrology report are based on topographic maps produced from aerial photography done in September, 1964.
SUMMARY AND CONCLUSION

The usefulness of the impoundment for recreation may be judged by area availability. A sailing triangle of total length 2,400 metres would be available for at least 95 percent of the time. This corresponds to a surface area of approximately 75 hectares. This area availability can be expected with the primary spillway level at 594 metres A.H.D. An annual draft of one million kilolitres could be taken from the storage for watering of parks and recreation grounds without significantly affecting the availability of water for recreation. On average, the dam would be filled to spill level about three times every year.

It is felt that this area availability would satisfactorily meet the proposed recreational uses, such as small craft sailing, canoeing, fishing and swimming.

Rapid rises of water levels during floods - as much as 2.5 metres in four hours for the 20-year return period flood - will necessitate careful location of amenities. During the recession limb of flood wave, the water level will drop quickly to the spillway level.

Streamflow through Alice Springs would be reduced. If the dam is constructed it is expected that on the long-term average streamflow volumes will be reduced by 20 percent. On a time basis, flow is reduced by approximately 30 percent. About 40 percent of individual stream flows would be fully retained in the impoundment. These figures refer to flows entering the town in the Todd River only and do not allow for continued inputs from the Charles River and town drains.

Flooding in Alice Springs would be reduced. The level of the expected 100-year return period flood would not exceed the level of the present 20-year return period flood. The 50-year return flood would be contained within the banks of the Todd River, except for areas along Leichhardt Terrace and South Terrace.

A conservative estimate of dam life before complete silting is 120 years. To achieve this life it will be necessary to incorporate a sediment trap immediately upstream of the impoundment. This will trap bed load at an estimated rate of 16,000 tonnes per year. By comparison, demand for construction sand in the Alice Springs region is currently averaging approximately 53,000 tonnes per year, and hence in most years it should be feasible to completely mine the trap.
Scour downstream of the dam will have a negligible effect on the stream reach through Alice Springs. Scour may occur near the Alice Spring at the Old Telegraph Station during floods of return periods larger than about six years. If scour does occur, its effect could easily be remedied by suitable rehabilitation of the reach downstream of the dam. Most scour will occur between the primary spillway and the Schwartz Crescent causeway.
ADDENDUM

The results in this report are based on storage/area/elevation curves derived from 1965 photogrammetric contour maps. Recently available (October 1979) photogrammetric maps show slight discrepancies when compared to the 1965 maps. Subject to the verification of the latest maps this report may slightly overestimate flood attenuation and underestimate spill surcharge levels. It is recommended that results of this report be rechecked before any final design is commenced.
Plan of Dam Site

Figure A

Key:
- Level Exceeded 95% of Time
- Top Water Level (590 m)
- Possible Sailing Triangles

Dimensions: 842.0 x 1190.0
PART A

RESERVOIR BEHAVIOUR

and

FLOOD ROUTING
1. INTRODUCTION

Part A discusses in detail the processing of all relevant stream and rainfall data, the synthesis of a historic flow record for the damsite and the generation of long-term flow series by a rainfall/runoff correlation and by a statistical means. The storage behaviour of the dam at various spillway levels is discussed. In addition, the derivation of design floods for various recurrence intervals and the routing of these through various spillway levels and widths is included. The effect of the dam on flooding in Alice Springs is also discussed.

The simulation of dam behaviour was performed using both a Dam Simulation method and Gould's Probability Matrix method. Three types of data were used in the former model. These were the historic flows (18 years), synthetic flows based on a correlation with the historic rainfall data (105 years) and statistically generated flows (105 years).
2. HYDROGRAPHIC DATA

2.1 General

This section summarizes and discusses the rainfall, streamflow, evaporation and impoundment data relevant to the reservoir behaviour studies. A more complete summary of the rainfall and streamflow records in the Alice Springs Region has been prepared by Hug (1979).

2.2 Rainfall

The recording of rainfall records commenced with a European settlement at the Alice Springs Telegraph Station in 1873. The first full water year of records - September to August - was 1873/74. In 1931, the recording site transferred to the site of the original town Post Office and is at present at the Post Office site in Hartley St. Monthly records are available from 1873/74 to the present - 103 years in all.

In addition, there are several daily read raingauges in the Todd River Catchment - see Hug (1979). There are also pluviograph records at Bond Springs in the Todd River Catchment, at the Water Investigations Office in Alice Springs immediately below G.S. 006009 Catchment and at Mt. Lloyd in the Charles River Catchment (Figure 1).

Based on the 105 years of rainfall records, the mean rainfall per water year is 281mm with a standard deviation of 155mm and a lag one serial correlation of 0.14. Ride and Cutler (1967) and Macqueen, Tongia and Verhoeven (1976) have both analysed the rainfall records available at the times of their reports. Attention is drawn to several points. Table 1 shows the water year totals since streamflow records commenced at Gauging Station GS006009, Todd River at Wills Terrace. 1964/65 is the second driest water year on record. The driest year, 1901/02, which had only 31mm was preceded by a year of 245mm and followed by a year of 356mm.
LOCATION OF RECORDING STATIONS AND DAMSITE

FIGURE 1
Table 1

ANNUAL RAINFALLS: WATER YEARS 1952/53 TO 1977/78 AT ALICE SPRINGS POST OFFICE.

<table>
<thead>
<tr>
<th>Water Year</th>
<th>Annual Rainfall (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1952/53</td>
<td>353</td>
</tr>
<tr>
<td>53/54</td>
<td>267</td>
</tr>
<tr>
<td>54/55</td>
<td>360</td>
</tr>
<tr>
<td>55/56</td>
<td>310</td>
</tr>
<tr>
<td>56/57</td>
<td>215</td>
</tr>
<tr>
<td>57/58</td>
<td>269</td>
</tr>
<tr>
<td>58/59</td>
<td>158</td>
</tr>
<tr>
<td>59/60</td>
<td>161</td>
</tr>
<tr>
<td>60/61</td>
<td>173</td>
</tr>
<tr>
<td>61/62</td>
<td>189</td>
</tr>
<tr>
<td>62/63</td>
<td>153</td>
</tr>
<tr>
<td>63/64</td>
<td>99</td>
</tr>
<tr>
<td>64/65</td>
<td>95</td>
</tr>
<tr>
<td>65/66</td>
<td>331</td>
</tr>
<tr>
<td>66/67</td>
<td>271</td>
</tr>
<tr>
<td>67/68</td>
<td>370</td>
</tr>
<tr>
<td>68/69</td>
<td>317</td>
</tr>
<tr>
<td>69/70</td>
<td>106</td>
</tr>
<tr>
<td>70/71</td>
<td>194</td>
</tr>
<tr>
<td>71/72</td>
<td>313</td>
</tr>
<tr>
<td>72/73</td>
<td>306</td>
</tr>
<tr>
<td>73/74</td>
<td>326</td>
</tr>
<tr>
<td>74/75</td>
<td>509</td>
</tr>
<tr>
<td>75/76</td>
<td>849</td>
</tr>
<tr>
<td>76/77</td>
<td>357</td>
</tr>
<tr>
<td>77/78</td>
<td>457</td>
</tr>
</tbody>
</table>
However the 1964/65 year was preceded by 8 years of lower than average rainfalls. The wettest year on record is 1973/74 with 926mm. The five year period from 1973/74 is the wettest period on record, the rainfall in each of these years being greater than the mean plus one standard deviation.

2.3 Streamflows

2.3.1 General

The location of the four current gauging stations closest to the damsite are shown in Figure 1. Table 2 shows the catchment areas of the stations and of the damsite. An additional gauging station operated at the damsite from 1967 to 1972, but was abandoned due to excessive silting of the recorder well. There are insufficient records from the station for any form of station analysis.

Table 2

<table>
<thead>
<tr>
<th>Station No.</th>
<th>Stream</th>
<th>Station Name</th>
<th>Catchment Area (km²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>G.S. 006046</td>
<td>Todd River</td>
<td>Wigley Gorge</td>
<td>360</td>
</tr>
<tr>
<td>G.S. 006009</td>
<td>Todd River</td>
<td>Wills Terrace</td>
<td>452</td>
</tr>
<tr>
<td>G.S. 006126</td>
<td>Todd River</td>
<td>Heavitree Gap</td>
<td>508</td>
</tr>
<tr>
<td>G.S. 006047</td>
<td>Charles River</td>
<td>Big Dipper</td>
<td>42</td>
</tr>
<tr>
<td>G.S. 006010</td>
<td>Todd River</td>
<td>Damsite</td>
<td>400</td>
</tr>
</tbody>
</table>

2.3.2 G.S. 006046

Records commence in July 1958 and continue to the present. For this period there are 42 months of incomplete records. There have been 86 stream gaugings done. The largest discharge gauged was 27.9 m³/s on 10/5/68 at a gauge height of 1.646m. On 15/3/77 the highest gauge height of 4.8 metres was recorded. Cease to flow gauge height is 0.301 metres. Control is a concrete weir.
2.3.3 G.S. 006009.

This station commenced operation at Wills Terrace causeway in February 1953. In April 1970 it was moved 200 metres upstream to the present site, retaining the same gauge datum. During the period of operation there have been 175 stream gaugings done. Before April 1970 the cease to flow gauge height was 1.838 metres. The largest discharge gauged was 233 m$^3$/s on 5/2/67 at a gauge height of 2.170 metres. After April 1970 the largest discharge gauged was 450 m$^3$/s on 5/3/72 at gauge height of 3.115 metres. The cease to flow gauge height is presently 1.2 metres.

The highest recorded gauge heights before and after April 1970 are 2.719 and 3.225 metres respectively.

There are 69 months of incomplete records. Low stage control is the sandy river bed while at medium and high stages the control is Wills Terrace causeway.

2.3.4 G.S. 006126

This section commenced operation in August 1959. There are 55 months with incomplete records. Control at the station is the Ross Highway causeway and the stable stream banks. 32 gaugings have been done with the highest discharge gauged being 145 m$^3$/s on 23/2/67 at a gauge height of 1.219 metres. The highest gauge height recorded is 1.950 metres. Cease to flow gauge height is 0.0m. It should be noted that gaugings at this station have encountered recurring problems such as shifting bed levels and debris interference. Most of the higher ratings are thought to have underestimated discharges.

2.3.5 G.S. 006047

Records commence in 1958 and since then there have been 64 months of incomplete records. 97 gaugings have been done, with the largest discharge being gauged at 52 m$^3$/s on 5/3/72 at a gauge height of 2.347 metres. Highest recorded gauge height is 3.164 metres on 17/4/61. Cease to flow gauge height is 0.456 metres.
2.3.6 Discussion

Figure 2 shows an example of monthly flows at the three stations on the Todd River. Flow volumes at G.S. 006009 are often larger than at G.S. 006046. This would be expected for high return period floods because G.S. 006009 commands a larger catchment. However the Wills Terrace Station often has smaller volumes than the Wigley Gorge Station. This can be partly attributed to large losses in the Todd River at low flows. Similarly at G.S. 006126, flow volumes at this station are not always larger than at stations further upstream.

2.4 Evaporation

The Bureau of Meteorology has operated a class A pan at the present Alice Springs Airport 15 km south of the township since 1955. The Water Resources Branch has operated three class A pans at other potential Recreational Damsites. Although the first of these was installed in September 1973, there are in total only forty months of evaporation records available.

It can be expected that the damsite to the north of Alice Springs and the MacDonnell Ranges would have slightly lower class A pan evaporation than that measured by the Bureau of Meteorology at the present airport. This is because the Range, and the hilly region north of the Range would protect the site from most dry winds. The three pans operated by the Department are in similar topography to the damsite.

A correlation factor of 0.90 has been applied to the evaporation measured at the Alice Springs Airport to transform it to the damsite. This factor was arrived at after a comparison of the evaporation data collected by the Department and from the Alice Springs Airport. The expected class A monthly evaporation rates at the damsite are included in Table 4.

Garrett and Hoy (1978) include monthly pan coefficients under various climatic conditions for lakes of various depths and sizes. The coefficients at Rifle Creek Reservoir in Queensland have been used as a basis for calculating the coefficients at Alice Springs. Table 3 shows some general information on the Rifle Creek Reservoir and the Alice Springs Dam.
FLOW COMPARISONS ON TODD RIVER

KEY
- Wigley Gorge (GS 006046)
- Wills Terrace (GS 006009)
- Heavitree Gap (GS 006126)
- Charles River (GS 006047)

FIGURE 2

Period of Incomplete Records:
Wills Terrace
Wigley Gorge
Heavitree Gap
Charles River
### Table 3

**COMPARISON OF RESERVOIRS**

<table>
<thead>
<tr>
<th>Location</th>
<th>Lat. (°S)</th>
<th>Long. (°E)</th>
<th>Approx. elev. (m) A.H.D.</th>
<th>Area (ha)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rifle Creek</td>
<td>20° 57'</td>
<td>139° 35'</td>
<td>420</td>
<td>190</td>
</tr>
<tr>
<td>Alice Springs</td>
<td>23° 43'</td>
<td>139° 50'</td>
<td>590</td>
<td>100</td>
</tr>
</tbody>
</table>

Figure 1g of Garrett's and Hoy's paper gives monthly pan coefficients for the Rifle Creek Reservoir for lake depths of 5 metres and 20 metres. The average of the pan coefficients for these two depths has been used. Monthly evaporation for the Alice Springs dam is shown in Table 4.

### Table 4

**ESTIMATED RESERVOIR EVAPORATION**

<table>
<thead>
<tr>
<th>Month</th>
<th>Class A Pan evaporation (mm)</th>
<th>Pan Coefficients</th>
<th>Reservoir evaporation (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>January</td>
<td>372</td>
<td>.80</td>
<td>298</td>
</tr>
<tr>
<td>February</td>
<td>328</td>
<td>.76</td>
<td>249</td>
</tr>
<tr>
<td>March</td>
<td>265</td>
<td>.71</td>
<td>202</td>
</tr>
<tr>
<td>April</td>
<td>209</td>
<td>.72</td>
<td>150</td>
</tr>
<tr>
<td>May</td>
<td>142</td>
<td>.72</td>
<td>102</td>
</tr>
<tr>
<td>June</td>
<td>104</td>
<td>.59</td>
<td>61</td>
</tr>
<tr>
<td>July</td>
<td>107</td>
<td>.51</td>
<td>55</td>
</tr>
<tr>
<td>August</td>
<td>145</td>
<td>.52</td>
<td>75</td>
</tr>
<tr>
<td>September</td>
<td>212</td>
<td>.51</td>
<td>108</td>
</tr>
<tr>
<td>October</td>
<td>274</td>
<td>.58</td>
<td>159</td>
</tr>
<tr>
<td>November</td>
<td>313</td>
<td>.56</td>
<td>175</td>
</tr>
<tr>
<td>December</td>
<td>358</td>
<td>.67</td>
<td>240</td>
</tr>
<tr>
<td>Year Total</td>
<td>2849</td>
<td>0.64 (av)</td>
<td>1923</td>
</tr>
</tbody>
</table>

**2.5 Impoundment Data**

The relationships between stage, surface area and dam capacity are shown in Figure 3. These relationships are based on 1:2400 photogrammetric contour maps. Forbes (1965) determined similar curves for the damsites from 1:2400 stadia survey contour maps. The relationships shown in Figure 3 and those derived by Forbes are within 5% of each other.
FIGURE 3

CAPACITY - AREA CURVES OF DAM AT OLD TELEGRAGH STATION
3. ANALYSIS OF DATA

3.1 General

This section discusses the analysis of the hydrographic data to determine flood frequency, unitgraphs and maximum probable floods, and also the generation of stream flow sequences.

3.2 Flood Frequency

At G.S. 006046 peak annual gauge heights are known for years 1960/61 to 1977/78 except for 1975/76. The peak water year floods and gauge heights are included in Table 6. The peak flood in 1975/76 was in early February, but there are no records from 6/2/76 to 16/2/76.

Three approaches were used to analyse records from G.S. 006046. These were:

Case (a) Ignore the 1975/76 water year and analyse 17 years of records.

Case (b) Assume the peak discharge at G.S. 006046 in 1975/76 is of the same order of magnitude as that at G.S. 006009 in the same year.

Case (c) Estimate a peak discharge for 1975/76 by using a rainfall - peak discharge correlation. This correlation used maximum daily rainfall from both Alice Springs Post Office and Bond Springs. Ranked peak discharges at Wigley Gorge and ranked maximum daily rainfalls were plotted at points of equal probability. Close examination indicated that storms in the Todd River catchment are often isolated thunderstorms. This is thought to indicate that a rainfall event of a particular return period need not necessarily be associated with a stream flow of the same return period. Correlation between ranked rainfall and runoff at G.S. 006046 resulted in a correlation of 0.92 for Alice Springs rainfall data and 0.88 for Bond Springs rainfall data. Using rainfall data from 1975/76 the peak discharge at G.S. 006046 in that year was estimated to be 650 m³/s. This is considered to be excessively high especially when compared with the peak discharge at G.S. 006009 and G.S. 006126 (see Table 6). Correlation with the actual storm also overestimated the peak discharge.
A log-Pearson Type III distribution was fitted to the data for cases (a), (b) and (c). Low flows were treated in accordance with the recommendation in "Australian Rainfall and Runoff" section 9.2.9. The distribution from case (b) was selected as being a suitable representation of the flood frequency although the difference between (a) and (b) is small and both distributions well within the confidence limits of each other. The adopted flood frequency curve for G.S. 006046 is shown in Figure 4.

At G.S. 006009, maximum gauge heights for all the water years 1952/53 to 1977/78 are known and it has been possible to reliably estimate the peak annual discharges. (see Table 6). A log-Pearson Type III distribution was fitted to the data using the technique outlined in "Australian Rainfall and Runoff" section 9.2.9 (see Figure 5).

At G.S. 006126 only the water years 1965/66, 1974/75 and 1975/76 have complete records though the maximum gauge heights are known for all years since 1960/61 except 1976/77 (see Table 6). The very small peak discharge for 1976/77 was estimated from G.S. 006046 and G.S. 006009. Again a log-Pearson Type III distribution was fitted to the data (Figure 6) using the same techniques as above. The small number of records and the large number of low discharges result in the confidence limits for the G.S. 006126 annual flood frequency distribution being very large, especially when compared with the confidence limits obtained for G.S. 006046 and G.S. 006009. A comparison of confidence limits for the 100 year return period flood is shown in Table 5.

