HYDROLOGICAL AND FLOOD STUDY OF THE SUNRISE AQUACULTURE ESTATE PROPOSAL

F.T.H.Barlow March 2003

1. Introduction

This study was undertaken to investigate the water management aspects of the proposed "Sunrise Aquaculture Estate" located in the Bynoe Harbour area of the Northern Territory. The basic components of the proposal, as far as water management is concerned, are shown on Fig.1. They comprise a dam on Broads Creek (also known as Wheatley Creek), a pump and pipeline to divert water from the dam to an offstream storage, the offstream storage, and the growing pond. The setup shown is the initial development. Additional ponds and Offstream Storages may be added in the future.

The intended operation of the proposal is to initially fill the growing pond with sea water, then to use only fresh water for make up. The aim is to have no discharge from the growing ponds or if this were not possible, to restrict such discharges to periods of extreme rainfall only, when significant dilution would occur.

The steps undertaken in the study were as follows:

1. Model the growing pond under incident rainfall to determine both the make up requirements and the probability of overflow.
2. Model the dam/offstream storage combination, using daily rainfall and the make up requirements from (1) to determine the required sizes for the storages.
3. Determine the 1 in 100 year average recurrence interval flood flow for Broads Creek and to use this to plot the Q100 floodline downstream of the dam and to size the required dam spillway.

2. Modeling of the Growing Pond

A simple monthly water balance model was used to assess pond behaviour. Inputs were monthly rainfall and evaporation. Water level, salinity, spill volumes and required fresh water makeup volumes were output for varying initial pond depths and overflow levels.

For the purpose of the model, the pond dimensions were assumed to be 500 x 200 x 3.5, with an inside bank slope of 1 on 1.5. The height-area-volume values are shown in Table 1.

Table 1
Stage 1 Growing Pond – assumed Depth/Area/Volume

Depth m	Area ha	Volume ML
0	10	0
0.5	10.1	50
1	10.2	101
1.5	10.3	152
2	10.4	204
2.5	10.5	257
3	10.6	310
3.5	10.7	363

As the only long term rainfall record available is for Darwin this was used in the study. Comparisons were however made with short term rainfall records for the Dundee Beach/Bynoe Harbour area. Available records were:

Table 2
Rainfall Record Bynoe Harbour Area

Station No	Station Name	Location Relative to Development	Period of Wet Season Record
014015	Darwin Airport		Jan 1941 – Jan 2003
014277	Dum In Mirrie AWS	22km NW	1990, 1993 - 2002
014241	Dum In Mirrie	22km NW	1996,1999 – 2002
014013	Geriatric Park	10km WSW	1998 - 2002
014010	Dundee Beach School	18km W	1999 - 2002

Stations 014013 and 014010 were considered to be the most representative of the four local stations being closer and situated on the mainland. Monthly wet season (December to March inclusive) rainfalls at these stations were compared to Darwin records for the same period. These comparisons are shown in Table 3.

Table 3
Rainfall Comparisons Darwin and Bynoe Stations

Season	Station	Dec	Jan	Feb	Mar	Total	Factor
1997-1998	Darwin	588	670	404	474	2136	
	Dundee School	N/A	N/A	N/A	N/A	N/A	
	Geriatric Park	598	727	251	843	2419	1.13
1998-1999	Darwin	598	360	524	375	1857	
	Dundee School	384	599	905	385	2273	1.22
	Geriatric Park	433	701	549	497	2180	1.17
1999-2000	Darwin	250	441	662	368	1721	
	Dundee School	249	476	712	537	1974	1.15
	Geriatric Park	373	443	747	537	2100	1.22
2000-2001	Darwin	206	262	417	257	1142	
	Dundee School	354	240	357	440	1391	1.22
	Geriatric Park	272	516	649	588	2025	1.77
2001-2002	Darwin	182	147	407	247	983	
	Dundee School	178	112	552	196	1038	1.06
	Geriatric Park	244	300	570	152	1266	1.29

As shown the local stations have generally recorded 10 to 20% more rainfall than Darwin.

A further comparison was carried out by running the model using available local records and Darwin rainfalls for the same periods. Darwin rainfall was factored by different amounts to approximate the same overflows as occurred with local rainfalls. On this basis the Darwin rainfalls needed to be factored by 1.1 to approximate the Geriatric Park results and 1.2 to agree with Dundee Beach School results. To be on the conservative side a factor of 1.2 was used with Darwin rainfall to determine probability of overflows. Unfactored rainfalls were used to assess fresh water make up requirements. This again is a conservative assumption. Monthly make-up volumes for 1.

