Government of Guyana

Task Force for Infrastructure Rehabilitation

Draft Report On Conservancy Flood Management Modelling

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1. Introduction

During January of 2005, the coastal regions of Guyana experienced prolonged and extremely heavy rainfall that resulted in serious flooding in Georgetown, and along the coastal belts of Regions 3 and 4. Many thousands of people had to leave their homes, and the entire population of the coastal region were affected in some way. In regions 3 and 4, conservancies lying behind the coastal strip provide significant flood mitigation, as well as water supply for irrigation during the dry seasons. Boerasirie Conservancy in Region 3 overtopped during the flood at a number of places, although the embankment did not breach. East Demerara Conservancy (EDC) in Region 4 also overtopped in some locations and there was considerable concern about the safety of sections of the embankment. The EDC embankment has been breached in the past, and widespread overtopping and breaching would have occurred had the January 2005 flood levels been only 60 mm higher (UNDAC, 2005).

A "Task Force for Infrastructure Rehabilitation" was established by the Government of Guyana in February 2005. The Task Force is implementing emergency works with funds committed by UNDP, DFID, USAID, CIDA, EU and WB. The study described in this report is in support of the Task Force and is concerned with an assessment of the characteristics of the January 2005 flood, of the impact that emergency works on the EDC will have on maximum water levels, and with providing guidance on control water levels and gate operation in the EDC.

The objectives of the flood routing and flood management studies may be summarised as follows:

- To establish the frequency of the January 2005 flood event, and to prepare "design storm" events for use in reservoir flood routing and flood management studies;
- To prepare conservancy flood inflows on the basis of the design storms;
- To model the response of the conservancies to design flood events, and evaluate the impacts of different operational regimes on peak water levels in the conservancies;
- To make recommendations for operational procedures in the wet season commencing mid-May 2005.

Extensive hydrological analysis was carried out in 2003 as part of the Guyana Drainage and Irrigation Systems Rehabilitation Project (GDISRP, Mott MacDonald, 2004). Rainfall frequency analysis was carried out using the available historic data up to 2002, and conservancy flood routing and water resources yield assessments were also carried out. The studies reported on here build on the previous work carried out by Mott MacDonald (2004), bringing the rainfall frequency analysis up to date, and extending the conservancy flood routing analysis through the use of a more versatile computational hydraulic model. This report has been prepared largely during a 10 day stay in Guyana (3 – 12th May as working days) by Dr Robin Wardlaw who had carried out the hydrological analysis on the GDISRP. Some UK time was used for data analysis and to finalise aspects of this report.

2. Hyrdometeorological Analysis

2.1 General

As part of the GDISRP extensive hydrometeorological analysis was carried out. The project was concerned with both drainage and irrigation, and the analysis thus addressed the characteristics of both flood and drought. Data were quality controlled, and analysed to produce annual maximum rainfall frequencies for use in conservancy flood evaluation, and in drainage design, as well as for the determination of crop water requirements.

2.1 Rainfall Frequency Analysis

The GDISRP identified the following stations as being suitable for use in determining design storm rainfalls for the conservancies:

Timehri
Boerasirie
Leonora Back
Georgetown Botanical Gardens
De Kenderin Back
Uitvlugt Back
Vryheid's Lust Back
Enmore Back
La Bonne Intention Back
Ogle Back

Design events for reservoir flood evaluations are generally for long return periods. Typically a 10,000 year event or greater would be used when a failure could result in loss of life downstream. In forecasting extremes, it is generally desirable not to extrapolate for more than three times the length of historical record available. Thus to reliably estimate a 1000 year event, it is desirable to have more than 300 years of record (a different approach is used for probable maximum precipitation PMP). It is also necessary generally to investigate events of different duration, particularly when a reservoir may introduce significant attenuation, as is the case with the Boerasirie and East Demerara conservancies. In order to achieve this longer record on which to base frequency analysis, the above stations were formed into a pooling group.

The rational for selection of the above stations has been outlined by Mott MacDonald (2004), as has the methodology for analysis of the pooling group records. HYDROMET supplied updated data for the above stations for the years 2003, 2004 and January and February 2005. Analysis has been carried out on a water year basis running April to March, and 3 further years have been added to the previous analysis, including what at many stations and durations, has been the largest event on record. March 2005 was simply excluded from the analysis, as at all stations annual maximums would have occurred in January 2005. In addition to updating the previous analysis with the recent records, the durations at which frequency analysis has been carried out have been extended to include 10, 15, 20 and 30 day durations. The method of L-moments was used to fit Generalised Extreme Value distributions.