Table 5
CONFIDENCE LIMITS FOR 100 YEAR RETURN PERIOD FLOOD

<table>
<thead>
<tr>
<th></th>
<th>G.S. 006046</th>
<th>G.S. 006009</th>
<th>G.S. 006126</th>
</tr>
</thead>
<tbody>
<tr>
<td>5% Limit (m³/s)</td>
<td>2300</td>
<td>1800</td>
<td>3800</td>
</tr>
<tr>
<td>Value (m³/s)</td>
<td>1040</td>
<td>830</td>
<td>1120</td>
</tr>
<tr>
<td>95% Limit (m³/s)</td>
<td>720</td>
<td>620</td>
<td>660</td>
</tr>
</tbody>
</table>

Table 5 also demonstrates that even at a fairly high return period the adopted peak discharge at G.S. 006009 is smaller than that at G.S. 006046 and G.S. 006126.
FIGURE 4

FLOOD FREQUENCY CURVE FOR GS006046, TODD RIVER AT WIGLEY GORGE
FIGURE 5

FLOOD FREQUENCY CURVE FOR GS 006 009, TODD RIVER AT WILLS TERRACE
FIGURE 6

FLOOD FREQUENCY CURVE FOR GS 006 126, TODD RIVER AT HEAVITREE GAP
Table 6

PEAK ANNUAL DISCHARGES ON TODD RIVER

<table>
<thead>
<tr>
<th>Water Year</th>
<th>G.S.006046 G.H.m Q(m³/s) Date</th>
<th>G.S.006009 G.H.m Q(m³/s) Date</th>
<th>G.S.006126 G.H.m Q(m³/s) Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>1952/53</td>
<td>2.377 305 5/2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>53/54</td>
<td>1.920 113 25/1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>54/55</td>
<td>1.524 36 24/10</td>
<td></td>
<td></td>
</tr>
<tr>
<td>55/56</td>
<td>2.621 471 24/2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>56/57</td>
<td>1.400 25 19/6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>57/58</td>
<td>1.830 85 9/12</td>
<td></td>
<td></td>
</tr>
<tr>
<td>58/59</td>
<td>1.768 71 22/11</td>
<td></td>
<td></td>
</tr>
<tr>
<td>59/60</td>
<td>1.981 130 29/1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>60/61</td>
<td>3.231 197 16/4</td>
<td>1.981 130 18/4</td>
<td>1.220 272 17/4</td>
</tr>
<tr>
<td>61/62</td>
<td>3.444 235 14/1</td>
<td>1.981 130 14/1</td>
<td>0.860 120 14/1</td>
</tr>
<tr>
<td>62/63</td>
<td>4.374 420 15/5</td>
<td>2.591 448 15/5</td>
<td>1.134 229 15/5</td>
</tr>
<tr>
<td>63/64</td>
<td>1.420 17 20/10</td>
<td>1.402 25 20/10</td>
<td>0.472 30 26/10</td>
</tr>
<tr>
<td>64/65</td>
<td>3.210 195 14/10</td>
<td>1.951 120 14/10</td>
<td>0.08 103 14/10</td>
</tr>
<tr>
<td>65/66</td>
<td>4.791 517 22/1</td>
<td>2.719 563 22/1</td>
<td>1.030 184 21/1</td>
</tr>
<tr>
<td>66/67</td>
<td>3.658 273 5/2</td>
<td>2.210 212 5/2</td>
<td>1.189 256 5/2</td>
</tr>
<tr>
<td>67/68</td>
<td>2.774 131 2/3</td>
<td>1.981 130 2/3</td>
<td>0.077 7 2/3</td>
</tr>
<tr>
<td>68/69</td>
<td>1.463 19 14/3</td>
<td>1.266 16 14/3</td>
<td>0.117 18 3/12</td>
</tr>
<tr>
<td>69/70</td>
<td>0.320 0 all yr</td>
<td>1.200 0 all yr</td>
<td>0.000 0 all yr</td>
</tr>
<tr>
<td>70/71</td>
<td>1.433 18 28/2</td>
<td>1.550 5 7/12</td>
<td>1.220 5 7/12</td>
</tr>
<tr>
<td>71/72</td>
<td>4.398 423 5/5</td>
<td>3.140 371 5/3</td>
<td>1.950 751 5/3</td>
</tr>
<tr>
<td>72/73</td>
<td>1.625 27 14/6</td>
<td>1.900 34 15/6</td>
<td>0.267 11 15/6</td>
</tr>
<tr>
<td>73/74</td>
<td>4.655 483 30/12</td>
<td>3.255 422 25/11</td>
<td>1.850 742 23/1</td>
</tr>
<tr>
<td>74/75</td>
<td>4.860 535 4/9</td>
<td>1.770 20 13/11</td>
<td>0.286 9 2/10</td>
</tr>
<tr>
<td>75/76</td>
<td>See Text</td>
<td>2.880 270 8/2</td>
<td>1.340 341 8/2</td>
</tr>
<tr>
<td>76/77</td>
<td>4.800 520 15/3</td>
<td>3.050 336 15/3</td>
<td>N.R. N.R.</td>
</tr>
<tr>
<td>77/78</td>
<td>2.330 80 21/11</td>
<td>2.080 59 21/11</td>
<td>0.545 42 21/11</td>
</tr>
</tbody>
</table>

Notes: 1. N.R. - no record
2. G.S.006009 changed location in April 1970.
It is not known whether this is a characteristic of the catchment or if it is due to the sequence of flood events. It can be partly attributed to high transmission losses into alluvial aquifers below and adjacent to the stream. It should also be noted that, as indicated in section 2.3, there have been no high stage ratings at G.S. 006046 and G.S. 006126 and that in addition to the confidence limits shown in Figures 4 and 5 the accuracy of the rating curve extension must be considered. A comparison of the flood frequency relationships at G.S. 006009 predicted by different authors is shown in Table 7. The relationship predicted in this report agrees satisfactorily with the findings of these other studies.

Table 7

<table>
<thead>
<tr>
<th>Return Period (years)</th>
<th>This Report</th>
<th>Hug</th>
<th>C.M.&amp; P.</th>
<th>Reinhard</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>410</td>
<td>450</td>
<td>480</td>
<td>410</td>
</tr>
<tr>
<td>20</td>
<td>540</td>
<td>590</td>
<td>600</td>
<td>600</td>
</tr>
<tr>
<td>50</td>
<td>740</td>
<td>790</td>
<td>840</td>
<td>900</td>
</tr>
<tr>
<td>100</td>
<td>900</td>
<td>950</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Figure 7 shows the flood frequency distribution for both G.S. 006046 and G.S. 006009 plotted on a log-log scale. An additional distribution has been shown in Figure 7 to represent the flood frequency at the damsite.

The estimates of peak discharge for selected return periods at the damsite are shown in Table 8.
Table II
ESTIMATED FLOOD FREQUENCY AT DAMSITE

<table>
<thead>
<tr>
<th>Return Period (years)</th>
<th>Peak Annual Discharge (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>350</td>
</tr>
<tr>
<td>10</td>
<td>490</td>
</tr>
<tr>
<td>20</td>
<td>650</td>
</tr>
<tr>
<td>50</td>
<td>850</td>
</tr>
<tr>
<td>100</td>
<td>1000</td>
</tr>
<tr>
<td>500</td>
<td>1400</td>
</tr>
</tbody>
</table>

3.3 Unitgraphs

Using the method of unitgraph derivation developed by Ash (1973) two one hour unitgraphs were derived for the Todd River at G.S. 006046 and at G.S. 006009. They are shown in Figure 8.

Many of the recorded hydrographs were found to be unacceptable for use in the Ash Method because they were either multiple peaked or the peak discharge was too small to be regarded as a flood. The method only allows sharply peaked hydrographs to be considered. At G.S. 006046 the unitgraph was derived from floods on the following dates: 17/4/61, 15/5/63, 13/10/64, 21/1/66, 6/1/66, 2/3/68, 30/12/73, 14/3/77, and 25/11/77. At G.S. 006009 the floods occurring on the following dates were used: 23/2/56, 15/5/63, 25/11/73, 15/12/75, 15/3/77 and 21/11/77.

The unitgraph for G.S. 006009 has a longer recession limb and a steeper rising limb. This is partly due to the input from the Charles River. Response time of the Charles River is quicker than the response time of the Todd River and this influences the time of the rising limb at G.S. 006009.

An average unitgraph (Figure 8) has been developed for input of hydrographs to the reservoir for examination of flood routing effects of the dam.

3.4 Maximum Probable Flood

Estimates of the maximum probable flood have been derived from three sources.
FIGURE 7
Ash (1978) includes a set of curves which give peak discharge for various return periods, and maximum known floods for Australia and the World, as a function of catchment area. Use of these curves requires estimation of the average stream slope. Ash has suggested a rainfall factor to be applied to these curves to adjust the discharges for arid regions. Using the curves and the rainfall factor, the 100 year return period flood at the damsite is estimated to have a peak discharge of 1000 m$^3$/s and the maximum probable flood is estimated to have a discharge of 4000 m$^3$/s.

The Bureau of Meteorology (1979) has estimated the maximum probable precipitation (MPP) for two storm durations in the Alice Springs Region. Using the U.S. Thunder Storm Model, the MPP for a 3 hour duration storm is estimated to be 170mm. By transposing recorded storms the MPP for a 12 hour duration is estimated to be 360mm. For the 12 hour duration storm, three likely temporal patterns have been supplied. The most critical temporal pattern gives a peak discharge of 4,620 m$^3$/s. The initial loss rate and the continuing loss rate were both set to zero, in order to get the maximum runoff.

A third estimate of the maximum probable flood relied on information from a storm at Rumbalara Siding in February 1976. Rumbalara Siding is approximately 170km south of Alice Springs on the Central Australian Railway. On the 9th of February, following three days of heavy rainfall, a 9am reading of 415mm was recorded. Because the temporal pattern for the storm is not available the 24 hour temporal pattern for arid regions in Figure 3.7 of "Australian Rainfall and Runoff" was used. The one hour unitgraph for the damsite (Figure 8) was converted to a four hour unitgraph using the principle of superposition with the volume of excess rainfall accordingly adjusted. Using the four hour unitgraph and six theoretical excess rainfall periods the probable maximum flood was estimated to be 5,300 m$^3$/s. Again, the initial and continuing loss rates were set to zero.

This report uses the maximum probable flood derived from the maximum probable precipitation estimates.
3.5 Historic Streamflows.

As was stated in section 2.3.1 there are no usable historic records of flow at the damsite. Records of flow are available from G.S. 006046 upstream and from G.S. 006009 downstream of the damsite. Section 2.2 indicates that the period of streamflow records include two extreme rainfall-patterns - a nine year drought and a five year wet period.

Using all four stations: G.S. 006046, G.S. 006009, G.S. 006126 and G.S. 006047 an attempted correlation of runoff volumes with catchment area for each month was found to be ineffective. The additional effect of the Charles River inflow at G.S. 006009 was removed using records at G.S. 006047. All of the graphs showed a great scatter of points. This is attributed to at least two reasons: (a) the non uniform spatial pattern of storms and (b) large infiltration losses in the river channel. An unsuccessful attempt was made to find a consistent relationship of the relative sizes of floods as they progress down the Todd River.

Being unable to determine a satisfactory area - runoff relationship the historic flows (m³/month) at the damsite were taken as the average of those at Wigley Gorge and Wills Terrace.

In order to do this it proved necessary to complete the records at both stations. This was done starting at September 1960. In many months although the record for the whole month is incomplete this incompleteness is due only to 2 or 3 days of no records. By examining the flow during those days at the other stations it was possible to estimate the flow which probably would have occurred on those 2 or 3 days. For the rest of the month the flow volume is known and hence the monthly flow volumes is estimated. In cases where records for the whole month, or a substantial part of the month are not available or are unreliable, or where both G.S. 006046 and G.S. 006009 do not have records, use was made of records at G.S. 006126 and G.S. 006047. In many cases, by examining the records at Charles River and Heavitree Gap and rainfall records it was concluded that no flow occurred for the period of no records. However, in the other cases it was obvious that substantial flows did occur and for these times it was necessary to estimate the flow volumes. A correlation between monthly rainfall at both Alice Springs and Bond Springs with flow volume/month at both G.S. 006046, and G.S. 006009 was developed. Using the monthly rainfall, an estimate of the volume of runoff for the month of no records is obtained.
Rather than fit a line of best fit through the points, as was done for the synthetic flows (see section 3.6) the points plotted were joined directly. Figures 9 and 10 show the points used. Bond Springs rainfall points are not shown in these figures.

Thus 'Complete Records' were available at Wigley Gorge and Wills Terrace and the average of these flow volumes for each month was used as the historic flow record at the Damsite.

3.6 Synthetic Streamflow Generation.

A rainfall runoff correlation was found. Using the 105 years of monthly rainfall records at the Telegraph Station and Alice Springs Post Office, this correlation was used to generate 105 years of monthly synthetic streamflow.

Using the Kolmogorov - Smirnov test, Spiegel (1965), it may be concluded that the rainfall records at the Telegraph Station (1874 - 1931) are from the same population as the rainfall records at the Post Office (1932 - 1972). Bond Springs rainfall records are also from the same population. The rainfall records at the Post Office and the Telegraph Station can therefore be used to extend the records.

Monthly rainfalls at Alice Springs Post Office and monthly runoff volumes at G.S. 006046 and G.S. 006009 over corresponding periods were ranked and points of equal probability were plotted.

At first this correlation was attempted on a month by month basis but due to lack of sufficient data and the large number of zeros in the record (40% to 70%) it is impossible to find a definite trend. The method was then done on a seasonal basis. Two "seasons" were chosen: October to March and April to September.

Using these two seasons, separate curves for Wigley Gorge and Wills Terrace were derived. The curves were extrapolated to get higher flow than recorded. This was done by firstly extending the frequency curves of monthly rainfall and monthly flow volumes.
Rainfall and runoff points of equal probability from these extended frequency curves were then plotted on the correlations shown in Figures 9 and 10.

At low flows (See insert Figure 9), the historic points give a more realistic correlation than the ranked points. This is because at low rainfalls, storm distribution is not uniform over the catchment, losses are high and rainfall at Alice Springs is less indicative of flow in the catchment than for the higher rainfalls.

The accuracy of the correlations shown in Figures 9 and 10 was tested using the Kolmogorov–Smirnov test (Spiegel 1965). This test was done on the historic flows at the damsite and the generated synthetic flows. For both seasons they were found to be within the 5% confidence limit.

Using the rainfall-runoff correlation, 105 years of synthetic flows were generated on a monthly basis.

3.7 Statistical Streamflow Generation

Two methods were investigated to generate stochastic streamflow sequences based on streamflow statistics.

One method generates log normal annual flows having zero serial correlation. i.e.

\[ \log Q = \log \bar{Q} + t \times (\log \sigma) \]

where \( t \) is a random variate
\( \log \bar{Q} \) is the mean of the log of the annual flows
and \( (\log \sigma) \) is the standard deviation.

These statistics are based on the 105 years of synthetic flows. An arbitrary cutoff point is applied to the generated annual flows to account for low flow years.

The statistically generated annual flows are ranked and the original annual flows are ranked. Monthly temporal patterns corresponding to the equivalent rank to the original flows are fitted to the statistically generated flows.

The other method assumes correlation between months. It is based on the Thomas and Piering seasonal model (McMahon & Mein 1978) applied to the 18 years historical data at the damsite. To allow for the large number of zeroes in each monthly sequence, Beard's Artificial Negative Analysis Method, Beard (1973), is used.
RAINFALL/RUNOFF CORRELATION OCTOBER TO MARCH

FIGURE 9

RANKED RAINFALL (mm/month) AT ALICE SPRINGS POST OFFICE

RANKED RUNOFF (Mm³/month)

KEY

- GS006009
- 3S006048
- GS006009, Extrapolated from equal probability
- GS006048, Extrapolated from equal probability
- Runs on Historic

SEE INSET

0 10 20 30 40 50 60 70
0 10 20 30 40
The seasonal model is based on the normal probability distribution but problems were encountered in reducing the large skew values, approximately 3, for monthly sequences for use in the model.

Streamflow sequences based on the first method have been used as data for the dam simulation (Section 4).
4. DAM SIMULATION

4.1 General

The behaviour of the dam was simulated with a computer programme. The programme simulates the behaviour of a dam by calculating levels, volumes and areas of the dam, on a monthly basis. The programme is given:
(a) rainfall,
(b) runoff from the catchment,
(c) evaporation and pan coefficients
(d) draft and
(e) area/volume/elevation data for the impoundment.
The user specifies the starting level.

If the dam overflows, the programme calculates the volume of water discharged by the spillway.

At the end of the simulation the programme prints the following statistics: for each month and the whole year, the number of months the area was between the areas given in the area/volume/elevation curves; and all durations at which the area was below a specified area.

Seepage was neglected in all simulations. Preliminary site investigations suggest that seepage is small. If this is subsequently proved to be false, the programme could be rerun with higher evaporation to allow for such seepage losses.

The programme was run with six constant drafts: zero, 0.5, 1, 2, 3, 4 and 5 mega cubic metres/year. This was done to assess the ability of the storage to supplement Alice Springs Water Supply or to compensate for reduced yield from the town basin. The simulation was run at three primary spillway levels: 591 metres, 592.5 metres and 594 metres A.H.D.. For all combinations, the historic and synthetic flows were used to simulate the behaviour.
4.2 Results Using Synthetic Streamflows.

Curves of the availability of area, volume and elevation as simulated with the synthetic flows are shown in figures 11, 12 and 13. Each graph shows the behaviour at different spillway levels and give the result found for the months October to March. The curves describing behaviour for April to September are similar.

For low drafts of 0.0, 0.5 and 1 Mm³/year, median values of area, volume and elevation are very close to full dam levels. As the drafts increase median values of area, volume and elevation decrease. For 75% of the time, area, volume and elevations corresponding to the low drafts are always above those corresponding to half the dam's capacity.

With the primary spillway level at 594 metres A.H.D. (Figure 13) area, volume and elevations corresponding to the low drafts are above 80% of the full capacity for 75% of the time.

The flatness of the curves is attributed to the small capacity of the dam - 5.1 Mm³ for a spillway level of 594 metres in comparison to the stream flows i.e. mean annual flow volume for the 105 years is 12.1 Mm³, median annual flow is 7.8 Mm³.

4.3 Results Using Historic Streamflows.

The result of the dam simulation using the historic flows were found to be very similar to those of the synthetic flows. These results are summarized in figures 11, 12 and 13.

4.4 Results Using Statistically Generated Streamflows.

These flows were used in the dam simulation for two combinations of drafts and spillway levels and verified the results found for the synthetic and historic flows.
FIGURE 12

SIMULATED DAM BEHAVIOUR. SPILLWAY AT 592.5 METRES.
FIGURE 13

SIMULATED DAM BEHAVIOUR SPILLWAY AT 594 METRES
4.5 Gould Analysis

As an independent check on the dam simulation model, a programme was written for Goulds Probability Matrix Method, as modified by McMahon and Mein (1978) to account for monthly rather than annual failures.

The method assumes an annual serial correlation of zero, and is independent of the historical sequencing of flows and initial reservoir conditions. The programme allows for monthly variations in draft and evaporation. Dead storage can be defined as a fraction of reservoir capacity. Probability of failure can be calculated either at steady state or as a time dependent function of the starting conditions.

Calculated steady state probabilities (Table 9) fitted the duration curves for percent of time exceeded (Figure 11, 12 and 13) very closely. Transient failure probabilities assuming the reservoir is initially empty are also shown in Table 9.

| Table 9 |
| PROBABILITY OF AREA BEING GREATER THAN INDICATED SIZE WITH PRIMARY SPILLWAY AT 594M AND A DRAFT OF 1.0 MM³/YEAR. |

<table>
<thead>
<tr>
<th>Probability in Percentage.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Years after Construction</td>
</tr>
<tr>
<td>-----------------------------</td>
</tr>
<tr>
<td>1</td>
</tr>
<tr>
<td>2</td>
</tr>
<tr>
<td>3</td>
</tr>
<tr>
<td>Steady State</td>
</tr>
</tbody>
</table>
4.6 Other Results

Of all the simulations performed (approximately forty five) the maximum duration of a period when the draft could not be met was 20 months. This was for the historic flow case with a draft of 5Mm³/year and at the lowest spillway level of 591 metres. This occurred from April 1969 to November 1970. During this time there was no surface runoff. With a draft of 1Mm³/year or less, at spillway levels of 592.5 metres and 594 metres the draft was able to be met for the whole period of simulation.

For the hypothetical case of a complete drought, i.e. no rain or no runoff, at a draft 1 Mm³/year and at a spillway level of 594 metres, it takes 27 months for the dam to empty. This is an extreme case and is not expected to occur.

Table 10 shows the total number of failures found from the dam simulation. In this context failure is defined as a period of time when the draft could not be met.

Table 10

<table>
<thead>
<tr>
<th>TOTAL NUMBER OF FAILURES (Months)</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>Draft Mm³/yr.</th>
<th>594</th>
<th>592.5</th>
<th>591</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>48 (22%)</td>
<td>61 (28%)</td>
<td>76 (35%)</td>
</tr>
<tr>
<td>4</td>
<td>35 (16%)</td>
<td>55 (25%)</td>
<td>60 (28%)</td>
</tr>
<tr>
<td>3</td>
<td>22 (10%)</td>
<td>32 (15%)</td>
<td>43 (20%)</td>
</tr>
<tr>
<td>2</td>
<td>6 (3%)</td>
<td>13 (6%)</td>
<td>23 (11%)</td>
</tr>
<tr>
<td>1</td>
<td>0 (0%)</td>
<td>0 (0%)</td>
<td>4 (2%)</td>
</tr>
<tr>
<td>0.5</td>
<td>0 (0%)</td>
<td>0 (0%)</td>
<td>0 (0%)</td>
</tr>
<tr>
<td>0</td>
<td>0 (0%)</td>
<td>0 (0%)</td>
<td>0 (0%)</td>
</tr>
</tbody>
</table>

Note: 1. Probability of failure in brackets.
The simulation indicates that if the dam is constructed on the long term average, stream flow volumes will be reduced by 20%. Table 11 shows a comparison of flow in the Todd River before and after the dam, on a daily basis.