To assess frequency of overflow, the model was operated for a range of initial operating levels and spill levels. The initial level is assumed to be the level of the original seawater fill. Fresh water make up is only added to maintain this level. Table 4 shows the results of these model runs.

Table 4
Pond Overflow Model Results

Op. Level m	Op. Storage ML	Spill Level m	Free board	Av. Level m	Av. Salinity ppt	Min. Salinity ppt	No. of Spills	O'flow Frequency
1.2	122	3	1.8	1.7	25	11	5	1 in 12
1.4	142	3	1.6	1.9	24	9	6	1 in 10
1.6	162	3	1.4	2.0	23	8	8	1 in 6
1.2	122	3.2	2.0	1.7	25	12	1	1 in 60
1.4	142	3.2	1.8	1.7	26	13	5	1 in 10
1.6	162	3.2	1.6	2.0	25	11	6	1 in 10
1.2	122	3.4	2.2	1.7	25	12	1	1 in 60
1.4	142	3.4	2.0	1.9	26	13	1	1 in 60
1.6	162	3.4	1.8	2.1	27	14	5	1 in 10
1.2	122	3.5	2.3	1.7	24	12	0	< 1 in 60
1.4	142	3.7	2.3	1.9	26	13	0	< 1 in 60
1.6	162	3.9	2.3	2.1	27	14	0	< 1 in 60

It is apparent from the results in Table 4 that frequency of overflow is dependent on freeboard. A minimum freeboard of 1.6m being required to limit overflows to once in 10 years. Reducing the freeboard to 1m results in overflow in 8 out of every 10 years

Salinity also needs to be considered. When the pond overflows, assuming that full mixing has occurred, some salt will be lost. Salt can be replenished by using sea water make up in the dry season but the minimum salinities listed in Table 4 could persist for some months before pond level dropped sufficiently to allow this topping up.

It is concluded that <u>substantial freeboard</u> must be provided if frequency of overflow is to be limited to say less that once in 5 years.

3. Dam/Offstream Storage Yield Study

The proposed water supply for the project comprises a dam on Broads Creek, a pumping station at the dam transferring water to an Offstream Storage. For the purpose of the study the dam was assumed to be a simple gully dam on the Creek. The dam depth/area/storage relationship was determined from the 1:10,000 topographic map. Storage provided by excavation to provide bank material was ignored. The off stream storage was assumed to be a 300m diameter circular tank with inside bank batters of 1 on 1.5. Depth /Area/Volume relationships for the two storages are shown in Table 5.

Table 5
Depth/Area/Volume

(a) Gully Dam

RL	Area ha	Volume ML
4	0.1	0
5	1.4	8
6	5.6	43
8	16.7	266
10	45	883
12	110	2433

(b) Offstream Storage

RL	Area ha	Volume ML
0	7.07	0
1	7.26	71
2	7.45	144
3	7.65	218
4	7.84	294
5	8.04	371
6	8.24	450
7	8.45	530
8	8.66	612
9	8.87	695
10	9.08	780
11	9.29	866
12	9.51	954

A daily rainfall/runoff model combined with a storage behaviour model was used to simulate the behaviour of the proposed dam on Broads Creek and the adjacent offstream storage. The computer program **2Dam** (Barlow 1990), with minor amendments, was used to carry out the study.

The model operates by calculating daily runoff from daily rainfall and monthly evaporation, using the AWBM Model (Boughton 1996). Runoff volume from this catchment model then becomes the inflow to the Gully Dam. Outflow from the Gully dam consists of spill, if any, and transfer to the Offstream Storage, if sufficient water is available and if the level in the Offstream Storage is low enough to accept the transfer volume. Evaporation and seepage from the Gully dam are also included in the water balance. Offstream Storage inflows are the transfer from the Gully Dam and direct rainfall on the storage. Outflows are the top up volumes required by the growing pond, as calculated by the pond water balance model, and evaporation. Zero seepage was assumed. The water balance is calculated on a daily basis but the model provides monthly outputs of water levels and storage volumes and monthly volumes of transfer, spills and draft. Totals of inflows and outflows and numbers of storage failures (dam level below dead storage or insufficient water to supply the specified draft) are also provided.

Typical catchment parameters were used for the AWBM model as no flow data were available to determine the parameters by calibration. The modeled runoffs will therefore be less accurate than if a calibrated model had been available. For the period of record the total volume of runoff is approximately 36% of total rainfall. Individual monthly runoff coefficients vary from zero to 0.8 depending on the amount of rain and the catchment conditions. These values are considered to be representative of Top End catchments.