The results of the frequency analysis are presented in Appendix A for the pooling group at all durations, and in Appendix B for the Georgetown Botanical Gardens station analysed independently. Table 1 summarises the results of the pooling group analysis. The 2005 rainfalls at Georgetown, Leonora Back and Timehri are included in Table 2.1 as a frame of reference for the 2005 event.

The storm of 2005 was very extreme, and a durations of 5-days and 7-days, the return period at many locations was in excess of 1000 years – i.e. an annual probability of occurrence of less than 0.001. The coastal regions certainly received the heaviest rainfalls. Recorded amounts at Timehri were significantly less than at Georgetown or Leonora Back, indicating that the return period of the rainfall over the catchment areas of the conservancies was somewhat less than that for the coastal areas.

Table 2.1
Annual maximum rainfalls for regions 3 and 4 estimated from pooled frequency analysis

Return Period	1-day	2-day	3-day	5-day	7-day	10-day	15-day	20-day
20	164	215	267	331	382	468	584	687
50	193	245	305	387	445	552	682	795
100	215	267	334	432	493	620	759	877
200	239	288	362	478	543	691	838	960
500	272	315	399	542	611	791	947	1071
1,000	298	336	427	593	664	872	1032	1157
10,000	397	402	519	778	851	1173	1335	1450
January 2005								
Georgetown	166	293	429	649	716	792	856	991
Leonora Back	159	255	352	527	721	786	879	1083
Timehri	88	170	208	272	327	357	480	824

2.2 Estimating Catchment Rainfalls

Estimates have been made of catchment rainfall during the January 2005 event. For the Boerasirie conservancy, rainfall was taken to be the average of that recorded at the Boerasirie and De Kenderin Back stations. The rainfall for the natural catchment draining to the conservancy was taken to be the average of Timehri and the calculated conservancy rainfall. For the EDC, conservancy rainfall was taken to be the average of that recorded at Georgtown, Ogle and LBI. The rainfall for the natural catchment draining to the conservancy was taken to be that recorded at Timehri.

For the 2005 event, rainfall intensities were recorded at Georgetown Botanical Gardens. The format in which data were readily available was in terms of intensities at different durations for each day of January, rather than the actual hourly or 15 minute rainfall values. These intensities were used to create daily rainfall intensity profiles centred on the middle of the day, that could subsequently be used in rainfall-runoff modelling. The conservancies are not sensitive to short duration high intensity rainfalls and this approach was therefore satisfactory.

2.3 Developing Synthetic Storm Profiles

Synthetic storm profiles were developed as part of the GDISRP by Mott MacDonald (2004) for durations ranging between 30 minutes and 7 days. The profiles were stacked profiles in which the frequency of the rainfall depth at all durations was the same. This type of analysis is satisfactory for storms of perhaps up to 2 days duration, but is not representative for long duration events of 3 to 20 days, where there will be distinct pulses of storm activity throughout the period, and what occurs is a succession of storms rather than a single storm. The previous analysis of Mott MacDonald has been updated with the rainfall intensity data collected since 2002. These storm profiles are not useful for conservancy flood evaluation, but would be of value in designing drainage in urban environments. Table 2.2 summarises the revised storm profiles for one, two and three day durations.

Table 2.2
Synthetic storm profiles for short duration events

	30 Mins	1 Hour	2 Hours	6 Hours	12 Hours	24 Hours	48 Hours	72 Hours
24 hour	0.34	0.51	0.71	0.84	0.86	1.00		
48 hour	0.29	0.43	0.59	0.71	0.72	0.84	1.00	
72 hour	0.23	0.34	0.48	0.57	0.58	0.67	0.80	1.00

2.4 Profiles for Long Duration Events

For design storms at 5, 7, and 10-day durations, the stacked profiles are not appropriate. Profiles for these longer durations have been based on the daily distributions of rainfall at these durations observed during the 2005 event in the estimated conservancy and catchment rainfalls. Proportions of the total storm rainfall at durations of 5, 7, and 10-day durations were calculated and these scaled with estimated storm depths for a range of return periods, to produce resulting daily storm profiles. The hourly distribution of daily rainfalls was based on the average distributions for the peak three days of the January 2005 event.