Table 11

<p>| COMPARISON OF FLOW IN THE TODD RIVER BEFORE AND AFTER DAM (1960/62 - 1977/78) (in days) |
|----------------------------------|---------------|---------------|---------------|</p>
<table>
<thead>
<tr>
<th>Spillway Level</th>
<th>594m</th>
<th>592.5m</th>
<th>591m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Drafts Mega m³/year</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.</td>
<td>844 (55%)</td>
<td>862 (56%)</td>
<td>874 (57%)</td>
</tr>
<tr>
<td>4.</td>
<td>895 (58%)</td>
<td>911 (59%)</td>
<td>949 (62%)</td>
</tr>
<tr>
<td>3.</td>
<td>937 (61%)</td>
<td>991 (65%)</td>
<td>991 (65%)</td>
</tr>
<tr>
<td>2.</td>
<td>991 (65%)</td>
<td>1004 (65%)</td>
<td>1004 (65%)</td>
</tr>
<tr>
<td>1.</td>
<td>1009 (66%)</td>
<td>1019 (66%)</td>
<td>1041 (68%)</td>
</tr>
<tr>
<td>0.5</td>
<td>1019 (66%)</td>
<td>1038 (69%)</td>
<td>1101 (72%)</td>
</tr>
<tr>
<td>0.0</td>
<td>1103 (72%)</td>
<td>1134 (74%)</td>
<td>1219 (79%)</td>
</tr>
</tbody>
</table>

Note: 1. Percentage of original number of days in brackets.
5. RESERVOIR ROUTING

5.1 General

From preliminary investigations of costing and operation, a draft of 1Mm³/year and a spillway level of 594 metres was chosen. Flood routing then proceeded with the following assumed spillway arrangement:

The main or primary spillway is approximately 150 metres downstream of the Old Telegraph Station and is at 594 metres A.H.D.. The secondary spillway is upstream of the Old Telegraph Station. This spillway has a crest level of 596 metres A.H.D.. Another two metres higher than the secondary spillway is the top of the dam wall. The spillways are arranged so that no water will spill over the Western Saddle and thus onto the site of the Old Telegraph Station. Two cases of spillways were investigated and these are shown in Table 12.

Table 12

<table>
<thead>
<tr>
<th>Elevation (m A.H.D.)</th>
<th>Case (a)</th>
<th>Case (b)</th>
</tr>
</thead>
<tbody>
<tr>
<td>594</td>
<td>50</td>
<td>40</td>
</tr>
<tr>
<td>596</td>
<td>130</td>
<td>120</td>
</tr>
<tr>
<td>598</td>
<td>200</td>
<td>190</td>
</tr>
</tbody>
</table>

5.2 Results

Floods with return periods of 5, 10, 20, 50, 100 and 500 years, and the maximum probable flood were routed through the dam using Pulse Method (see Wilson (1974)). For both cases of spillway arrangement (i.e. Case (a) and Case (b), Table 12) alternative routes were done with the dam starting full and at a level of 592.7 metres. 592.7 metres is the elevation which us exceeded 75% of the time for a draft of 1Mm³/year and a spillway level of 594 metres.
Tables 13 and 14 give the results of the routes for cases (a) and (b) respectively, starting conditions being full. Although the attenuation is larger when the starting level is 592.7 metres, it is not significantly larger - for example - case (a), 100 year return period flood is discharged as 560 m$^3$/s when the starting level is 592.7, compared with 618 m$^3$/s in Table 13.

Table 13

**INFLows AND OUTFLOWS OF DAM Case (a)**

<table>
<thead>
<tr>
<th>Floods (yrs)</th>
<th>Starting Elevation - 534m</th>
<th>Inflow (m$^3$/s)</th>
<th>Discharge (m$^3$/s)</th>
<th>Max. Level (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>350</td>
<td>168</td>
<td>595.7</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>490</td>
<td>251</td>
<td>596.2</td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>650</td>
<td>348</td>
<td>596.5</td>
<td></td>
</tr>
<tr>
<td>50</td>
<td>850</td>
<td>502</td>
<td>597.0</td>
<td></td>
</tr>
<tr>
<td>100</td>
<td>1000</td>
<td>618</td>
<td>597.3</td>
<td></td>
</tr>
<tr>
<td>500</td>
<td>1400</td>
<td>951</td>
<td>598.0</td>
<td></td>
</tr>
<tr>
<td>M.P.F.</td>
<td>4620</td>
<td>4274</td>
<td>602.0</td>
<td></td>
</tr>
</tbody>
</table>

Table 15 is an example output of Puls' Routing calculations. All discharges are in m$^3$/s. It serves to illustrate the rapid increase in level water as a flood passes. In this case it rises 2.1 metres in 4 hours. For the 100 year return period flood, it rises 3.3 metres in 4 hours.
Table 14
INFLOWS AND OUTFLOWS OF DAM Case (b)

<table>
<thead>
<tr>
<th>Floods (yrs)</th>
<th>Starting Elevation - 594m</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Return Periods</td>
</tr>
<tr>
<td>5</td>
<td>350</td>
</tr>
<tr>
<td>10</td>
<td>490</td>
</tr>
<tr>
<td>20</td>
<td>650</td>
</tr>
<tr>
<td>50</td>
<td>850</td>
</tr>
<tr>
<td>100</td>
<td>1000</td>
</tr>
<tr>
<td>500</td>
<td>1400</td>
</tr>
<tr>
<td>M.P.F.</td>
<td>4620</td>
</tr>
</tbody>
</table>

For case (a) the secondary spillway is expected to be used, on average, once every 8.7 years. For case (b) it is once every 5.4 years. The spillway at 598 metres (dam wall) has return periods of use of 500 and 380 years for cases (a) and case (b) respectively. These return periods are based on the dam starting full. They are slightly higher if the dam starts at elevation 592.7m.
### Table 15

#### EXAMPLE OF FLOOD ROUTE

**ALICE SPRINGS DAM, 10 Year Return Period Flood**

<table>
<thead>
<tr>
<th>Height (m)</th>
<th>Width (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>594.00</td>
<td>50</td>
</tr>
<tr>
<td>596.00</td>
<td>130</td>
</tr>
<tr>
<td>598.00</td>
<td>200</td>
</tr>
</tbody>
</table>

**STARTING ELEVATION = 594 m.**

<table>
<thead>
<tr>
<th>Hour</th>
<th>Rout</th>
<th>Infl o</th>
<th>I (av)</th>
<th>Disch</th>
<th>S-D/2</th>
<th>S+D/2</th>
<th>Level</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>1418</td>
<td>1426</td>
<td>594.02</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>2</td>
<td>1</td>
<td>33</td>
<td>1425</td>
<td>1429</td>
<td>594.32</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>3</td>
<td>152</td>
<td>321</td>
<td>16</td>
<td>1493</td>
<td>1814</td>
<td>595.07</td>
</tr>
<tr>
<td>3</td>
<td>4</td>
<td>424</td>
<td>448</td>
<td>87</td>
<td>1729</td>
<td>2175</td>
<td>595.95</td>
</tr>
<tr>
<td>4</td>
<td>5</td>
<td>405</td>
<td>327</td>
<td>208</td>
<td>1967</td>
<td>2294</td>
<td>596.12</td>
</tr>
<tr>
<td>5</td>
<td>6</td>
<td>248</td>
<td>205</td>
<td>243</td>
<td>2050</td>
<td>2255</td>
<td>596.05</td>
</tr>
<tr>
<td>6</td>
<td>7</td>
<td>161</td>
<td>134</td>
<td>227</td>
<td>2028</td>
<td>2162</td>
<td>595.92</td>
</tr>
<tr>
<td>7</td>
<td>8</td>
<td>106</td>
<td>87</td>
<td>204</td>
<td>1957</td>
<td>2045</td>
<td>595.65</td>
</tr>
<tr>
<td>8</td>
<td>9</td>
<td>69</td>
<td>56</td>
<td>163</td>
<td>1882</td>
<td>1938</td>
<td>595.37</td>
</tr>
<tr>
<td>9</td>
<td>10</td>
<td>43</td>
<td>32</td>
<td>124</td>
<td>1814</td>
<td>1946</td>
<td>595.15</td>
</tr>
<tr>
<td>10</td>
<td>11</td>
<td>20</td>
<td>14</td>
<td>96</td>
<td>1750</td>
<td>1764</td>
<td>594.97</td>
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<td>1673</td>
<td>594.77</td>
</tr>
<tr>
<td>12</td>
<td>13</td>
<td>0</td>
<td>0</td>
<td>54</td>
<td>1639</td>
<td>1639</td>
<td>594.67</td>
</tr>
<tr>
<td>13</td>
<td>14</td>
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<td>44</td>
<td>1595</td>
<td>1595</td>
<td>594.52</td>
</tr>
<tr>
<td>14</td>
<td>15</td>
<td>0</td>
<td>0</td>
<td>31</td>
<td>1565</td>
<td>1565</td>
<td>594.47</td>
</tr>
<tr>
<td>15</td>
<td>16</td>
<td>0</td>
<td>0</td>
<td>27</td>
<td>1538</td>
<td>1538</td>
<td>594.42</td>
</tr>
<tr>
<td>16</td>
<td>17</td>
<td>0</td>
<td>0</td>
<td>23</td>
<td>1516</td>
<td>1516</td>
<td>594.32</td>
</tr>
<tr>
<td>17</td>
<td>18</td>
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<td>0</td>
<td>16</td>
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<td>594.27</td>
</tr>
<tr>
<td>18</td>
<td>19</td>
<td>0</td>
<td>0</td>
<td>12</td>
<td>1488</td>
<td>1488</td>
<td>594.22</td>
</tr>
<tr>
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<td>20</td>
<td>0</td>
<td>0</td>
<td>9</td>
<td>1478</td>
<td>1478</td>
<td>594.22</td>
</tr>
<tr>
<td>20</td>
<td>21</td>
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<td>594.17</td>
</tr>
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<td>22</td>
<td>0</td>
<td>0</td>
<td>7</td>
<td>1462</td>
<td>1462</td>
<td>594.17</td>
</tr>
<tr>
<td>22</td>
<td>23</td>
<td>0</td>
<td>0</td>
<td>7</td>
<td>1458</td>
<td>1458</td>
<td>594.12</td>
</tr>
<tr>
<td>23</td>
<td>24</td>
<td>0</td>
<td>0</td>
<td>4</td>
<td>1451</td>
<td>1451</td>
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<tr>
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<td>1445</td>
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<td>594.07</td>
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<tr>
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<td>27</td>
<td>0</td>
<td>0</td>
<td>2</td>
<td>1442</td>
<td>1442</td>
<td>594.07</td>
</tr>
</tbody>
</table>
6. FLOODING IN ALICE SPRINGS

Table 16 shows the expected peak flood discharge at Wills Terrace and Heavitree Gap if the dam is constructed. These values are conservative as they assume that the discharges are reduced by the amount of attenuation achieved when the dam is full at the start of the flood. This was done because of the uncertainty of the effect of the Charles River input at Wills Terrace. At floods of high return periods, normally both the Charles River and the Todd River flow, so if the flood at the dams site is reduced by 350 m$^3$/s (50 year return period flood), then it is reasonable to assume that it is reduced by this same amount at Wills Terrace and Heavitree Gap. The peak discharges in Table 16 have been derived using the results of flood routing for spillway arrangement case (a).

Table 16

PEAK DISCHARGE AT G.S. 006009 and G.S. 006126 AFTER DAM

<table>
<thead>
<tr>
<th>Return Period (yrs)</th>
<th>G.S. 006009</th>
<th>G.S. 006126</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>170 (m$^3$/s)</td>
<td>250 (m$^3$/s)</td>
</tr>
<tr>
<td>20</td>
<td>240 (m$^3$/s)</td>
<td>360 (m$^3$/s)</td>
</tr>
<tr>
<td>50</td>
<td>380 (m$^3$/s)</td>
<td>560 (m$^3$/s)</td>
</tr>
<tr>
<td>100</td>
<td>520 (m$^3$/s)</td>
<td>740 (m$^3$/s)</td>
</tr>
</tbody>
</table>

By comparing Table 16 with figures 5 and 6, it is seen that the peak discharges are reduced considerably, especially at the lower return periods. The new 100 year return period flood at G.S. 006009 is of the same order of magnitude as the present 20 year return period flood. According to maps included in flood studies of Alice Springs conducted by Cameron McNamara and Partners (1978) and Huq (1979) the present 20 year return period flood will inundate approximately 30 ha of developed land to a shallow depth. The present 50 and 100 year return period floods inundate areas in excess of 200 ha.
PART B

SEDIMENTATION STUDY
1. INTRODUCTION

The damming of the Todd River at the Old Telegraph Station will result in two sediment related effects. Upstream of the dam, sediment will deposit in the reservoir, gradually decreasing the useable reservoir area and volume. In contrast the river bed downstream of the dam will scour.

Both effects are due to the perturbation of a steady state system by the construction of a dam. Sediment transport is a complex function of hydraulic and fluid variables, channel geometry and sediment properties. Conceptually the river is considered to be in an equilibrium state before dam construction, with all the interacting variables being compatible. Following dam construction, one or more of these variables will change and the equilibrium between variables is upset. To achieve the new steady state in which the system is compatible with modified conditions, changes will occur in other variables such as bed elevation and channel width.

The aim of this sediment study was to look at these upstream and downstream effects and specifically to examine: 1) the effect of silting on the useful life of the reservoir. 2) the effect of the dam on downstream channel morphology.
2. DATA

2.1 Introduction

Field data is very sparse. Suspended load measurements were made on two floods in March 1965 and on three floods in early 1979. All measurements were taken at G.S. 006009. One measurement of bed load was made in 1979.

Permanent ranges have been established between G.S. 006126 and the dam site for long term monitoring of large scale changes in channel morphology. The locations of the cross-sections are shown in figure 1.

Throughout the sediment section of this report a distinction is made between suspended sediment and bed load. Suspended sediment consists of fine particles, typically having a particle diameter less than 0.06 mm. In the Todd catchment most fine particles would be classified as silt with a small proportion (10%) of clays. Bed load comprises larger particles which move along the river bed by saltation or rolling. An individual bed particle alternates between being at rest and in motion on a random basis. At any observation, the overall impression is one of net particle transport although individual particles may have changed from a moving to a rest state or vice-versa during the observation.

A distinction should also be made between sediment transport capacity and the actual sediment load. Transport capacity is a measure of the sediment that could be transported for a given river section and flow rate and assumes that the sediment source can adequately supply sediment at the rate of transport. In fact, the actual sediment load can be less than the transport capacity because of an inadequate supply of sediment. The rate of suspended sediment transport is very dependent on the availability of fine particles within the catchment. On an alluvial river bed where there exists a large source of bed load material, sediment transport capacity is a reasonable estimate of actual bed load. Total sediment load comprises both suspended load and bed load.
2.2 **Suspended Sediment**

The arid nature of the Todd Catchment in conjunction with grazing as its land use suggests that small diameter suspended sediment transport rates will be high.

The measurements of suspended sediment in 1965 and 1979 were both made during climatically extreme periods. The 1965 water year recorded the second lowest rainfall on record and was preceded by the third lowest rainfall in 1965. In addition, 1965 was the last year of a continuous nine year period of below average rainfalls which was the worst drought on record. In contrast the 1979 water year occurs at the end of a seven year period of above average rainfalls and is the wettest period on record.

Suspended sediment mass flow rates from samples taken at Wills Terrace are plotted against discharge at G.S. 006009 in figure 2. All measurements were made on the falling stage of the hydrograph.

No analysis of suspended sediment particle size was made.

2.3 **Bed Load**

One bed load measurement of 650g/s at a discharge of 10m$^3$/s was made at G.S. 006009 on 4/3/79. This represents 30% of suspended load or 25% of total load.

Estimates of the bed load transport rate at Wills Terrace were made using the Meyer-Peter/Müller formula (Simons and Senturk, 1977). These estimates are sensitive to stream slope values. Stream slopes can be estimated either by assuming a constant Mannings 'n' value and using Mannings equation for uniform steady flow or by approximating the 'bed slope' to the 'stream slope'. An 'n' value of 0.035 was used in calculating bed load transport at Wills Terrace. Bed load transport estimates made for a cross-section downstream of Wills Terrace using alternative formulae (Cameron McNamara 1978) and the estimates for bed load transport at Wills Terrace are shown in figure 2.
FIGURE 2

SEDIMENT TRANSPORT FLOW RELATIONSHIP

**KEY**

- Suspended Sediment: 
  - 7/3/1965
  - 19/3/1965
  - 22/7/1979
  - 3/2/1979
  - 4/3/1979

- Predicted Bedload: 
  - Upstream of Sediment Trap

- 1979 Bedload Measurement

- Meyer-Peter Müller Formula

- Woolf Tree Gap Study

**DISCHARGE (m³/sec)**

**SEDIMENT TRANSPORT RATE (g/sec)**

**Notes:**
- Discharge and sediment transport rate are plotted on a logarithmic scale.
- The graph shows the relationship between discharge and sediment transport rate for different dates and locations.
Bed load transport in the channel section immediately upstream of the reservoir was estimated using the Meyer - Peter/Muller formula with the bed slope approximation and is also shown in figure 2. Because of the steeper channel slope in this section, the bed load transport rate is much higher than the rate at Wills Terrace.

Assuming that transport rate is at transport capacity, the high rate upstream of the damsite suggests that in order to maintain continuity of sediment, bed load material is being stored between the two sections. In fact the Todd River undergoes a distinct change in channel morphology at the damsite. The river bed changes from a shallow sandy, rocky, low storage channel to a deep sand, high storage channel.

Bed samples were taken at third points for cross section ranges upstream from Wills Terrace. Figures 3 and 4 show the range of particle sizes of samples taken at the surface and 1m depth respectively. Samples taken at 1m depth are coarser than surface samples i.e.:-

<table>
<thead>
<tr>
<th>Depth</th>
<th>D50</th>
<th>D90</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0m depth</td>
<td>1.1mm</td>
<td>7mm</td>
</tr>
<tr>
<td>Bed surface</td>
<td>0.75mm</td>
<td>3mm</td>
</tr>
</tbody>
</table>

There is no evidence of particle size sorting between cross-sections downstream from the dam.
SIZE DISTRIBUTION OF BEDLOAD SAMPLES AT BED SURFACE
CROSS SECTIONS 6 TO 13

FIGURE 3
SIZE DISTRIBUTION OF BEDLOAD SAMPLES AT 1 METRE DEPTH
CROSS SECTIONS 6 TO 13

FIGURE 4
3 ANALYSIS

3.1 Introduction

A power relationship between sediment flow rate and stream discharge was used to estimate monthly and annual sediment loads. A trap efficiency model was used to estimate the life of the dam. The feasibility of a sediment trap immediately upstream of the reservoir to trap bed load was examined.

A computer model for transient scour and deposition was used to examine the effects of downstream scour. Several problems were encountered in using this model and its value was in giving an overall impression of where scour would occur for various downstream controls rather than giving absolute depths of scour.

3.2 Transport Relationships

Both floods in 1965 included flow at G.S. 006047. Table 1 shows estimated flow volumes at gauging stations.