The model was run for varying maximum levels of storage in both the Gully Dam and the Offstream storage and with different pump capacities to determine minimum requirements to meet water requirements for the 62 years of record. A pump capacity of 50 L/s was found to be adequate provided storages were large enough. Higher pump capacities made little or no difference to the performance of the system. Table 6 shows the minimum Gully Dam full supply levels required for different stages of development, assuming a usable depth of the Offstream Storage of 5.5m. For modeling purposes it was assumed that for each 10ha of growing pond area, an additional Offstream Storage of the same size would be provided.

Table 6
Minimum Storage Requirements

Pond Area ha	Gully Dam Crest RL
10	6.00
20	8.00
30	10.00
40	10.00
50	11.00
60	11.00
70	12.00
80	12.00*

* The Offstream Storage would have to be increased by about 20% to meet the requirements of 80 ha of pond area without failure.

Gully Dam crest levels could be reduced in some cases, generally by a metre or so, if larger Offstream storage were used. A crest RL of 6.00 is however the minimum required even for the initial 10ha. For the maximum development of 80ha a minimum crest level of about RL11.00 would be required.

It is concluded that the fresh water requirements of the proposal can be met by a combination of Gully Dam and Offstream Storage provided that the construction of storages of the appropriate size is feasible and economic.

4. Flood Study

A flood study of the Creek catchment was required to:

(a) Define the extent of the 1 in 100 annual exceedence probability flood adjacent to the proposed works.
(b) To provide design flood flows to assist in the design of the Gully Dam Spillway.

The rainfall/runoff model RORB (Laurenson & Mein 1999) was used with statistical rainfall intensity data for the Q100 event, obtained from the publication Australian Rainfall and Runoff (I.E.AUST. 1987). The catchment was modelled to a point downstream of the proposed development with output being provided at the catchment outlet and at the proposed dam site.

The catchment is shown on Fig 1.

Table 7 shows the rainfall Intensity-Frequency-Duration data for the site obtained from AR&R and Table 8 shows the 1 in 100 ARI flows at the catchment outlet and at the dam site for varying rainfall durations.

Table 7
Intensity – Frequency – Duration

	Average Recurrence Interval - years						
Duration	**1**	**2**	**5**	**10**	**20**	**50**	**100**
5min	209.1	200.8	134.2	106.1	90.4	74.8	65.6
6min	193.2	188.4	130.1	104.8	90.6	76.5	68
10min	151	154.4	118	100.7	91.5	81.8	75.7
20min	102.3	112.8	101.2	94.5	92.9	91.1	89.8
30min	79.4	91.9	91.6	90.7	93.9	97.7	100.5
1 hr	49.6	62.8	76	84.1	95.6	111.3	123.6
2 hr	29.9	38	46.1	51	58	67.6	75.1
3 hr	22.1	28.1	34.1	37.8	43	50.2	55.8
6 hr	13.2	16.7	20.4	22.6	25.7	30	33.4
12 hr	7.9	10	12.2	13.5	15.4	18	20.1
24 hr	5.1	6.5	8.1	9.2	10.6	12.6	14.1
48 hr	3.2	4.1	5.3	6.1	7.1	8.6	9.7
72 hr	2.4	3.1	4	4.6	5.5	6.7	7.7

Table 8
Flood Flows

Rainfall Duration - hours	Flow at Dam Site – m^3/s	Flow at Catchment Outlet - m^3/s
1	203	169
2	231	208
3	228	228
4.5	223	238
6	207	233

As shown in Table 8 the maximum Q100 flow at the dam site was 231 m^3/s and at the catchment outlet 238 m^3/s.

Q100 levels were determined downstream of the dam site using the steady state back water model HEC RAS (Hydrologic Engineering Centre 1996). Cross sections were scaled from the 1:10,000 topographic map at approximately 300m intervals from the dam site to the catchment outlet. A flow of 250 m^3/s was used. This produced a level of RL6.5 at the dam site reducing to less than RL 3.0 at the catchment outlet. **It is concluded that flooding will not be a problem to the proposed development.**

The RORB model allows preliminary spillway design to be carried out. Using the dam storage from Table 5(a) a spillway width of at least 100m would be required to limit overflow depth to about 1m. Velocity would be of the order of 2 m/s at the crest but could be considerably higher at the toe of the bank. **It is concluded that substantial and probably expensive spillway works will be required.**

References

Barlow F.T.H.	2Dam Model - Fortran Program Water Resources Division, Power and Water Authority 1990 unpublished
Boughton W.	AWBM Catchment Water Balance Model – CRC for Catchment Hydrology, Monash University 1996.
Hydrologic Engineering Center US Army Corps of Engineers	HEC-RAS River Analysis System Version 1.2 April 1996.
Institution of Engineers Australia.	Australian Rainfall and Runoff A Guide to Flood Estimation Two Volumes 1987.
Laurenson E.M. & Mein R.G.	RORB Version 4 Runoff Routing Program User Manual Monash University Dept. of Civil Engineering May 1990.