Table 1

<table>
<thead>
<tr>
<th>Gauging Station</th>
<th>7/3/65</th>
<th>18-19/3/65</th>
</tr>
</thead>
<tbody>
<tr>
<td>G.S. 006009</td>
<td>0.83</td>
<td>0.69</td>
</tr>
<tr>
<td>G.S. 006046</td>
<td>0.45</td>
<td>0.51</td>
</tr>
<tr>
<td>G.S. 006047</td>
<td>0.13</td>
<td>0.34</td>
</tr>
</tbody>
</table>

It is known flow occurred for the 1979 storms at each of the gauging stations G.S. 006009, G.S. 006046, G.S. 006 047, but flow volumes are not yet available.
Rainfall data for the 1965 storms and the first two storms of 1979 are shown in Table 2.

Table 2
RAINFALL (1965 AND 1979 STORMS) mm

<table>
<thead>
<tr>
<th>RAINFALL STATION</th>
<th>DATE</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>7/3/65</td>
</tr>
<tr>
<td>Water Resources Branch</td>
<td>8.1</td>
</tr>
<tr>
<td>Bond Springs Turnoff</td>
<td>NR</td>
</tr>
<tr>
<td>Bond Springs (pluviograph)</td>
<td>0</td>
</tr>
<tr>
<td>Big Dipper (G.S. 006047)</td>
<td>21.1</td>
</tr>
<tr>
<td>Mt. Lloyd (pluviograph)</td>
<td>16.3</td>
</tr>
<tr>
<td>Yuendumu Road</td>
<td>7.6</td>
</tr>
<tr>
<td>Station Creek</td>
<td>NR</td>
</tr>
<tr>
<td>12 Mile</td>
<td>NR</td>
</tr>
<tr>
<td>Flynn's Grave</td>
<td>NR</td>
</tr>
<tr>
<td>Undoolya Road</td>
<td>NR</td>
</tr>
</tbody>
</table>

Notes. N.R. No Record.

From Table 2, both storms in 1965 appear to have been centred on the Charles and Collyer Creek Catchments. The storm which occurred on the 22/1/79 was generally widespread whereas that which occurred on the 3/2/79 was concentrated in the upper part of the Todd Catchment north of Bond Springs.

Comparing the areal distribution of rainfall for the January 1979 storm with the distribution of the 1965 storms it can be seen that the contributing area for the 1965 flows (i.e. approximately west of the Stuart Highway) forms part of the contributing area for the 1979 flow.
This suggests that differences in suspended sediment transport rates between the higher 1965 rates and the lower 1979 rates may be due to differences in catchment condition, rather than to differences in contributing area.

No correction factor has been applied to transfer suspended sediment data at Wills Terrace to the damsite for two reasons. Firstly there is insufficient data to separate the contribution of the Charles Catchment to suspended sediment load at Wills Terrace. Secondly a large proportion of fine sediment is probably originating in the northern section of the Todd Catchment where even in 1979 vegetation is sparse.

The suspended sediment data for 1979 shows a fall in transport rate for a given discharge for the three floods. This effect may be due to:

1) influence of different contributing areas
2) a general decrease in sediment supply after successive floods.
3) a decrease in sediment supply available for transport on the falling stage of the hydrograph as the peak flow increases.

Without further measurements these effects cannot be isolated.

If there is a general decrease in transport rate for successive floods, any analysis which assumes independent flood events will overestimate suspended sediment load.

For analysis, each flow event was assumed to be independent and power relationships were fitted to the 1965 (drought conditions) and the 1979 (wet conditions) data respectively. Transport rates under 1965 conditions are approximately ten times those under 1979 conditions for equivalent flows. Bed load transport upstream of the impoundment was assumed to follow the relationship for 1979 suspended sediment data. The power relationships are plotted in figure 2.
3.3 DMASS Computer Programme

The DMASS (daily, monthly annual sediment summary) computer programme integrates instantaneous values of suspended sediment transport rate over time. The programme is based on a standard DMAS (daily, monthly annual summary) for flow volumes (Lee 1979) that has been modified to include a power relationship between instantaneous sediment transport rate and instantaneous flow. This relationship is of the form $C = A Q^B$ where $C$ is the instantaneous sediment transport rate in g/s, $Q$ is flow rate in m$^3$/s and $A$ and $B$ are constants.

The DMASS and DMAS programmes were run using historical flows (1953 - 1978) at G.S. 006009 to develop relationships between suspended sediment load and flow volume on a monthly basis. Two runs were made: one assuming 1965 conditions applied throughout the historical record and the other assuming 1979 conditions applied. Figure 5 shows the derived points and the fitted relationship for 1979 conditions and the fitted relationship for 1965 conditions. Because there is generally only one flow per month in the historical record, the monthly relationships between sediment load and stream flow reflect the relationships for individual hydrographs obtained from field data. The points in the monthly relationships approximate a power relationship and 1965 condition values are again ten times 1979 condition values.

An estimate of annual load for each of 105 years was made by substituting the monthly flow volumes derived synthetically (section 3 Part A) into the monthly power relationship to obtain monthly sediment loads and then summing the monthly loads over twelve months. Because 1965 and 1979 are representative of extreme climatic conditions, an average relationship (figure 5) was used in a particular year if the rainfall for that year was within one standard deviation of the mean for the 105 year rainfall record. Otherwise the 1965 (drought condition) or 1979 (wet condition) relationship was used as appropriate.

The results from this method of classifying catchment condition are shown in Table 3, along with the results from the assumptions that the catchment is in one of the two extremes (either drought or wet conditions) for the entire 105 years.
MONTHLY SUSPENDED LOAD FLOW RELATIONSHIPS  Figure 5
Table 3

ANNUAL SUSPENDED SEDIMENT LOAD (105 years) TONNES

<table>
<thead>
<tr>
<th></th>
<th>Average</th>
<th>Dry (1965)</th>
<th>Wet (1979)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Conditions</td>
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<td>Conditions</td>
<td>Conditions</td>
</tr>
<tr>
<td>Mean</td>
<td>5.5 x 10^4</td>
<td>1.6 x 10^5</td>
<td>1.6 x 10^4</td>
</tr>
<tr>
<td>Median</td>
<td>3.8 x 10^4</td>
<td>7.7 x 10^3</td>
<td>8.4 x 10^3</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>5.1 x 10^4</td>
<td>2.2 x 10^5</td>
<td>2.2 x 10^4</td>
</tr>
</tbody>
</table>

3.4 Reservoir Sedimentation

3.4.1 Trap Efficiency Model - Brune's Method

A standard trap efficiency analysis based on Brune's curve (Thomas 1977) was carried out using mean annual suspended sediment loads calculated in section 3.3. For a reservoir capacity of 5.1Mm³ and a mean annual water yield of 12.1 Mm³, a reservoir detention time (detention time = reservoir capacity/mean annual water yield) of 0.42 years was calculated. This detention time corresponds to 95% trap efficiency.

Assuming all bed load was trapped in the upstream sediment trap (section 3.5) and that the suspended sediment consisted of 10% clay and 90% silt, estimates of reservoir capacity were made. Table 4 shows percentages of original reservoir capacity remaining after various time periods for each of the three suspended sediment relationships.

Table 4

PERCENTAGE OF ORIGINAL CAPACITY REMAINING AFTER DIFFERENT PERIODS OF OPERATION, ASSUMING 95% TRAP EFFICIENCY.

<table>
<thead>
<tr>
<th>Time</th>
<th>Average</th>
<th>Dry (1965)</th>
<th>Wet (1979)</th>
</tr>
</thead>
<tbody>
<tr>
<td>25 Years</td>
<td>77</td>
<td>37</td>
<td>93</td>
</tr>
<tr>
<td>50 Years</td>
<td>54</td>
<td>*</td>
<td>87</td>
</tr>
<tr>
<td>100 Years</td>
<td>18</td>
<td>*</td>
<td>75</td>
</tr>
</tbody>
</table>
* Reservoir full of silt.
Estimates of the time required to fill the reservoir with silt are 120 years, 40 years and 400 years for average, dry and wet conditions respectively.

Those estimates of useable reservoir volume and time to fill with silt are lower limits. Brune's analysis uses mean annual sediment loads. For the Todd Catchment median values are up to 50% lower than mean values and are a better estimate of the most frequently occurring annual load in the arid environment. Furthermore a detention time of 0.42 years (150 days) is unrealistic in relation to typical lengths of hydrographs (one to three days) and the short length of the dam at top water level (1600m).

For a typical 1 in 10 year hydrograph in which mean daily flow is of the order of 100 m^3/s,
\[
\text{detention time} = \frac{5.1 \text{ Mm}^3}{100 \times 3600 \times 24 \times 10^{-6} \text{ Mm}^3/\text{day}} = 0.59 \text{ days} = 0.002 \text{ year}.
\]

Trap efficiency from Brune's curve approaches 5% for a 0.002 year detention time instead of the 95% obtained for a 0.42 year detention time. Table 5 shows reworked estimates of useable reservoir capacity for a 0.002 year detention time and the median annual sediment load.

Table 5

<table>
<thead>
<tr>
<th>Time</th>
<th>Suspended Sediment Relationship</th>
</tr>
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<tbody>
<tr>
<td></td>
<td>Average Conditions</td>
</tr>
<tr>
<td>25 Years</td>
<td>99</td>
</tr>
<tr>
<td>50 Years</td>
<td>98</td>
</tr>
<tr>
<td>100 Years</td>
<td>97</td>
</tr>
</tbody>
</table>
Although the percentages in table 5 are not completely correct in that the 5% trap efficiency is taken from Brune’s curve which was derived empirically using annual yields; they are possibly more indicative of sediment-reservoir behaviour for intermittent rivers than the values in table 4. This is especially the case for higher return period flows which contribute a large proportion of the sediment load.

3.4.2 Trap Efficiency Model - Camp's Method.

Camp's method (Chan 1975) is frequently used in the design of sedimentation retention ponds. The method calculates trap efficiencies in terms of a relation between particle settling velocity and the forward velocity in the pond. As such, outflow characteristics are accounted for explicitly rather than implicitly as in the retention time concept used in Brune's method. According to Camp's method, the trap efficiency decreases as the basin outflow increases. Table 6 shows trap efficiencies for different particle sizes and peak flows. Trap efficiency \( E \) is defined as:

\[
E = \frac{W}{Q} \frac{A}{Q_0}
\]

where \( W \) = settling velocity for a given particle size, (Table 7) and \( Q / A \) is the overflow velocity of the basin, which is dependent on the surface area \( A \).

Table 6

<table>
<thead>
<tr>
<th>Return Period (Years)</th>
<th>Peak Outflow Rate (m³/s)</th>
<th>Trap Efficiency (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>170</td>
<td>100 100 100 71 20</td>
</tr>
<tr>
<td>10</td>
<td>250</td>
<td>100 100 95 51 14</td>
</tr>
<tr>
<td>20</td>
<td>350</td>
<td>100 100 91 38 10</td>
</tr>
<tr>
<td>50</td>
<td>500</td>
<td>100 100 85 27 7</td>
</tr>
<tr>
<td>100</td>
<td>620</td>
<td>100 99 80 22 6</td>
</tr>
<tr>
<td>500</td>
<td>950</td>
<td>100 95 55 14 4</td>
</tr>
</tbody>
</table>

NOTE 1: Abbreviations are explained in Table 7.
TABLE 7
SETTLING VELOCITIES OF SEDIMENTS IN WATER AT 16°C
(AMERICAN GEOPHYSICAL UNION CLASSIFICATIONS)

<table>
<thead>
<tr>
<th>SEDIMENT CLASS</th>
<th>ABBREVIATION</th>
<th>DIAMETER (mm)</th>
<th>SETTLING Velocity (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very Coarse Sand</td>
<td>VSCa</td>
<td>2.000-1.000</td>
<td>0.27 - 0.15</td>
</tr>
<tr>
<td>Coarse Sand</td>
<td>CSA</td>
<td>1.000-0.500</td>
<td>0.15 - 0.07</td>
</tr>
<tr>
<td>Medium Sand</td>
<td>MSA</td>
<td>0.500-0.250</td>
<td>0.07 - 0.03</td>
</tr>
<tr>
<td>Fine Sand</td>
<td>PSA</td>
<td>0.250-0.125</td>
<td>0.03 - 0.01</td>
</tr>
<tr>
<td>Very Fine Sand</td>
<td>VPSa</td>
<td>0.125-0.062</td>
<td>10^{-2} - 3.0x10^{-3}</td>
</tr>
<tr>
<td>Coarse Silt</td>
<td>CS</td>
<td>0.062-0.031</td>
<td>3.0x10^{-3} - 8.2x10^{-4}</td>
</tr>
<tr>
<td>Medium Silt</td>
<td>MS</td>
<td>0.031-0.016</td>
<td>8.2x10^{-4} - 2.0x10^{-5}</td>
</tr>
<tr>
<td>Fine Silt</td>
<td>PS</td>
<td>0.016-0.008</td>
<td>2.0x10^{-5} - 5.2x10^{-6}</td>
</tr>
<tr>
<td>Very Fine Silt</td>
<td>VFS</td>
<td>0.008-0.004</td>
<td>5.2x10^{-6} - 1.3x10^{-7}</td>
</tr>
</tbody>
</table>

3.4.3 Discussion

The estimates of capacity depletion in Table 4 and the estimates of time to fill the reservoir with sediment are conservative for two reasons. Firstly, Brune's method makes no allowance for flow through velocities which would be expected to be significant because of the short length of the reservoir. Camp's method shows that trap efficiencies decrease as flow through velocities increase. This result is hinted at in deriving the 5% trap efficiency estimates (Table 5) using Brune's method. Secondly, no differentiation is made between particle sizes in Brune's method. Camp's method shows (Table 6) that only VPSa, CS, and MS fractions are likely to be trapped at a trap efficiency in the order of 95%.

Better estimates of useful reservoir capacities and reservoir life will require the measurement of suspended sediment particle size fractions. These measurements are not available at present.
The 40 year life of the reservoir under high suspended load conditions stresses the necessity for management of catchment land use, especially in drought periods. The present reconstruction of the Stuart Highway through the catchment may also lead to similar high sediment conditions. Care should be taken in construction to preserve natural slopes and vegetation and to prevent erosion of road embankments.

3.5 Sediment Trap

In order to alleviate the effect of bed load on the dam a sediment trap was considered. A suitable site exists immediately upstream of the reservoir where the two arms of the Todd River diverge, (figure A). It is approximately 50m wide, 700m long and 3m high.

Table 8 shows trap efficiencies for different sand particle sizes and peak inflow rates at the sediment trap.

The results show that the sediment trap will retain most of the sand fractions down to the FSa. The trap efficiency for the FSa and VFSa fractions can be improved by increasing the surface area of the sand trap to minimise overflow velocities.

Table 8

<table>
<thead>
<tr>
<th>Return Period (Years)</th>
<th>Peak Inflow Rate (m³/s)</th>
<th>Trap Efficiencies (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>VCSa</td>
<td>CSa</td>
</tr>
<tr>
<td>5</td>
<td>350</td>
<td>100</td>
</tr>
<tr>
<td>10</td>
<td>490</td>
<td>100</td>
</tr>
<tr>
<td>20</td>
<td>650</td>
<td>100</td>
</tr>
<tr>
<td>50</td>
<td>850</td>
<td>100</td>
</tr>
<tr>
<td>100</td>
<td>1000</td>
<td>100</td>
</tr>
<tr>
<td>500</td>
<td>1400</td>
<td>100</td>
</tr>
</tbody>
</table>
Following on from the assumption that all bed load is trapped, a comparison between bed load and demand for sand was made.

In section 3.2 bed load transport for this section of the river was calculated to approximate the 1979 relationship for suspended sediment load at GS006009. From table 3, the median annual load for this case was 8,400 tonnes while the mean annual load was 16,000 tonnes.

Estimates of river sand demand for the Alice Springs region based on withdrawal permits issued by the Water Resources Branch since 1969 show a mean annual demand of 20,000 m$^3$ or 53,000 tonnes. This demand figure is an underestimate since it takes no account of other sources of construction sand and gravel. In addition the withdrawal permit system was not policed and by itself would give a lower bound for demand. The figures have no definite trend and follow variable levels of building and construction activity. Future demand is likely to increase as future developments in Alice Springs will require landfill in many cases.

On comparing the estimates of bed load and sand and gravel demands, it is apparent that on average, demand will exceed trapped bed material. A wall 2m high will provide 70,000 m$^3$ of storage. This capacity is sufficient for bed load from historical floods having return periods of up to 25 years. Most of these floods have bed load volumes in the range 10,000-40,000 m$^3$.

During a wet period when flows are sustained or during very high return period floods, the available sediment storage capacity may be filled and bed load material will spill into the reservoir. This spillover is not expected to affect the reservoir significantly because:
- by the time the sediment trap is filled, a delta will have formed upstream of the sediment trap, thus reducing water slopes and transport capacity for the river section.
- any bed material in the reservoir will occupy a very small percentage of the total reservoir capacity (1.5% for a spillover of 70,000 m$^3$).
- the bed material will deposit in the reservoir immediately downstream of the sediment trap where it is easily excavated.
In the event of an extended wet period with sustained flows or an overall drop in demand for river sand, another sediment trap somewhere further upstream could be constructed.

3.6 Downstream Scour

3.6.1 Moveable Bed Computer Model

A programme based on an alluvial bed transient model (Ponce, Garcia and Simons, 1979) was written to examine the effects of downstream scour. The model solves the equations of water motion, water continuity and sediment continuity using an implicit finite difference scheme. The equations are solved simultaneously at each time step.

The model is one dimensional and calculates changes in bed level only. Sediment transport is characterised by the D50 size and is not routed by size fractions. No account is taken of armouring phenomena. Roughness is calculated using the Darcy-Weisbach friction factor.

Hydrographs are divided into a series of discrete steady state flow rates. At the beginning of each steady state flow, a backwater calculation gives the initial depth of flow at each cross-section. The simultaneous finite difference equations are then solved at time increments within the steady flow time period to give flow depth, energy slope, bed load transport rate and bed elevation at each cross-section.

Problems were encountered in the solution algorithm and also in meeting the convergence and stability criteria. Convergence and stability criteria for the model have been estimated using time periods of years and degradation lengths of miles. To meet similar criteria on the Todd River, increments of the order of 5 minutes and 50m are required; the main constraint being the short time base of typical flood hydrographs.

It was planned to run the model with different return period floods to give an estimate of the location and depth of scour downstream of the spillway. There has been insufficient time to properly define a suitable discretization scheme and to test the model on the river without the dam.

However initial model runs suggest that the majority of scour will be very localised; within 100m of the spillway for a 1 in 10 year flood.
3.6.2. Discussion

Given the localised extent of scour, degradation of the river bed downstream of Wills Terrace is expected to be negligible. Any small bed load deficit will be compensated in large floods by sediment from the Charles River. Causeways at Schwartz Crescent, Wills Terrace, the Golf Club, the Casino and Ross River Road will further limit the amount of degradation that can occur.

Scour will occur at the original Alice Spring at the Old Telegraph Station. However this event occurs only when the secondary spillway is in operation (1 in 6 years on average). If the scour were found to be excessive, a suitable management policy would be to replace scoured material by carting sand from the sediment trap.

Some general degradation will occur in the river section between the primary spillway and the confluence with the Charles River. The effects of scour on this section will not be as environmentally critical as the effects at the Old Telegraph Station and downstream of Wills Terrace. Scour downstream of the spillway is very dependant on exist velocities from the spillway. In any case downstream scour would be decreased by leaving the alluvial bed of the spillway in place. The spillway itself will scour, thereby decreasing the capacity of the flow for scour downstream. In the event that scour has become serious in this river section after a period of reservoir operation, control structures such as weirs or sheet piling walls can be constructed.
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NORTHERN TERRITORY OF AUSTRALIA
DEPARTMENT OF TRANSPORT & WORKS
WATER DIVISION

ALICE SPRINGS
RECREATIONAL DAM
HYDROLOGY REPORT

DECEMBER, 1979.
PROJECT 6
REPORT 12/1979

Prepared by D. Jackson & D. Paige,
Water Division,
Department of Transport & Works,
DARWIN,
October, 1979.
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INTRODUCTION

This report examines the overall hydrological behaviour, including sedimentation of the proposed recreational dam on the Todd River at the Old Telegraph Station. Part A examines dam behaviour, flow reductions downstream of the dam and associated flood attenuation in Alice Springs. Sediment inflow into the dam and downstream scour effects are examined in Part B.

The report was prepared under the supervision of:

- Mr. Andy Macqueen, Senior Engineer, Water Investigations, Alice Springs.
  & Project Leader
- Mr. Ralph Ash, ex Senior Engineer, Hydrographic Section, Darwin
- Mr. Ron Reinhard, Senior Engineer, Analysis and Computation, Darwin

All Australian Height Datum (AHD) levels in this hydrology report are based on topographic maps produced from aerial photography done in September, 1964.
SUMMARY AND CONCLUSION

The usefulness of the impoundment for recreation may be judged by area availability. A sailing triangle of total length 2,400 metres would be available for at least 95 percent of the time. This corresponds to a surface area of approximately 75 hectares. This area availability can be expected with the primary spillway level at 594 metres A.H.D. An annual draft of one million kilolitres could be taken from the storage for watering of parks and recreation grounds without significantly affecting the availability of water for recreation. On average the dam would be filled to spill level about three times every year.

It is felt that this area availability would satisfactorily meet the proposed recreational uses, such as small craft sailing, canoeing, fishing and swimming.

Rapid rises of water levels during floods - as much as 2.5 metres in four hours for the 20 year return period flood - will necessitate careful location of amenities. During the recession limb of flood wave, the water level will drop quickly to the spillway level.

Streamflow through Alice Springs would be reduced. If the dam is constructed it is expected that on the long term average streamflow volumes will be reduced by 20 percent. On a time basis, flow is reduced by approximately 30 percent. About 40 percent of individual stream flows would be fully retained in the impoundment. These figures refer to flows entering the town in the Todd River only and do not allow for continued inputs from the Charles River and town drains.

Flooding in Alice Springs would be reduced. The level of the expected 100 year return period flow would not exceed the level of the present 20 year return period flood. The 50 year return flood would be contained within the banks of the Todd River, except for areas along Leichhardt Terrace and South Terrace.

A conservative estimate of dam life before complete silting is 120 years. To achieve this life it will be necessary to incorporate a sediment trap immediately upstream of the impoundment. This will trap bed load at an estimated rate of 16,000 tonnes per year. By comparison demand for construction sand in the Alice Springs region is currently averaging approximately 53,000 tonnes per year, and hence in most years it should be feasible to completely mine the trap.
Scour downstream of the dam will have a negligible effect on the stream reach through Alice Springs. Scour may occur near the Alice Spring at the Old Telegraph Station during floods of return periods larger than about six years. If scour does occur, its effect could easily be remedied by suitable rehabilitation of the reach downstream of the dam. Most scour will occur between the primary spillway and the Schwartz Crescent causeway.
ADDENDUM

The results in this report are based on storage/area/ elevation curves derived from 1965 photogrammetric contour maps. Recently available (October 1979) photogrammetric maps show slight discrepancies when compared to the 1965 maps. Subject to the verification of the latest maps this report may slightly overestimate flood attenuation and underestimate spill surcharge levels. It is recommended that results of this report be rechecked before any final design is commenced.
Sediment Trap

KEY

Level Exceeded 95% of Time (590 m.)
Top Water Level (594 m.)
Possible Sailing Triangle

SCALE 1:10,000

PLAN OF DAMSITE

FIGURE A
PART A

RESERVOIR BEHAVIOUR

and

FLOOD ROUTING
1. *INTRODUCTION*

Part A discusses in detail the processing of all relevant stream and rainfall data, the synthesis of a historic flow record for the damsite and the generation of long-term flow series by a rainfall/runoff correlation and by a statistical means. The storage behaviour of the dam at various spillway levels is discussed. In addition, the derivation of design floods for various recurrence intervals and the routing of these through various spillway levels and widths is included. The effect of the dam on flooding in Alice Springs is also discussed.

The simulation of dam behaviour was performed using both a Dam Simulation method and Gould's Probability Matrix method. Three types of data were used in the former model. These were the historic flows (18 years), synthetic flows based on a correlation with the historic rainfall data (105 years) and statistically generated flows (105 years).
2. HYDROGRAPHIC DATA

2.1 General

This section summarizes and discusses the rainfall, streamflow, evaporation and impoundment data relevant to the reservoir behaviour studies. A more complete summary of the rainfall and streamflow records in the Alice Springs Region has been prepared by Huq (1979).

2.2 Rainfall

The recording of rainfall records commenced with a European settlement at the Alice Springs Telegraph Station in 1873. The first full water year of records - September to August - was 1873/74. In 1931, the recording site transferred to the site of the original town Post Office and is at present at the Post Office site in Hartley St. Monthly records are available from 1873/74 to the present - 105 years in all.

In addition, there are several daily read raingauges in the Todd River Catchment - see Huq (1979). There are also pluviograph records at Bond Springs in the Todd River Catchment, at the Water Investigations Office in Alice Springs immediately below G.S. 006009 Catchment and at Mt. Lloyd in the Charles River Catchment (Figure 1).

Based on the 105 years of rainfall records, the mean rainfall per water year is 281mm with a standard deviation of 155mm and a lag one serial correlation of 0.14. Rice and Cutler (1967) and Macqueen, Tongia and Verhoeven (1976) have both analysed the rainfall records available at the times of their reports. Attention is drawn to several points. Table 1 shows the water year totals since streamflow records commenced at Gauging Station GS006009, Todd River at Wills Terrace. 1964/65 is the second driest water year on record. The driest year, 1901/02, which had only 31mm was preceded by a year of 245mm and followed by a year of 356mm.
LOCATION OF RECORDING STATIONS AND DAMSITE

ALICE SPRINGS

Mt Lloyd

Pluviograph

GS 006 026

Damsite

WATER INVESTIGATIONS
UNIT PLUVIOGRAPH

FIGURE 1
<table>
<thead>
<tr>
<th>Water Year</th>
<th>Annual Rainfall (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1952/53</td>
<td>353</td>
</tr>
<tr>
<td>53/54</td>
<td>267</td>
</tr>
<tr>
<td>54/55</td>
<td>360</td>
</tr>
<tr>
<td>55/56</td>
<td>310</td>
</tr>
<tr>
<td>56/57</td>
<td>215</td>
</tr>
<tr>
<td>57/58</td>
<td>269</td>
</tr>
<tr>
<td>58/59</td>
<td>158</td>
</tr>
<tr>
<td>59/60</td>
<td>161</td>
</tr>
<tr>
<td>60/61</td>
<td>173</td>
</tr>
<tr>
<td>61/62</td>
<td>189</td>
</tr>
<tr>
<td>62/63</td>
<td>153</td>
</tr>
<tr>
<td>63/64</td>
<td>99</td>
</tr>
<tr>
<td>64/65</td>
<td>95</td>
</tr>
<tr>
<td>65/66</td>
<td>331</td>
</tr>
<tr>
<td>66/67</td>
<td>271</td>
</tr>
<tr>
<td>67/68</td>
<td>370</td>
</tr>
<tr>
<td>68/69</td>
<td>317</td>
</tr>
<tr>
<td>69/70</td>
<td>106</td>
</tr>
<tr>
<td>70/71</td>
<td>194</td>
</tr>
<tr>
<td>71/72</td>
<td>313</td>
</tr>
<tr>
<td>72/73</td>
<td>306</td>
</tr>
<tr>
<td>73/74</td>
<td>926</td>
</tr>
<tr>
<td>74/75</td>
<td>509</td>
</tr>
<tr>
<td>75/76</td>
<td>849</td>
</tr>
<tr>
<td>76/77</td>
<td>357</td>
</tr>
<tr>
<td>77/78</td>
<td>457</td>
</tr>
</tbody>
</table>
However the 1964/65 year was preceded by 8 years of lower than average rainfalls. The wettest year on record is 1973/74 with 926mm. The five year period from 1973/74 is the wettest period on record, the rainfall in each of these years being greater than the mean plus one standard deviation.

2.3 Streamflows

2.3.1 General

The location of the four current gauging stations closest to the damsite are shown in Figure 1. Table 2 shows the catchment areas of the stations and of the damsite. An additional gauging station operated at the damsite from 1967 to 1972, but was abandoned due to excessive silting of the recorder well. There are insufficient records from the station for any form of station analysis.

Table 2

<table>
<thead>
<tr>
<th>Station No.</th>
<th>Stream</th>
<th>Station Name</th>
<th>Catchment Area(km²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>G.S. 006046</td>
<td>Todd River</td>
<td>Wigley Gorge</td>
<td>360</td>
</tr>
<tr>
<td>G.S. 006009</td>
<td>Todd River</td>
<td>Wills Terrace</td>
<td>452</td>
</tr>
<tr>
<td>G.S. 006126</td>
<td>Todd River</td>
<td>Heavitree Gap</td>
<td>508</td>
</tr>
<tr>
<td>G.S. 006047</td>
<td>Charles River</td>
<td>Big Dipper</td>
<td>42</td>
</tr>
<tr>
<td>G.S. 006010</td>
<td>Todd River</td>
<td>Damsite</td>
<td>400</td>
</tr>
</tbody>
</table>

2.3.2 G.S. 006046

Records commence in July 1958 and continue to the present. For this period there are 42 months of incomplete records. There have been 86 stream gaugings done. The largest discharge gauged was 27.9 m³/s on 10/5/68 at a gauge height of 1.646m. On 15/3/77 the highest gauge height of 4.8 metres was recorded. Cease to flow gauge height is 0.301 metres. Control is a concrete weir.
2.3.3 G.S. 006009

This station commenced operation at Wills Terrace causeway in February 1953. In April 1970 it was moved 200 metres upstream to the present site, retaining the same gauge datum. During the period of operation there have been 175 stream gaugings done. Before April 1970 the cease to flow gauge height was 0.838 metres. The largest discharge gauged was 233 m$^3$/s on 5/2/67 at a gauge height of 2.170 metres. After April 1970 the largest discharge gauged was 450 m$^3$/s on 5/3/72 at gauge height of 3.115 metres. The cease to flow gauge height is presently 1.2 metres. The highest recorded gauge heights before and after April 1970 are 2.719 and 3.225 metres respectively.

There are 69 months of incomplete records. Low stage control is the sandy river bed while at medium and high stages the control is Wills Terrace causeway.

2.3.4 G.S. 006126

This section commenced operation in August 1959. There are 55 months with incomplete records. Control at the station is the Ross Highway causeway and the stable stream banks. 32 gaugings have been done with the highest discharge gauged being 145 m$^3$/s on 23/2/67 at a gauge height of 1.219 metres. The highest gauge height recorded is 1.950 metres. Cease to flow gauge height is 0.0m. It should be noted that gaugings at this station have encountered recurring problems such as shifting bed levels and debris interference. Most of the higher ratings are thought to have underestimated discharges.

2.3.5 G.S. 006047

Records commence in 1958 and since then there have been 54 months of incomplete records. 97 gaugings have been done, with the largest discharge being gauged at 52 m$^3$/s on 5/3/72 at a gauge height of 2.347 metres. Highest recorded gauge height is 3.164 metres on 17/4/61. Cease to flow gauge height is 0.456 metres.
2.3.6 Discussion

Figure 2 shows an example of monthly flows at the three stations on the Todd River. Flow volumes at G.S. 006009 are often larger than at G.S. 006046. This would be expected for high return period floods because G.S. 006009 commands a larger catchment. However, the Wills Terrace Station often has smaller volumes than the Wigley Gorge Station. This can be partly attributed to large losses in the Todd River at low flows. Similarly at G.S. 006126, flow volumes at this station are not always larger than at stations further upstream.

2.4 Evaporation

The Bureau of Meteorology has operated a class A pan at the present Alice Springs Airport 15 km south of the township since 1955. The Water Resources Branch has operated three class A pans at other potential Recreational Dam sites. Although the first of these was installed in September 1973, there are in total only forty months of evaporation records available.

It can be expected that the damsite to the north of Alice Springs and the MacDonnell Ranges would have slightly lower class A pan evaporation than that measured by the Bureau of Meteorology at the present airport. This is because the Range, and the hilly region north of the Range would protect the site from most dry winds. The three pans operated by the Department are in similar topography to the damsite.

A correlation factor of 0.90 has been applied to the evaporation measured at the Alice Springs Airport to transform it to the damsite. This factor was arrived at after a comparison of the evaporation data collected by the Department and from the Alice Springs Airport. The expected class A monthly evaporation rates at the damsite are included in Table 4.

Garrett and Hoy (1978) include monthly pan coefficients under various climatic conditions for lakes of various depths and sizes. The coefficients at Rifle Creek Reservoir in Queensland have been used as a basis for calculating the coefficients at Alice Springs. Table 3 shows some general information on the Rifle Creek Reservoir and the Alice Springs Dam.
FLOW COMPARISONS ON TODD RIVER

KEY
- - - Wigley Gorge (GS 006046)
. . . . Wills Terrace (GS 006009)
- - - Heavitree Gap (GS 006126)
- - - Charles River (GS 006047)

Period of Incomplete Records
Wills Terrace
Wigley Gorge
Heavitree Gap
Charles River

1974/75 1975/76 1976/77 1977/78

Monthly Flow Volume (M^3)
### Table 3
**COMPARISON OF RESERVOIRS**

<table>
<thead>
<tr>
<th>Location</th>
<th>Lat. (°S)</th>
<th>Long. (°E)</th>
<th>Approx. elev. (m) A.H.D.</th>
<th>Area (ha)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rifle Creek</td>
<td>20° 57'</td>
<td>139° 35'</td>
<td>420</td>
<td>190</td>
</tr>
<tr>
<td>Alice Springs</td>
<td>23° 43'</td>
<td>133° 50'</td>
<td>590</td>
<td>100</td>
</tr>
</tbody>
</table>

Figure 1g of Garrett’s and Hoy’s paper gives monthly pan coefficients for the Rifle Creek Reservoir for lake depths of 5 metres and 20 metres. The average of the pan coefficients for these two depths has been used. Monthly evaporation for the Alice Springs dam is shown in Table 4.

### Table 4
**ESTIMATED RESERVOIR EVAPORATION**

<table>
<thead>
<tr>
<th>Month</th>
<th>Class A Pan evaporation (mm)</th>
<th>Class A Pan Coefficients</th>
<th>Pan Coefficients</th>
<th>Pan evaporation (mm)</th>
<th>Reservoir evaporation (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>January</td>
<td>372</td>
<td>.80</td>
<td></td>
<td></td>
<td>298</td>
</tr>
<tr>
<td>February</td>
<td>328</td>
<td>.76</td>
<td></td>
<td></td>
<td>249</td>
</tr>
<tr>
<td>March</td>
<td>285</td>
<td>.71</td>
<td></td>
<td></td>
<td>202</td>
</tr>
<tr>
<td>April</td>
<td>209</td>
<td>.72</td>
<td></td>
<td></td>
<td>150</td>
</tr>
<tr>
<td>May</td>
<td>142</td>
<td>.72</td>
<td></td>
<td></td>
<td>102</td>
</tr>
<tr>
<td>June</td>
<td>104</td>
<td>.59</td>
<td></td>
<td></td>
<td>61</td>
</tr>
<tr>
<td>July</td>
<td>107</td>
<td>.51</td>
<td></td>
<td></td>
<td>55</td>
</tr>
<tr>
<td>August</td>
<td>145</td>
<td>.52</td>
<td></td>
<td></td>
<td>75</td>
</tr>
<tr>
<td>September</td>
<td>212</td>
<td>.51</td>
<td></td>
<td></td>
<td>108</td>
</tr>
<tr>
<td>October</td>
<td>274</td>
<td>.58</td>
<td></td>
<td></td>
<td>159</td>
</tr>
<tr>
<td>November</td>
<td>313</td>
<td>.56</td>
<td></td>
<td></td>
<td>175</td>
</tr>
<tr>
<td>December</td>
<td>358</td>
<td>.67</td>
<td></td>
<td></td>
<td>240</td>
</tr>
<tr>
<td>Year Total</td>
<td>2849</td>
<td>0.64 (av)</td>
<td></td>
<td></td>
<td>1923</td>
</tr>
</tbody>
</table>

#### 2.5 Impoundment Data

The relationships between stage, surface area and dam capacity are shown in Figure 3. These relationships are based on 1:2400 photogrammetric contour maps. Forbes (1965) determined similar curves for the damsites from 1:2400 stadia survey contour maps. The relationships shown in Figure 3 and those derived by Forbes are within 5% of each other.
FIGURE 3

CAPACITY - AREA CURVES OF DAM AT OLD TELEGRAPH STATION
3. ANALYSIS OF DATA

3.1 General

This section discusses the analysis of the hydrographic data to determine flood frequency, unit graphs and maximum probable floods, and also the generation of stream flow sequences.

3.2 Flood Frequency

At G.S. 006046 peak annual gauge heights are known for years 1960/61 to 1977/78 except for 1975/76. The peak water year floods and gauge heights are included in Table 6. The peak flood in 1975/76 was in early February, but there are no records from 6/2/76 to 16/2/76.

Three approaches were used to analyse records from G.S. 006046. These were:

- **Case (a)**: Ignore the 1975/76 water year and analyse 17 years of records.
- **Case (b)**: Assume the peak discharge at G.S. 006046 in 1975/76 is of the same order of magnitude as that at G.S. 006009 in the same year. Estimate a peak discharge for 1975/76 by using a rainfall - peak discharge correlation. This correlation used maximum daily rainfall from both Alice Springs Post Office, and Bond Springs. Ranked peak discharges at Wigley Gorge and ranked maximum daily rainfalls were plotted at points of equal probability. Close examination indicated that storms in the Todd River catchment are often isolated thunderstorms. This is thought to indicate that a rainfall event of a particular return period need not necessarily be associated with a stream flow of the same return period. Correlation between ranked rainfall and runoff at G.S. 006046 resulted in a correlation of 0.92 for Alice Springs rainfall data and 0.88 for Bond Springs rainfall data. Using rainfall data from 1975/76 the peak discharge at G.S. 006046 in that year was estimated to be 650 m$^3$/s. This is considered to be excessively high especially when compared with the peak discharge at G.S. 006009 and G.S. 006126 (see Table 6). Correlation with the actual storm also overestimated the peak discharge.
A log-Pearson Type III distribution was fitted to the data for cases (a), (b) and (c). Low flows were treated in accordance with the recommendation in "Australian Rainfall and Runoff" section 9.2.9. The distribution from case (b) was selected as being a suitable representation of the flood frequency although the difference between (a) and (b) is small and both distributions well within the confidence limits of each other. The adopted flood frequency curve for G.S. 006046 is shown in Figure 4.

At G.S. 006009, maximum gauge heights for all the water years 1952/53 to 1977/78 are known and it has been possible to reliably estimate the peak annual discharges. (see Table 6). A log-Pearson Type III distribution was fitted to the data using the technique outlined in "Australian Rainfall and Runoff" section 9.2.9 (see Figure 5).

At G.S. 006126 only the water years 1965/66, 1974/75 and 1975/76 have complete records though the maximum gauge heights are known for all years since 1960/61 except 1976/77 (see Table 6). The very small peak discharge for 1976/77 was estimated from G.S. 006046 and G.S. 006009. Again a log-Pearson Type III distribution was fitted to the data (Figure 6) using the same techniques as above. The small number of records and the large number of low discharges result in the confidence limits for the G.S. 006126 annual flood frequency distribution being very large, especially when compared with the confidence limits obtained for G.S. 006046 and G.S. 006009. A comparison of confidence limits for the 100 year return period flood is shown in Table 5.