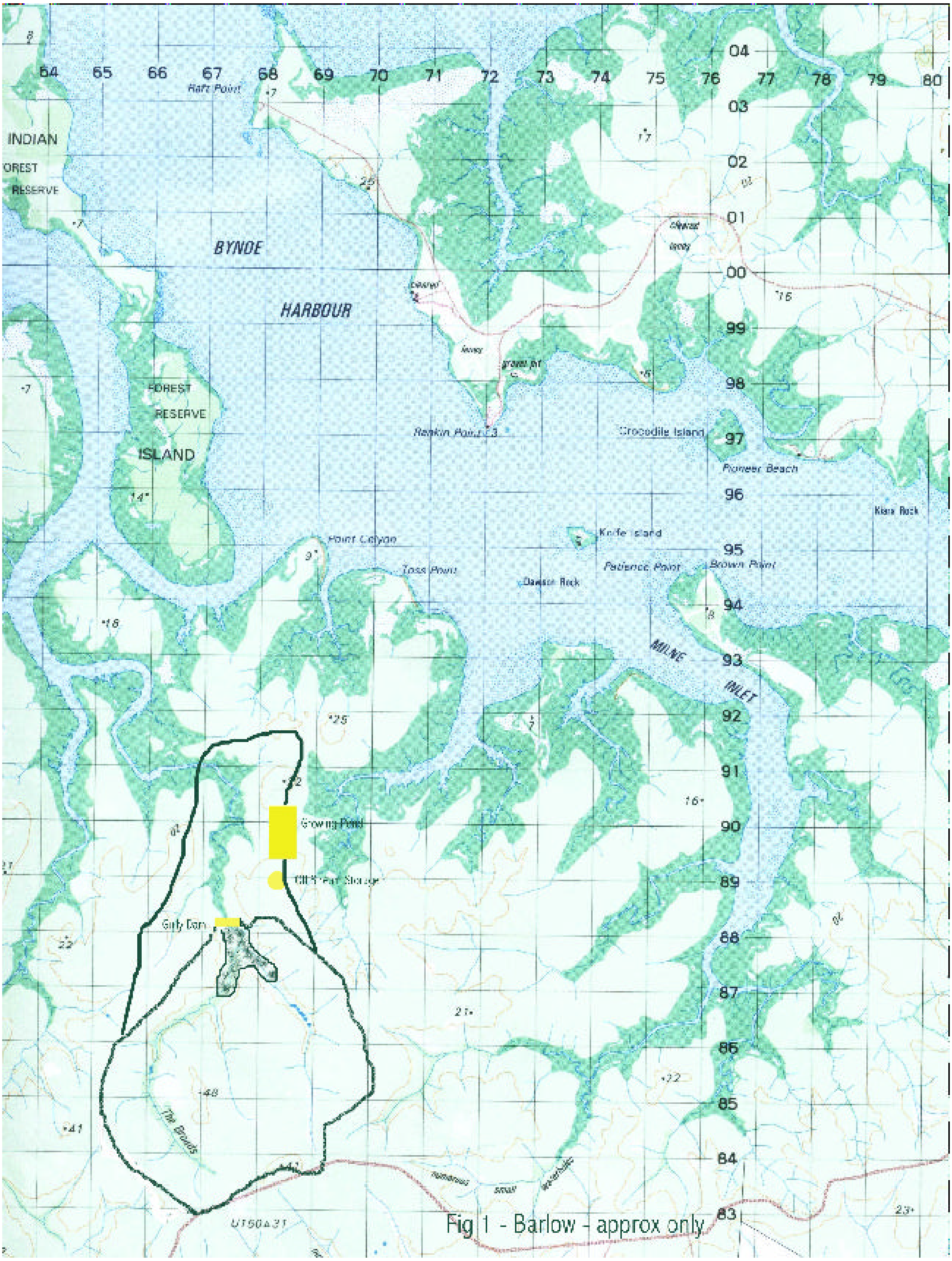

Fig 1 - Barlow - approx only