Table 5
CONFIDENCE LIMITS FOR 100 YEAR RETURN PERIOD FLOOD

<table>
<thead>
<tr>
<th></th>
<th>G.S. 006046</th>
<th>G.S. 006009</th>
<th>G.S. 006126</th>
</tr>
</thead>
<tbody>
<tr>
<td>5% Limit (m³/s)</td>
<td>2300</td>
<td>1800</td>
<td>3800</td>
</tr>
<tr>
<td>Value (m³/s)</td>
<td>1040</td>
<td>890</td>
<td>1120</td>
</tr>
<tr>
<td>95% Limit (m³/s)</td>
<td>720</td>
<td>620</td>
<td>660</td>
</tr>
</tbody>
</table>

Table 5 also demonstrates that even at a fairly high return period the adopted peak discharge at G.S. 006009 is smaller than that at G.S. 006046 and G.S. 006126.
FIGURE 4
FIGURE 5

FLOOD FREQUENCY CURVE FOR GS 006009, TODD RIVER AT WILLS TERRACE
### Table 6

#### PEAK ANNUAL DISCHARGES ON TODD RIVER

<table>
<thead>
<tr>
<th>Water Year</th>
<th>G.S. 006046</th>
<th>G.S. 006009</th>
<th>G.S. 006126</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>G.H.m Q(m³/s) Date</td>
<td>G.H.m Q(m³/s) Date</td>
<td>G.H.m Q(m³/s) Date</td>
</tr>
<tr>
<td>1952/53</td>
<td>2.377 305 5/2</td>
<td>1.981 130 18/4</td>
<td>1.220 272 17/4</td>
</tr>
<tr>
<td>53/54</td>
<td>1.920 113 25/1</td>
<td>1.981 130 14/1</td>
<td>0.860 120 14/1</td>
</tr>
<tr>
<td>54/55</td>
<td>1.524 36 24/10</td>
<td>2.591 448 15/5</td>
<td>1.134 229 15/5</td>
</tr>
<tr>
<td>55/56</td>
<td>2.621 471 24/2</td>
<td>1.402 25 20/10</td>
<td>0.472 30 26/10</td>
</tr>
<tr>
<td>56/57</td>
<td>1.400 25 19/6</td>
<td>1.402 25 20/10</td>
<td>0.08 103 14/10</td>
</tr>
<tr>
<td>57/58</td>
<td>1.830 85 9/12</td>
<td>2.591 448 15/5</td>
<td>1.134 229 15/5</td>
</tr>
<tr>
<td>58/59</td>
<td>1.768 71 22/11</td>
<td>2.719 563 22/1</td>
<td>1.030 184 21/1</td>
</tr>
<tr>
<td>59/60</td>
<td>1.981 130 29/1</td>
<td>3.210 195 14/10</td>
<td>1.220 272 17/4</td>
</tr>
<tr>
<td>60/61</td>
<td>3.231 197 18/4</td>
<td>1.981 130 18/4</td>
<td>1.220 272 17/4</td>
</tr>
<tr>
<td>61/62</td>
<td>3.444 235 14/1</td>
<td>1.981 130 14/1</td>
<td>0.860 120 14/1</td>
</tr>
<tr>
<td>62/63</td>
<td>4.374 420 15/5</td>
<td>2.591 448 15/5</td>
<td>1.134 229 15/5</td>
</tr>
<tr>
<td>63/64</td>
<td>1.420 17 20/10</td>
<td>1.402 25 20/10</td>
<td>0.472 30 26/10</td>
</tr>
<tr>
<td>64/65</td>
<td>3.210 195 14/10</td>
<td>1.951 120 14/10</td>
<td>0.08 103 14/10</td>
</tr>
<tr>
<td>65/66</td>
<td>4.791 517 22/1</td>
<td>2.719 563 22/1</td>
<td>1.030 184 21/1</td>
</tr>
<tr>
<td>66/67</td>
<td>3.658 273 5/2</td>
<td>2.210 212 5/2</td>
<td>1.189 256 5/2</td>
</tr>
<tr>
<td>67/68</td>
<td>2.774 131 2/3</td>
<td>1.981 130 2/3</td>
<td>0.077 7 2/3</td>
</tr>
<tr>
<td>68/69</td>
<td>1.463 19 14/3</td>
<td>1.286 16 14/3</td>
<td>0.117 18 3/12</td>
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<tr>
<td>69/70</td>
<td>0.320 0 all yr</td>
<td>1.200 0 all yr</td>
<td>0.000 0 all yr</td>
</tr>
<tr>
<td>70/71</td>
<td>1.433 18 28/2</td>
<td>1.550 5 7/12</td>
<td>1.220 5 7/12</td>
</tr>
<tr>
<td>71/72</td>
<td>4.399 423 5/5</td>
<td>3.140 371 5/3</td>
<td>1.950 751 5/3</td>
</tr>
<tr>
<td>72/73</td>
<td>1.625 27 14/6</td>
<td>1.900 34 15/6</td>
<td>0.257 11 15/6</td>
</tr>
<tr>
<td>73/74</td>
<td>4.655 483 30/12</td>
<td>3.255 422 25/11</td>
<td>1.850 742 23/1</td>
</tr>
<tr>
<td>74/75</td>
<td>4.860 535 4/9</td>
<td>1.770 20 13/11</td>
<td>0.286 9 2/10</td>
</tr>
<tr>
<td>75/76</td>
<td>See Text  8/2</td>
<td>2.880 270 8/2</td>
<td>1.340 341 8/2</td>
</tr>
<tr>
<td>76/77</td>
<td>4.800 520 15/3</td>
<td>3.050 336 15/3</td>
<td>N.R. N.R.</td>
</tr>
<tr>
<td>77/78</td>
<td>2.330 80 21/11</td>
<td>2.080 59 21/11</td>
<td>0.545 42 21/11</td>
</tr>
</tbody>
</table>

**Notes:**
1. N.R. - no record
2. G.S.006009 changed location in April 1970.
It is not known whether this is a characteristic of the catchment or if it is due to the sequence of flood events. It can be partly attributed to high transmission losses into alluvial aquifers below and adjacent to the stream. It should also be noted that, as indicated in section 2.3, there have been no high stage ratings at G.S. 006046 and G.S. 006126 and that in addition to the confidence limits shown in Figures 4 and 6 the accuracy of the rating curve extension must be considered. A comparison of the flood frequency relationships at G.S. 006009 predicted by different authors is shown in Table 7. The relationship predicted in this report agrees satisfactorily with the findings of these other studies.

Table 7

<table>
<thead>
<tr>
<th>Return Period (years)</th>
<th>This Report</th>
<th>Hua</th>
<th>C.M.S. P.</th>
<th>Reinhard</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>410</td>
<td>450</td>
<td>480</td>
<td>410</td>
</tr>
<tr>
<td>20</td>
<td>540</td>
<td>590</td>
<td>600</td>
<td>600</td>
</tr>
<tr>
<td>50</td>
<td>740</td>
<td>790</td>
<td>840</td>
<td>900</td>
</tr>
<tr>
<td>100</td>
<td>900</td>
<td>950</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Figure 7 shows the flood frequency distribution for both G.S. 006046 and G.S. 006009 plotted on a log-log scale. An additional distribution has been shown in Figure 7 to represent the flood frequency at the damsite.

The estimates of peak discharge for selected return periods at the damsite are shown in Table 8.
Table 8

ESTIMATED FLOOD FREQUENCY AT DAMSITE

<table>
<thead>
<tr>
<th>Return Period (years)</th>
<th>Peak Annual Discharge (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>350</td>
</tr>
<tr>
<td>10</td>
<td>490</td>
</tr>
<tr>
<td>20</td>
<td>650</td>
</tr>
<tr>
<td>50</td>
<td>850</td>
</tr>
<tr>
<td>100</td>
<td>1000</td>
</tr>
<tr>
<td>500</td>
<td>1400</td>
</tr>
</tbody>
</table>

3.3 Unitgraphs

Using the method of unitgraph derivation developed by Ash (1973) two one hour unitgraphs were derived for the Todd River at G.S. 006046 and at G.S. 006009. They are shown in Figure 8.

Many of the recorded hydrographs were found to be unacceptable for use in the Ash Method because they were either multiple peaked or the peak discharge was too small to be regarded as a flood. The method only allows sharply peaked hydrographs to be considered. At G.S. 006046 the unitgraph was derived from floods on the following dates: 17/4/61, 15/5/63, 13/10/64, 21/1/66, 6/1/68, 2/3/68, 30/12/73, 14/3/77, and 25/11/77. At G.S. 006009 the floods occurring on the following dates were used: 23/2/56, 15/5/63, 25/11/73, 13/12/75, 15/3/77 and 21/11/77.

The unitgraph for G.S. 006009 has a longer recession limb and a steeper rising limb. This is partly due to the input from the Charles River. Response time of the Charles River is quicker than the response time of the Todd River and this influences the time of the rising limb at G.S. 006009.

An average unitgraph (Figure 8) has been developed for input of hydrographs to the reservoir for examination of flood routing effects of the dam.

3.4 Maximum Probable Flood

Estimates of the maximum probable flood have been derived from three sources.
FIGURE 7
ONE HOUR UNTAGRAPH AT GS006046, GS006009 AND DAM SITE

FIGURE 8
Ash (1978) includes a set of curves which give peak discharge for various return periods, and maximum known floods for Australia and the World, as a function of catchment area. Use of these curves requires estimation of the average stream slope. Ash has suggested a rainfall factor to be applied to these curves to adjust the discharges for arid regions. Using the curves and the rainfall factor, the 100 year return period flood at the damsite is estimated to have a peak discharge of 1000 m³/s and the maximum probable flood is estimated to have a discharge of 4000 m³/s.

The Bureau of Meteorology (1979) has estimated the maximum probable precipitation (MPP) for two storm durations in the Alice Springs Region. Using the U.S. Thunder Storm Model, the MPP for a 3 hour duration storm is estimated to be 170mm. By transposing recorded storms the MPP for a 12 hour duration is estimated to be 360mm. For the 12 hour duration storm, three likely temporal patterns have been supplied. The most critical temporal pattern gives a peak discharge of 4,620 m³/s. The initial loss rate and the continuing loss rate were both set to zero, in order to get the maximum runoff.

A third estimate of the maximum probable flood relied on information from a storm at Rumbalara Siding in February 1976. Rumbalara Siding is approximately 170km south of Alice Springs on the Central Australian Railway. On the 9th of February, following three days of heavy rainfall, a 9am reading of 415mm was recorded. Because the temporal pattern for the storm is not available the 24 hour temporal pattern for arid regions in Figure 3.7 of "Australian Rainfall and Runoff" was used. The one hour unitgraph for the damsite (Figure 8) was converted to a four hour unitgraph using the principle of superposition with the volume of excess rainfall accordingly adjusted. Using the four hour unitgraph and six theoretical excess rainfall periods the probable maximum flood was estimated to be 5,300 m³/s. Again, the initial and continuing loss rates were set to zero.

This report uses the maximum probable flood derived from the maximum probable precipitation estimates.
3.5 Historic Streamflows.

As was stated in section 2.3.1 there are no usable historic records of flow at the damsite. Records of flow are available from G.S. 006046 upstream and from G.S. 006009 downstream of the damsite. Section 2.2 indicates that the period of streamflow records include two extreme rainfall patterns - a nine year drought and a five year wet period.

Using all four stations: G.S. 006046, G.S. 006009, G.S. 006126 and G.S. 006047 an attempted correlation of runoff volumes with catchment area for each month was found to be ineffective. The additional effect of the Charles River inflow at G.S. 006009 was removed using records at G.S. 006047. All of the graphs showed a great scatter of points. This is attributed to at least two reasons: (a) the non-uniform spatial pattern of storms and (b) large infiltration losses in the river channel. An unsuccessful attempt was made to find a consistent relationship of the relative sizes of floods as they progress down the Todd River.

Being unable to determine a satisfactory area-runoff relationship the historic flows (m$^3$/month) at the damsite were taken as the average of those at Wigley Gorge and Wills Terrace.

In order to do this it proved necessary to complete the records at both stations. This was done starting at September 1980. In many months although the record for the whole month is incomplete this incompleteness is due only to 2 or 3 days of no records. By examining the flow during those days at the other stations it was possible to estimate the flow which probably would have occurred on those 2 or 3 days. For the rest of the month the flow volume is known and hence the monthly flow volumes is estimated. In cases where records for the whole month, or a substantial part of the month are not available or are unreliable, or where both G.S. 006046 and G.S. 006009 do not have records, use was made of records at G.S. 006126 and G.S. 006047. In many cases, by examining the records at Charles River and Heavitree Gap and rainfall records it was concluded that no flow occurred for the period of no records. However, in the other cases it was obvious that substantial flows did occur and for these times it was necessary to estimate the flow volumes. A correlation between monthly rainfall at both Alice Springs and Bond Springs with flow volume/month at both G.S. 006046, and G.S. 006009 was developed. Using the monthly rainfall, an estimate of the volume of runoff for the month of no records is obtained.
Rather than fit a line of best fit through the points, as was done for the synthetic flows (see section 3.6) the points plotted were joined directly. Figures 9 and 10 show the points used. Bond Springs rainfall points are not shown in these figures.

Thus 'Complete Records' were available at Wigley Gorge and Wills Terrace and the average of these flow volumes for each month was used as the historic flow record at the Damsite.

3.6 Synthetic Streamflow Generation.

A rainfall runoff correlation was found. Using the 105 years of monthly rainfall records at the Telegraph Station and Alice Springs Post Office, this correlation was used to generate 105 years of monthly synthetic stream flow.

Using the Kolmogorov - Smirnov test, Spiegel (1965), it may be concluded that the rainfall records at the Telegraph Station (1874 - 1931) are from the same population as the rainfall records at the Post Office (1932 - 1972). Bond Springs rainfall records are also from the same population. The rainfall records at the Post Office and the Telegraph Station can therefore be used to extend the records.

Monthly rainfalls at Alice Springs Post Office and monthly runoff volumes at G.S. 006046 and G.S. 006009 over corresponding periods were ranked and points of equal probability were plotted.

At first this correlation was attempted on a month by month basis but due to lack of sufficient data and the large number of zeros in the record (40% to 70%) it is impossible to find a definite trend. The method was then done on a seasonal basis. Two "seasons" were chosen: October to March and April to September.

Using these two seasons, separate curves for Wigley Gorge and Wills Terrace were derived. The curves were extrapolated to get higher flow than recorded. This was done by firstly extending the frequency curves of monthly rainfall and monthly flow volumes.
Rainfall and runoff points of equal probability from these extended frequency curves were then plotted on the correlations shown in Figures 9 and 10.

At low flows (see insert Figure 9), the historic points give a more realistic correlation than the ranked points. This is because at low rainfalls, storm distribution is not uniform over the catchment, losses are high and rainfall at Alice Springs is less indicative of flow in the catchment than for the higher rainfalls.

The accuracy of the correlations shown in Figures 9 and 10 was tested using the Kolmogorov - Smirnov test (Spiegel 1965). This test was done on the historic flows at the damsite and the generated synthetic flows. For both seasons they were found to be within the 5% confidence limit.

Using the rainfall-runoff correlation, 105 years of synthetic flows were generated on a monthly basis.

3.7 Statistical Streamflow Generation

Two methods were investigated to generate stochastic streamflow sequences based on streamflow statistics.

One method generates log normal annual flows having zero serial correlation. i.e.

$$\log Q = \log \bar{Q} + t \times \sigma$$

where $t$ is a random variate

$\log \bar{Q}$ is the mean of the log of the annual flows

$\sigma$ is the standard deviation.

These statistics are based on the 105 years of synthetic flows. An arbitrary cutoff point is applied to the generated annual flows to account for low flow years.

The statistically generated annual flows are ranked and the original annual flows are ranked. Monthly temporal patterns corresponding to the equivalent rank to the original flows are fitted to the statistically generated flows.

The other method assumes correlation between months. It is based on the Thomas and Fiering seasonal model (McMahon & Mein 1978) applied to the 18 years historical data at the damsite. To allow for the large number of zeroes in each monthly sequence, Beard's Artificial Negative Analysis Method, Beard (1973), is used.
RAINFALL/RUNOFF CORRELATION OCTOBER TO MARCH

**Figure 9**

- **Legend**:
  - ○ GS006009
  - ▲ GS006046
  - GS006009, GS006046: Extrapolated from equal probability rainfall and runoff events.
  - ○ Historic rainfall:

**Axes**:
- RANKED RAINFALL (m.m./month) at Alice Springs Post Office
- RANKED RUNOFF (Mm$^3$/month)

**Inset**
-ENSET
RAINFALL / RUNOFF CORRELATION
APRIL TO SEPTEMBER

FIGURE 10

KEY
- GS006009
- GS006046
- GS006009
- GS006046
- Extrapolated from equal probability
- Rainfall and runoff events
- Dam data historic

RANKED RUNOFF (mm/month)
RANKED RAINFALL (mm/month) AT ALICE SPRINGS POST OFFICE
The seasonal model is based on the normal probability distribution but problems were encountered in reducing the large skew values, approximately 3, for monthly sequences for use in the model.

Streamflow sequences based on the first method have been used as data for the dam simulation (Section 4).
4. DAM SIMULATION

4.1 General

The behaviour of the dam was simulated with a computer programme. The programme simulates the behaviour of a dam by calculating levels, volumes and areas of the dam, on a monthly basis. The programme is given:

(a) rainfall,
(b) runoff from the catchment,
(c) evaporation and pan coefficients
(d) draft and
(e) area/volume/elevation data for the impoundment.

The user specifies the starting level.

If the dam overflows, the programme calculates the volume of water discharged by the spillway.

At the end of the simulation the programme prints the following statistics: for each month and the whole year, the number of months the area was between the areas given in the area/volume/elevation curves; and all durations at which the area was below a specified area.

Seepage was neglected in all simulations. Preliminary site investigations suggest that seepage is small. If this is subsequently proved to be false, the programme could be rerun with higher evaporation to allow for such seepage losses.

The programme was run with six constant drafts: zero, 0.5, 1, 2, 3, 4 and 5 mega cubic metres/year. This was done to assess the ability of the storage to supplement Alice Springs Water Supply or to compensate for reduced yield from the town basin. The simulation was run at three primary spillway levels: 591 metres, 592.5 metres and 594 metres A.H.D. For all combinations, the historic and synthetic flows were used to simulate the behaviour.
4.2 Results Using Synthetic Streamflows.

Curves of the availability of area, volume and elevation as simulated with the synthetic flows are shown in figures 11, 12 and 13. Each graph shows the behaviour at different spillway levels and give the result found for the months October to March. The curves describing behaviour for April to September are similar.

For low drafts of 0.0, 0.5 and 1 Mm$^3$/year, median values of area, volume and elevation are very close to full dam levels. As the drafts increase median values of area, volume and elevation decrease. For 75% of the time, area, volume and elevations corresponding to the low drafts are always above those corresponding to half the dam's capacity.

With the primary spillway level at 594 metres A.H.D. (Figure 13) area, volume and elevations corresponding to the low drafts are above 80% of the full capacity for 75% of the time.

The flatness of the curves is attributed to the small capacity of the dam - 5.1 Mm$^3$ for a spillway level of 594 metres in comparison to the stream flows i.e. mean annual flow volume for the 105 years is 12.1 Mm$^3$, median annual flow is 7.9 Mm$^3$.

4.3 Results Using Historic Streamflows.

The result of the dam simulation using the historic flows were found to be very similar to those of the synthetic flows. These results are summarized in figures 11, 12 and 13.

4.4 Results Using Statistically Generated Streamflows.

These flows were used in the dam simulation for two combinations of drafts and spillway levels and verified the results found for the synthetic and historic flows.
SIMULATED DAM BEHAVIOUR. SPILLWAY AT 591 METRES.

**Figure 11**

Graph showing the relationship between volume (Mm³) and elevation (m) for different discharge rates (in m³/year). The percentage of time the elevation, area, or volume is exceeded is plotted on the x-axis (% of time). The graph illustrates how the discharge rate affects the exceedance of these parameters.
SIMULATED DAM BEHAVIOUR. SPILLWAY AT 592.5 METRES.
SIMULATED DAM BEHAVIOUR. SPILLWAY AT 594 METRES
4.5 Gould Analysis

As an independent check on the dam simulation model, a programme was written for Goulds Probability Matrix Method, as modified by McMahon and Mein (1978) to account for monthly rather than annual failures.

The method assumes an annual serial correlation of zero, and is independent of the historical sequencing of flows and initial reservoir conditions. The programme allows for monthly variations in draft and evaporation. Dead storage can be defined as a fraction of reservoir capacity. Probability of failure can be calculated either at steady state or as a time dependent function of the starting conditions.

Calculated steady state probabilities (Table 9) fitted the duration curves for percent of time exceeded (Figure 11,12 and 13) very closely. Transient failure probabilities assuming the reservoir is initially empty are also shown in Table 9.

Table 9

PROBABILITY OF AREA BEING GREATER THAN INDICATED SIZE WITH PRIMARY SPILLWAY AT 594M AND A DRAFT OF 1.0 MM³/YEAR.

<table>
<thead>
<tr>
<th>Probability in Percentage.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Years after Construction 100ha</td>
</tr>
<tr>
<td>1</td>
</tr>
<tr>
<td>2</td>
</tr>
<tr>
<td>3</td>
</tr>
<tr>
<td>Steady State</td>
</tr>
</tbody>
</table>
4.6 Other Results

Of all the simulations performed (approximately forty five) the maximum duration of a period when the draft could not be met was 20 months. This was for the historic flow case with a draft of 5Mm\(^3\)/year and at the lowest spillway level of 591 metres. This occurred from April 1969 to November 1970. During this time there was no surface runoff. With a draft of 1Mm\(^3\)/year or less, at spillway levels of 592.5 metres and 594 metres the draft was able to be met for the whole period of simulation.

For the hypothetical case of a complete drought, i.e. no rain or no runoff, at a draft 1 Mm\(^3\)/year and at a spillway level of 594 metres, it takes 27 months for the dam to empty. This is an extreme case and is not expected to occur.

Table 10 shows the total number of failures found from the dam simulation. In this context failure is defined as a period of time when the draft could not be met.

**Table 10**

**TOTAL NUMBER OF FAILURES**

<table>
<thead>
<tr>
<th>Draft Mm(^3)/yr.</th>
<th>Spillway Level (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>594</td>
</tr>
<tr>
<td>5</td>
<td>48 (22%)</td>
</tr>
<tr>
<td>4</td>
<td>35 (16%)</td>
</tr>
<tr>
<td>3</td>
<td>22 (10%)</td>
</tr>
<tr>
<td>2</td>
<td>6 (3%)</td>
</tr>
<tr>
<td>1</td>
<td>0 (0%)</td>
</tr>
<tr>
<td>0.5</td>
<td>0 (0%)</td>
</tr>
<tr>
<td>0</td>
<td>0 (0%)</td>
</tr>
</tbody>
</table>

**Note:** 1. Probability of failure in brackets.
The simulation indicates that if the dam is constructed on the long term average, stream flow volumes will be reduced by 20%. Table 11 shows a comparison of flow in the Todd River before and after the dam, on a daily basis.

Table 11
COMPARISON OF FLOW IN THE TODD RIVER BEFORE AND AFTER DAM (1960/62 - 1977/78) (in days)

<table>
<thead>
<tr>
<th>Drafts</th>
<th>Spillway Level</th>
<th>594m</th>
<th>592.5m</th>
<th>591m</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.</td>
<td>844 (55%)</td>
<td>862 (56%)</td>
<td>874 (57%)</td>
<td></td>
</tr>
<tr>
<td>4.</td>
<td>895 (58%)</td>
<td>911 (59%)</td>
<td>949 (62%)</td>
<td></td>
</tr>
<tr>
<td>3.</td>
<td>937 (61%)</td>
<td>991 (65%)</td>
<td>991 (65%)</td>
<td></td>
</tr>
<tr>
<td>2.</td>
<td>991 (65%)</td>
<td>1004 (65%)</td>
<td>1004 (65%)</td>
<td></td>
</tr>
<tr>
<td>1.</td>
<td>1009 (66%)</td>
<td>1019 (66%)</td>
<td>1041 (68%)</td>
<td></td>
</tr>
<tr>
<td>0.5</td>
<td>1019 (66%)</td>
<td>1058 (69%)</td>
<td>1101 (72%)</td>
<td></td>
</tr>
<tr>
<td>0.0</td>
<td>1103 (72%)</td>
<td>1134 (74%)</td>
<td>1219 (79%)</td>
<td></td>
</tr>
</tbody>
</table>

Note: 1. Percentage of original number of days in brackets.
5. RESERVOIR ROUTING

5.1 General

From preliminary investigations of costing and operation, a draft of 1Mm³/year and a spillway level of 594 metres was chosen. Flood routing then proceeded with the following assumed spillway arrangement:

The main or primary spillway is approximately 150 metres downstream of the Old Telegraph Station and is at 594 metres A.H.D.. The secondary spillway is upstream of the Old Telegraph Station. This spillway has a crest level of 596 metres A.H.D.. Another two metres higher than the secondary spillway is the top of the dam wall. The spillways are arranged so that no water will spill over the Western Saddle and thus onto the site of the Old Telegraph Station. Two cases of spillways were investigated and these are shown in Table 12.

<table>
<thead>
<tr>
<th>Table 12</th>
<th>SPILLWAY ARRANGEMENTS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Case (a)</td>
</tr>
<tr>
<td>Elevation (m A.H.D.)</td>
<td>Total Usable Spillway Width (m)</td>
</tr>
<tr>
<td>594</td>
<td>50</td>
</tr>
<tr>
<td>596</td>
<td>130</td>
</tr>
<tr>
<td>598</td>
<td>200</td>
</tr>
</tbody>
</table>

5.2 Results

Floods with return periods of 5, 10, 20, 50, 100 and 500 years, and the maximum probable flood were routed through the dam using Puls' Method (see Wilson (1974)). For both cases of spillway arrangement (i.e. Case (a) and Case (b), Table 12) alternative routes were done with the dam starting full and at a level of 592.7 metres. 592.7 metres is the elevation which was exceeded 75% of the time for a draft of 1Mm³/year and a spillway level of 594 metres.
Tables 13 and 14 give the results of the routes for cases (a) and (b) respectively, starting conditions being full. Although the attenuation is larger when the starting level is 592.7 metres, it is not significantly larger - for example - case (a), 100 year return period flood is discharged as 560 m$^3$/s when the starting level is 592.7, compared with 618 m$^3$/s in Table 13.

Table 13
INFLows AND OUTFlows OF DAM Case (a)

<table>
<thead>
<tr>
<th>Floods (yrs) Return Periods</th>
<th>Starting Elevation</th>
<th>Inflow(m$^3$/s)</th>
<th>Discharge (m$^3$/s)</th>
<th>Max. Level (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>594</td>
<td>350</td>
<td>168</td>
<td>595.7</td>
</tr>
<tr>
<td>10</td>
<td>594</td>
<td>490</td>
<td>251</td>
<td>596.2</td>
</tr>
<tr>
<td>20</td>
<td>594</td>
<td>650</td>
<td>348</td>
<td>596.5</td>
</tr>
<tr>
<td>50</td>
<td>594</td>
<td>850</td>
<td>502</td>
<td>597.0</td>
</tr>
<tr>
<td>100</td>
<td>594</td>
<td>1000</td>
<td>618</td>
<td>597.3</td>
</tr>
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<td>500</td>
<td>594</td>
<td>1400</td>
<td>951</td>
<td>598.0</td>
</tr>
<tr>
<td>M.P.F.</td>
<td>594</td>
<td>4620</td>
<td>4274</td>
<td>602.0</td>
</tr>
</tbody>
</table>

Table 15 is an example output of Pulse′ Routing calculations. All discharges are in m$^3$/s. It serves to illustrate the rapid increase in level water as a flood passes. In this case it rises 2.1 metres in 4 hours. For the 100 year return period flood, it rises 3.3 metres in 4 hours.
Table 14

INFLOWS AND OUTFLOWS OF DAM Case (b)

<table>
<thead>
<tr>
<th>Floods (yrs)</th>
<th>Starting Elevation - 594m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Return Periods</td>
<td>Inflow(m³/s)</td>
</tr>
<tr>
<td>5</td>
<td>350</td>
</tr>
<tr>
<td>10</td>
<td>490</td>
</tr>
<tr>
<td>20</td>
<td>650</td>
</tr>
<tr>
<td>50</td>
<td>850</td>
</tr>
<tr>
<td>100</td>
<td>1000</td>
</tr>
<tr>
<td>500</td>
<td>1400</td>
</tr>
<tr>
<td>M.P.F.</td>
<td>4620</td>
</tr>
</tbody>
</table>

For case (a) the secondary spillway is expected to be used, on average, once every 8½ years. For case (b) it is once every 5½ years. The spillway at 598 metres (dam wall) has return periods of use of 500 and 380 years for cases (a) and case (b) respectively. These return periods are based on the dam starting full. They are slightly higher if the dam starts at elevation 592.7m.
Table 15
EXAMPLE OF FLOOD ROUTE

ALICE SPRINGS DAM. 10 Year Return Period Flood

DETAILS OF CREST:

<table>
<thead>
<tr>
<th>Height (m)</th>
<th>Width (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>594.00</td>
<td>50</td>
</tr>
<tr>
<td>596.00</td>
<td>130</td>
</tr>
<tr>
<td>598.00</td>
<td>200</td>
</tr>
</tbody>
</table>

STARTING ELEVATION = 594 m.

<table>
<thead>
<tr>
<th>Hour Rout Info</th>
<th>I (avg)</th>
<th>Disch</th>
<th>S-D/2</th>
<th>S+D/2</th>
<th>Level</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>1419</td>
<td>1425</td>
</tr>
<tr>
<td>1</td>
<td>2</td>
<td>15</td>
<td>38</td>
<td>1470</td>
<td>1500</td>
</tr>
<tr>
<td>2</td>
<td>3</td>
<td>152</td>
<td>16</td>
<td>1493</td>
<td>1814</td>
</tr>
<tr>
<td>3</td>
<td>4</td>
<td>490</td>
<td>87</td>
<td>1728</td>
<td>2175</td>
</tr>
<tr>
<td>4</td>
<td>5</td>
<td>405</td>
<td>208</td>
<td>1967</td>
<td>2294</td>
</tr>
<tr>
<td>5</td>
<td>6</td>
<td>248</td>
<td>448</td>
<td>1728</td>
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<tr>
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</tr>
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<td>8</td>
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<td>124</td>
<td>1846</td>
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<td>15</td>
<td>16</td>
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<td>17</td>
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<td>0</td>
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</tr>
<tr>
<td>17</td>
<td>18</td>
<td>0</td>
<td>0</td>
<td>16</td>
<td>1500</td>
</tr>
<tr>
<td>18</td>
<td>19</td>
<td>0</td>
<td>0</td>
<td>12</td>
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</tr>
<tr>
<td>19</td>
<td>20</td>
<td>0</td>
<td>0</td>
<td>9</td>
<td>1479</td>
</tr>
<tr>
<td>20</td>
<td>21</td>
<td>0</td>
<td>0</td>
<td>9</td>
<td>1469</td>
</tr>
<tr>
<td>21</td>
<td>22</td>
<td>0</td>
<td>0</td>
<td>7</td>
<td>1462</td>
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<tr>
<td>22</td>
<td>23</td>
<td>0</td>
<td>0</td>
<td>7</td>
<td>1456</td>
</tr>
<tr>
<td>23</td>
<td>24</td>
<td>0</td>
<td>0</td>
<td>4</td>
<td>1451</td>
</tr>
<tr>
<td>24</td>
<td>25</td>
<td>0</td>
<td>0</td>
<td>4</td>
<td>1447</td>
</tr>
<tr>
<td>25</td>
<td>26</td>
<td>0</td>
<td>0</td>
<td>2</td>
<td>1445</td>
</tr>
<tr>
<td>26</td>
<td>27</td>
<td>0</td>
<td>0</td>
<td>2</td>
<td>1442</td>
</tr>
</tbody>
</table>
6. FLOODING IN ALICE SPRINGS

Table 16 shows the expected peak flood discharge at Wills Terrace and Heavitree Gap if the dam is constructed. These values are conservative as they assume that the discharges are reduced by the amount of attenuation achieved when the dam is full at the start of the flood. This was done because of the uncertainty of the effect of the Charles River input at Wills Terrace. At floods of high return periods, normally both the Charles River and the Todd River flow, so if the flood at the damsite is reduced by 350 m³/s (50 year return period flood), then it is reasonable to assume that it is reduced by this same amount at Wills Terrace and Heavitree Gap. The peak discharges in Table 16 have been derived using the results of flood routing for spillway arrangement case (a).

Table 16

<table>
<thead>
<tr>
<th>Return Period (yrs)</th>
<th>G.S. 006009</th>
<th>G.S. 006126</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>170 (m³/s)</td>
<td>250 (m³/s)</td>
</tr>
<tr>
<td>20</td>
<td>240 (m³/s)</td>
<td>360 (m³/s)</td>
</tr>
<tr>
<td>50</td>
<td>380 (m³/s)</td>
<td>560 (m³/s)</td>
</tr>
<tr>
<td>100</td>
<td>520 (m³/s)</td>
<td>740 (m³/s)</td>
</tr>
</tbody>
</table>

By comparing Table 16 with figures 5 and 6, it is seen that the peak discharges are reduced considerably, especially at the lower return periods. The new 100 year return period flood at G.S. 006009 is of the same order of magnitude as the present 20 year return period flood. According to maps included in flood studies of Alice Springs conducted by Cameron McNamara and Partners (1978) and Huq (1979) the present 20 year return period flood will inundate approximately 30 ha of developed land to a shallow depth. The present 50 and 100 year return period floods inundate areas in excess of 200 ha.
PART B

SEDIMENTATION STUDY
1. INTRODUCTION

The damming of the Todd River at the Old Telegraph Station will result in two sediment related effects. Upstream of the dam, sediment will deposit in the reservoir, gradually decreasing the useable reservoir area and volume. In contrast the river bed downstream of the dam will scour.

Both effects are due to the perturbation of a steady state system by the construction of a dam. Sediment transport is a complex function of hydraulic and fluid variables, channel geometry and sediment properties. Conceptually the river is considered to be in an equilibrium state before dam construction, with all the interacting variables being compatible. Following dam construction, one or more of these variable will change and the equilibrium between variables is upset. To achieve the new steady state in which the system is compatible with modified conditions, changes will occur in other variables such as bed elevation and channel width.

The aim of this sediment study was to look at these upstream and downstream effects and specifically to examine: 1) the effect of silting on the useful life of the reservoir.
2) the effect of the dam on downstream channel morphology.
2. DATA

2.1 Introduction

Field data is very sparse. Suspended load measurements were made on two floods in March 1965 and on three floods in early 1979. All measurements were taken at G.S. 006009. One measurement of bed load was made in 1979.

Permanent ranges have been established between G.S. 006126 and the dam site for long term monitoring of large scale changes in channel morphology. The locations of the cross-sections are shown in figure 1.

Throughout the sediment section of this report a distinction is made between suspended sediment and bed load. Suspended sediment consists of fine particles, typically having a particle diameter less than 0.06 mm. In the Todd catchment most fine particles would be classified as silt with a small proportion (10%) of clays. Bed load comprises larger particles which move along the river bed by saltation or rolling. An individual bed particle alternates between being at rest and in motion on a random basis. At any observation, the overall impression is one of net particle transport although individual particles may have changed from a moving to a rest state or vice-versa during the observation.

A distinction should also be made between sediment transport capacity and the actual sediment load. Transport capacity is a measure of the sediment that could be transported for a given river section and flow rate and assumes that the sediment source can adequately supply sediment at the rate of transport. In fact, the actual sediment load can be less than the transport capacity because of an inadequate supply of sediment. The rate of suspended sediment transport is very dependent on the availability of fine particles within the catchment. On an alluvial river bed where there exists a large source of bed load material, sediment transport capacity is a reasonable estimate of actual bed load. Total sediment load comprises both suspended load and bed load.
2.2 Suspended Sediment

The arid nature of the Todd Catchment in conjunction with grazing as its land use suggests that small diameter suspended sediment transport rates will be high.

The measurements of suspended sediment in 1965 and 1979 were both made during climatically extreme periods. The 1965 water year recorded the second lowest rainfall on record and was preceded by the third lowest rainfall in 1965. In addition, 1965 was the last year of a continuous nine year period of below average rainfalls which was the worst drought on record. In contrast the 1979 water year occurs at the end of a seven year period of above average rainfalls and is the wettest period on record.

Suspended sediment mass flow rates from samples taken at Wills Terrace are plotted against discharge at G.S. 006009 in figure 2. All measurements were made on the falling stage of the hydrograph.

No analysis of suspended sediment particle size was made.

2.3 Bed Load

One bed load measurement of 650g/s at a discharge of 10m³/s was made at G.S. 006009 on 4/3/79. This represents 30% of suspended load or 25% of total load.

Estimates of the bed load transport rate at Wills Terrace were made using the Meyer-Peter/Müller formula (Simons and Senturk, 1977). These estimates are sensitive to stream slope values. Stream slopes can be estimated either by assuming a constant Mannings 'n' value and using Mannings equation for uniform steady flow or by approximating the 'bed slope' to the 'stream slope'. An 'n' value of 0.035 was used in calculating bed load transport at Wills Terrace. Bed load transport estimates made for a cross-section downstream of Wills Terrace using alternative formulae (Cameron McNamara 1978) and the estimates for bed load transport at Wills Terrace are shown in figure 2.
FIGURE 2

SEDIMENT TRANSPORT FLOW RELATIONSHIP
Bed load transport in the channel section immediately upstream of the reservoir was estimated using the Meyer - Peter/Muller formula with the bed slope approximation and is also shown in figure 2. Because of the steeper channel slope in this section, the bed load transport rate is much higher than the rate at Wills Terrace.

Assuming that transport rate is at transport capacity, the high rate upstream of the damsite suggests that in order to maintain continuity of sediment, bed load material is being stored between the two sections. In fact the Todd River undergoes a distinct change in channel morphology at the damsite. The river bed changes from a shallow sandy, rocky, low storage channel to a deep sand, high storage channel.

Bed samples were taken at third points for cross section ranges upstream from Wills Terrace. Figures 3 and 4 show the range of particle sizes of samples taken at the surface and 1m depth respectively. Samples taken at 1m depth are coarser than surface samples i.e.:-

- **1.0m depth**  $D_{50} = 1.1mm$  $D_{90} = 7mm$
- **Bed surface**  $D_{50} = 0.75mm$  $D_{90} = 3mm$

There is no evidence of particle size sorting between cross-sections downstream from the dam.
SIZE DISTRIBUTION OF BEDLOAD SAMPLES AT BED SURFACE
CROSS SECTIONS 6 TO 13

FIGURE 3
SIZE DISTRIBUTION OF BEDLOAD SAMPLES AT 1 METRE DEPTH
CROSS SECTIONS 6 TO 13

FIGURE 4
3 ANALYSIS

3.1 Introduction

A power relationship between sediment flow rate and stream discharge was used to estimate monthly and annual sediment loads. A trap efficiency model was used to estimate the life of the dam. The feasibility of a sediment trap immediately upstream of the reservoir to trap bed load was examined.

A computer model for transient scour and deposition was used to examine the effects of downstream scour. Several problems were encountered in using this model and its value was in giving an overall impression of where scour would occur for various downstream controls rather than giving absolute depths of scour.

3.2 Transport Relationships

Both floods in 1965 included flow at G.S. 006047. Table 1 shows estimated flow volumes at gauging stations.

Table 1

<table>
<thead>
<tr>
<th>Gauging Station</th>
<th>7/3/65</th>
<th>18-19/3/65</th>
</tr>
</thead>
<tbody>
<tr>
<td>G.S. 006009</td>
<td>0.83</td>
<td>0.69</td>
</tr>
<tr>
<td>G.S. 006046</td>
<td>0.45</td>
<td>0.51</td>
</tr>
<tr>
<td>G.S. 006047</td>
<td>0.13</td>
<td>0.34</td>
</tr>
</tbody>
</table>

It is known flow occurred for the 1979 storms at each of the gauging stations G.S. 006009, G.S. 006046, G.S. 006 047, but flow volumes are not yet available.
Rainfall data for the 1965 storms and the first two storms of 1979 are shown in Table 2.

Table 2

RAINFALL (1965 AND 1979 STORMS) mm

<table>
<thead>
<tr>
<th>RAINFALL STATION</th>
<th>7/3/65</th>
<th>18-19/3/65</th>
<th>22/1/79</th>
<th>3/2/79</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water Resources Branch</td>
<td>8.1</td>
<td>11.9</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bond Springs Turnoff</td>
<td>NR</td>
<td>NR</td>
<td>44</td>
<td>7.5</td>
</tr>
<tr>
<td>Bond Springs (pluviograph)</td>
<td>0</td>
<td>0.5</td>
<td>44</td>
<td>3.1</td>
</tr>
<tr>
<td>Big Dipper (G.S. 006047)</td>
<td>21.1</td>
<td>45.7</td>
<td>26</td>
<td>7</td>
</tr>
<tr>
<td>Mt. Lloyd (pluviograph)</td>
<td>16.3</td>
<td>15</td>
<td>44.5</td>
<td>2</td>
</tr>
<tr>
<td>Yuendumu Road</td>
<td>7.6</td>
<td>23.1</td>
<td>25</td>
<td>2</td>
</tr>
<tr>
<td>Station Creek</td>
<td>NR</td>
<td>NR</td>
<td>40</td>
<td>40</td>
</tr>
<tr>
<td>12 Mile</td>
<td>NR</td>
<td>NR</td>
<td>39</td>
<td>38</td>
</tr>
<tr>
<td>Flynn's Grave</td>
<td>NR</td>
<td>NR</td>
<td>51</td>
<td>2</td>
</tr>
<tr>
<td>Undoolya Road</td>
<td>NR</td>
<td>NR</td>
<td>31.6</td>
<td>2.5</td>
</tr>
</tbody>
</table>

Notes. N.R. No Record.

From Table 2, both storms in 1965 appear to have been centred on the Charles and Collyer Creek Catchments. The storm which occurred on the 22/1/79 was generally widespread whereas that which occurred on the 3/2/79 was concentrated in the upper part of the Todd Catchment north of Bond Springs.

Comparing the areal distribution of rainfall for the January 1979 storm with the distribution of the 1965 storms it can be seen that the contributing area for the 1965 flows (i.e. approximately west of the Stuart Highway) forms part of the contributing area for the 1979 flow.
This suggests that differences in suspended sediment transport rates between the higher 1965 rates and the lower 1979 rates may be due to differences in catchment condition, rather than to differences in contributing area.

No correction factor has been applied to transfer suspended sediment data at Wills Terrace to the damsite for two reasons. Firstly there is insufficient data to separate the contribution of the Charles Catchment to suspended sediment load at Wills Terrace. Secondly a large proportion of fine sediment is probably originating in the northern section of the Todd Catchment where even in 1979 vegetation is sparse.

The suspended sediment data for 1979 shows a fall in transport rate for a given discharge for the three floods. This effect may be due to:
1) influence of different contributing areas
2) a general decrease in sediment supply after successive floods.
3) a decrease in sediment supply available for transport on the falling stage of the hydrograph as the peak flow increases.

Without further measurements these effects cannot be isolated.

If there is a general decrease in transport rate for successive floods, any analysis which assumes independent flood events will overestimate suspended sediment load.

For analysis, each flow event was assumed to be independent and power relationships were fitted to the 1965 (drought conditions) and the 1979 (wet conditions) data respectively. Transport rates under 1965 conditions are approximately ten times those under 1979 conditions for equivalent flows. Bed load transport upstream of the impoundment was assumed to follow the relationship for 1979 suspended sediment data. The power relationships are plotted in figure 2.
3.3 DMASS Computer Programme

The DMASS (daily, monthly annual sediment summary) computer programme integrates instantaneous values of suspended sediment transport rate over time. The programme is based on a standard DMAS (daily, monthly annual summary) for flow volumes (Lee 1979) that has been modified to include a power relationship between instantaneous sediment transport rate and instantaneous flow. This relationship is of the form \( C = AQ^b \) where \( C \) is the instantaneous sediment transport rate in g/s, \( Q \) is flow rate in m\(^3\)/s and \( A \) and \( b \) are constants.

The DMASS and DMAS programmes were run using historical flows (1953 - 1978) at G.S. 006009 to develop relationships between suspended sediment load and flow volume on a monthly basis. Two runs were made: one assuming 1965 conditions applied throughout the historical record and the other assuming 1979 conditions applied. Figure 5 shows the derived points and the fitted relationship for 1979 conditions and the fitted relationship for 1965 conditions. Because there is generally only one flow per month in the historical record, the monthly relationships between sediment load and stream flow reflect the relationships for individual hydrographs obtained from field data. The points in the monthly relationships approximate a power relationship and 1965 condition values are again ten times 1979 condition values.

An estimate of annual load for each of 105 years was made by substituting the monthly flow volumes derived synthetically (section 3 Part A) into the monthly power relationship to obtain monthly sediment loads and then summing the monthly loads over twelve months. Because 1965 and 1979 are representative of extreme climatic conditions, an average relationship (figure 5) was used in a particular year if the rainfall for that year was within one standard deviation of the mean for the 105 year rainfall record. Otherwise the 1965 (drought condition) or 1979 (wet condition) relationship was used as appropriate.

The results from this method of classifying catchment condition are shown in Table 3, along with the results from the assumptions that the catchment is in one of the two extremes (either drought or wet conditions) for the entire 105 years.
MONTHLY SUSPENDED LOAD FLOW RELATIONSHIPS

FIGURE 5
Table 3

ANNUAL SUSPENDED SEDIMENT LOAD (105 years) TONNES

<table>
<thead>
<tr>
<th></th>
<th>Average Conditions</th>
<th>Dry (1965) Conditions</th>
<th>Wet (1979) Conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean</td>
<td>$5.5 \times 10^4$</td>
<td>$1.6 \times 10^5$</td>
<td>$1.6 \times 10^4$</td>
</tr>
<tr>
<td>Median</td>
<td>$3.8 \times 10^4$</td>
<td>$7.7 \times 10^4$</td>
<td>$8.4 \times 10^3$</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>$5.1 \times 10^4$</td>
<td>$2.2 \times 10^5$</td>
<td>$2.2 \times 10^4$</td>
</tr>
</tbody>
</table>

3.4 Reservoir Sedimentation

3.4.1 Trap Efficiency Model - Brune's Method

A standard trap efficiency analysis based on Brune's curve (Thomas 1977) was carried out using mean annual suspended sediment loads calculated in section 3.3. For a reservoir capacity of 5.1Mm$^3$ and a mean annual water yield of 12.1 Mm$^3$, a reservoir detention time (detention time = reservoir capacity/mean annual water yield) of 0.42 years was calculated. This detention time corresponds to 95% trap efficiency.

Assuming all bed load was trapped in the upstream sediment trap (section 3.5) and that the suspended sediment consisted of 10% clay and 90% silt, estimates of reservoir capacity were made. Table 4 shows percentages of original reservoir capacity remaining after various time periods for each of the three suspended sediment relationships.

Table 4

PERCENTAGE OF ORIGINAL CAPACITY REMAINING AFTER DIFFERENT PERIODS OF OPERATION, ASSUMING 95% TRAP EFFICIENCY.

<table>
<thead>
<tr>
<th>Time (Years)</th>
<th>Suspended</th>
<th>Sediment Relationship</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Average Conditions</td>
<td>Dry (1965) Conditions</td>
</tr>
<tr>
<td>25 Years</td>
<td>77</td>
<td>37</td>
</tr>
<tr>
<td>50 Years</td>
<td>54</td>
<td>*</td>
</tr>
<tr>
<td>100 Years</td>
<td>18</td>
<td>*</td>
</tr>
</tbody>
</table>

* Reservoir full of silt.
Estimates of the time required to fill the reservoir with silt are 120 years, 40 years and 400 years for average, dry and wet conditions respectively.

Those estimates of useable reservoir volume and time to fill with silt are lower limits. Brune's analysis uses mean annual sediment loads. For the Todd Catchment median values are up to 50% lower than mean values and are a better estimate of the most frequently occurring annual load in the arid environment. Furthermore a detention time of 0.42 years (150 days) is unrealistic in relation to typical lengths of hydrographs (one to three days) and the short length of the dam at top water level (1600m).

For a typical 1 in 10 year hydrograph in which mean daily flow is of the order of 100 m$^3$/s:

\[
\text{detention time} = \frac{5.1 \text{ Mm}^3}{100 \times 3600 \times 24 \times 10^{-6} \text{ Mm}^3/\text{day}} = 0.59 \text{ days} = 0.002 \text{ year}.
\]

Trap efficiency from Brune's curve approaches 5% for a 0.002 year detention time instead of the 95% obtained for a 0.42 year detention time. Table 5 shows reworked estimates of useable reservoir capacity for a 0.002 year detention time and the median annual sediment load.

**Table 5**

<table>
<thead>
<tr>
<th>Time</th>
<th>Suspended Conditions</th>
<th>Sediment Relationship</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Average Conditions</td>
<td>Dry (1965) Conditions</td>
</tr>
<tr>
<td>25 Years</td>
<td>99</td>
<td>96</td>
</tr>
<tr>
<td>50 Years</td>
<td>98</td>
<td>97</td>
</tr>
<tr>
<td>100 Years</td>
<td>97</td>
<td>94</td>
</tr>
</tbody>
</table>
Although the percentages in table 5 are not completely correct in that the 5% trap efficiency is taken from Brune's curve which was derived empirically using annual yields; they are possibly more indicative of sediment-reservoir behaviour for intermittent rivers than the values in table 4. This is especially the case for higher return period flows which contribute a large proportion of the sediment load.

3.4.2 Trap Efficiency Model - Camp's Method.

Camp's method (Chan 1975) is frequently used in the design of sedimentation retention ponds. The method calculates trap efficiencies in terms of a relation between particle settling velocity and the forward velocity in the pond. As such, outflow characteristics are accounted for explicitly rather than implicitly as in the retention time concept used in Brune's method. According to Camp's method, the trap efficiency decreases as the basin outflow increases. Table 6 shows trap efficiencies for different particle sizes and peak flows. Trap efficiency \( E \) is defined as:

\[
E = \frac{W}{Q/A}
\]

where \( W \) = settling velocity for a given particle size, (Table 7) and \( Q/A \) is the overflow velocity of the basin, which is dependent on the surface area \( A \).

Table 6

<table>
<thead>
<tr>
<th>Return Period (Years)</th>
<th>Peak Outflow Rate (m³/s)</th>
<th>Trap Efficiency (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>VFSa CS MS FS VFS</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>170 100 100 100 71 20</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>250 100 100 95 51 14</td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>350 100 100 91 38 10</td>
<td></td>
</tr>
<tr>
<td>50</td>
<td>500 100 100 85 27 7</td>
<td></td>
</tr>
<tr>
<td>100</td>
<td>620 100 99 80 22 6</td>
<td></td>
</tr>
<tr>
<td>500</td>
<td>950 100 95 55 14 4</td>
<td></td>
</tr>
</tbody>
</table>

NOTE 1: Abbreviations are explained in Table 7.
TABLE 7
SETTLING VELOCITIES OF SEDIMENTS IN WATER AT 16°C
(AMERICAN GEOPHYSICAL UNION CLASSIFICATIONS)

<table>
<thead>
<tr>
<th>SEDIMENT CLASS</th>
<th>ABBREVIATION</th>
<th>DIAMETER (mm)</th>
<th>SETTLING VELOCITY (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very Coarse Sand</td>
<td>VSCa</td>
<td>2.000-1.000</td>
<td>0.27 - 0.15</td>
</tr>
<tr>
<td>Coarse Sand</td>
<td>CSa</td>
<td>1.000-0.500</td>
<td>0.15 - 0.07</td>
</tr>
<tr>
<td>Medium Sand</td>
<td>Msa</td>
<td>0.500-0.250</td>
<td>0.07 - 0.03</td>
</tr>
<tr>
<td>Fine Sand</td>
<td>Fsa</td>
<td>0.250-0.125</td>
<td>0.03 - 0.01</td>
</tr>
<tr>
<td>Very Fine Sand</td>
<td>VFSa</td>
<td>0.125-0.062</td>
<td>$10^{-2}$ - 3.0x10^{-3}</td>
</tr>
<tr>
<td>Coarse Silt</td>
<td>CS</td>
<td>0.062-0.031</td>
<td>3.0x10^{-4} - 8.2x10^{-5}</td>
</tr>
<tr>
<td>Medium Silt</td>
<td>MS</td>
<td>0.031-0.016</td>
<td>8.2x10^{-5} - 2.0x10^{-5}</td>
</tr>
<tr>
<td>Fine Silt</td>
<td>FS</td>
<td>0.016-0.008</td>
<td>2.0x10^{-5} - 5.2x10^{-5}</td>
</tr>
<tr>
<td>Very Fine Silt</td>
<td>VFS</td>
<td>0.008-0.004</td>
<td>5.2x10^{-5} - 1.3x10^{-5}</td>
</tr>
</tbody>
</table>

3.4.3 Discussion

The estimates of capacity depletion in Table 4 and the estimates of time to fill the reservoir with sediment are conservative for two reasons. Firstly, Brune's method makes no allowance for flow through velocities which would be expected to be significant because of the short length of the reservoir. Camp's method shows that trap efficiencies decrease as flow through velocities increase. This result is hinted at in deriving the 5% trap efficiency estimates (Table 5) using Brune's method. Secondly, no differentiation is made between particle sizes in Brune's method. Camp's method shows (Table 6) that only VFSa, CS, and MS fractions are likely to be trapped at a trap efficiency in the order of 95%.

Better estimates of useful reservoir capacities and reservoir life will require the measurement of suspended sediment particle size fractions. These measurements are not available at present.
The 40 year life of the reservoir under high suspended load conditions stresses the necessity for management of catchment land use, especially in drought periods. The present reconstruction of the Stuart Highway through the catchment may also lead to similar high sediment conditions. Care should be taken in construction to preserve natural slopes and vegetation and to prevent erosion of road embankments.

3.5 Sediment Trap

In order to alleviate the effect of bed load on the dam a sediment trap was considered. A suitable site exists immediately upstream of the reservoir where the two arms of the Todd River diverge, (figure A). It is approximately 50m wide, 700m long and 3m high.

Table 8 shows trap efficiencies for different sand particle sizes and peak inflow rates at the sediment trap.

The results show that the sediment trap will retain most of the sand fractions down to the FSa. The trap efficiency for the FSa and VFSa fractions can be improved by increasing the surface area of the sand trap to minimise overflow velocities.

Table 8

TRAP EFFICIENCIES FOR DIFFERENT PARTICLE SIZES AND PEAK INFLOW RATES - SEDIMENT TRAP (PERCENT).

<table>
<thead>
<tr>
<th>Return Period (Years)</th>
<th>Peak Inflow Rate (m³/s)</th>
<th>Trap Efficiencies %</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>VCSa</td>
<td>CSA</td>
</tr>
<tr>
<td>5</td>
<td>350</td>
<td>100</td>
</tr>
<tr>
<td>10</td>
<td>490</td>
<td>100</td>
</tr>
<tr>
<td>20</td>
<td>650</td>
<td>100</td>
</tr>
<tr>
<td>50</td>
<td>850</td>
<td>100</td>
</tr>
<tr>
<td>100</td>
<td>1000</td>
<td>100</td>
</tr>
<tr>
<td>500</td>
<td>1400</td>
<td>100</td>
</tr>
</tbody>
</table>

Table 8
Following on from the assumption that all bed load is trapped, a comparison between bed load and demand for sand was made.

In section 3.2, bed load transport for this section of the river was calculated to approximate the 1979 relationship for suspended sediment load at GS006009. From table 3, the median annual load for this case was 8,400 tonnes while the mean annual load was 16,000 tonnes.

Estimates of river sand demand for the Alice Springs region based on withdrawal permits issued by the Water Resources Branch since 1969 show a mean annual demand of 20,000 m$^3$ or 53,000 tonnes. This demand figure is an underestimate since it takes no account of other sources of construction sand and gravel. In addition, the withdrawal permit system was not policed and by itself would give a lower bound for demand. The figures have no definite trend and follow variable levels of building and construction activity. Future demand is likely to increase as future developments in Alice Springs will require landfill in many cases.

On comparing the estimates of bed load and sand and gravel demands, it is apparent that on average, demand will exceed trapped bed material. A wall 2m high will provide 70,000 m$^3$ of storage. This capacity is sufficient for bed load from historical floods having return periods of up to 25 years. Most of these floods have bed load volumes in the range 10,000–40,000 m$^3$.

During a wet period when flows are sustained or during very high return period floods, the available sediment storage capacity may be filled and bed load material will spill into the reservoir. This spillover is not expected to affect the reservoir significantly because:
- by the time the sediment trap is filled, a delta will have formed upstream of the sediment trap, thus reducing water slopes and transport capacity for the river section.
- any bed material in the reservoir will occupy a very small percentage of the total reservoir capacity (1.5% for a spillover of 70,000 m$^3$).
- the bed material will deposit in the reservoir immediately downstream of the sediment trap where it is easily excavated.
In the event of an extended wet period with sustained flows or an overall drop in demand for river sand, another sediment trap somewhere further upstream could be constructed.

3.6 Downstream Scour

3.6.1 Mobile Bed Computer Model

A programme based on an alluvial bed transient model (Ponce, Garcia and Simons, 1979) was written to examine the effects of downstream scour. The model solves the equations of water motion, water continuity and sediment continuity using an implicit finite difference scheme. The equations are solved simultaneously at each time step.

The model is one dimensional and calculates changes in bed level only. Sediment transport is characterised by the D50 size and is not routed by size fractions. No account is taken of armouring phenomena. Roughness is calculated using the Darcy-Weisbach friction factor.

Hydrographs are divided into a series of discrete steady state flow rates. At the beginning of each steady state flow, a backwater calculation gives the initial depth of flow at each cross-section. The simultaneous finite difference equations are then solved at time increments within the steady flow time period to give flow depth, energy slope, bed load transport rate and bed elevation at each cross-section.

Problems were encountered in the solution algorithm and also in meeting the convergence and stability criteria. Convergence and stability criteria for the model have been estimated using time periods of years and degradation lengths of miles. To meet similar criteria on the Todd River, increments of the order of 5 minutes and 50m are required; the main constraint being the short time base of typical flood hydrographs.

It was planned to run the model with different return period floods to give an estimate of the location and depth of scour downstream of the spillway. There has been insufficient time to properly define a suitable discretization scheme and to test the model on the river without the dam.

However initial model runs suggest that the majority of scour will be very localised; within 100m of the spillway for a 1 in 10 year flood.
3.6.2. Discussion

Given the localised extent of scour, degradation of the river bed downstream of Wills Terrace is expected to be negligible. Any small bed load deficit will be compensated in large floods by sediment from the Charles River. Causeways at Schwartz Crescent, Wills Terrace, the Golf Club, the Casino and Ross River Road will further limit the amount of degradation that can occur.

Scour will occur at the original Alice Spring at the Old Telegraph Station. However this event occurs only when the secondary spillway is in operation (1 in 6 years on average). If the scour were found to be excessive, a suitable management policy would be to replace scoured material by carting sand from the sediment trap.

Some general degradation will occur in the river section between the primary spillway and the confluence with the Charles River. The effects of scour on this section will not be as environmentally critical as the effects at the Old Telegraph Station and downstream of Wills Terrace. Scour downstream of the spillway is very dependent on exist velocities from the spillway. In any case downstream scour would be decreased by leaving the alluvial bed of the spillway in place. The spillway itself will scour, thereby decreasing the capacity of the flow for scour downstream. In the event that scour has become serious in this river section after a period of reservoir operation, control structures such as weirs or sheet piling walls can be constructed.
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