3. Estimating Conservancy Flood Runoff

3.1 The HEC-HMS Model

The HEC-HMS model was set up to generate flood inflows to the conservancies as part of GDISRP (Mott MacDonald, 2004). On each conservancy, the model set up included two sub-basins, one representing the natural catchment area, and one representing the conservancy itself, which was treated as an impermeable surface. The models were set up with conservancy areas set at the design high water level for the purpose of sub-basin definition. Clearly conservancy area, and impervious area, varies with water level, but it was considered that since relatively high curve numbers (for SCS runoff generation) were being used for the natural catchment areas, there would not be significant error in assuming fixed areas for runoff generation purposes.

In the GDISRP, the conservancies were also modelled directly with HEC-HMS. However, HEC-HMS is only capable of representing one reservoir outflow, and it had been necessary to develop combined discharge rating relationships for each conservancy. This was not considered to be satisfactory if the impact of improved outfall conditions was to be investigated, and in the work described in this report, HEC-HMS reservoir routing has not been used. HEC-HMS has been used to synthesise inflow hydrographs to conservancy models that have been set up using the HYDRO1D model.

HEC-HMS has been used to synthesise inflows to both conservancies for the January 2005 flood, and for a series of “design floods” with return periods in the range of 100, 200, 500, 1000 and 10,000 years, at durations of 3, 5 and 10-days.

4. Conservancy Flood Routing

4.1 The HYDRO1D Model

HYDRO1D is a computational hydraulic model developed by Mott MacDonald. It provides a full solution of the de St Venant equations and has been widely used in flood mitigation studies throughout the world. The model is capable of representing a wide variety of structures and is capable of representing complex channel geometries, and floodplain storage arrangements. It can deal looped channels and flow reversals and is not limited in the number of outlet structures that may be linked to a storage node in the model.

4.2 Representation of the Conservancies

The conservancies serve functions of water supply and flood mitigation. They have been represented in the HYDRO1D model (at present) as level pools – i.e. as normal large open surface reservoirs, in which appreciable gradients in water surface elevation over the reservoir area are not expected. However, the water level range in the conservancies is not large, and they may in fact be characterised as wetlands with a number of inter-linking channels. Photographs of the conservancies are shown in Figures 4.1 and 4.2.

Figure 4.1 Boerasirie Conservancy close to Leonora

Figure 4.2 East Demerara Conservancy close to Maduni Sluice

Ideally the conservancies would be modelled in a pseudo two-dimensional mode. The interlinking channels would be represented as the main flow routes through the conservancy and the main storage areas linked to the channels either as floodplain, or as storage cells with elevation–area–storage characteristics. However, such an approach is not possible without extensive hydrographic survey of the conservancies. No suitable survey information exists at present, and until it does, the conservancies can only be modelled as single storage cells. Significant hydraulic gradients were recorded in the Boerasirie conservancy during the 2005 floods, and indicate that a level pool approach for that conservancy will not be appropriate. This is discussed further in subsequent sections.

4.3 Modelling the EDC

4.3.1 General

The following description is largely taken from the GDISRP (Mott MacDonald, 2004), and is entirely relevant to the present study. Sluice gate discharge functions have been developed under the present project.

The East Demerara Conservancy has a total catchment area of 582 km^2. This estimate is based on catchment delineation on 1:50,000 scale topographic mapping and compares with a figure of 518 km^2 estimated by Hutchinson in the 1951. Figure 4.3 shows the catchment area. As with the Boerasirie Conservancy, the conservancy itself is a significant part of the total catchment area. At an elevation of 17.53 m (57.51 ft), the reservoir area is some 335 km^2. The remaining natural catchment area is heavily vegetated and of low relief. It is not underlain by white sand deposits, and is likely to have a higher storm runoff than the Boerasirie catchment. The slope of the longest stream path is 0.00061.

The elevation-area characteristics for the East Demerara Conservancy are shown in Figure 4.4, and elevation-storage-area characteristics are summarised in Table 4.1. These data are as prepared by Hutchinson in 1951, and it should be noted that a discrepancy was found between the area data and the storage data. Storage computed from changes in area does not correspond with the published storage data. It is assumed that the area data are correct, and that the error lies in the 1950s published storage data. The storage values given in Table 4.1 are computed from the areas.

Table 4.1
Elevation-area-storage characteristics for East Demerara Conservancy

Elevation (m G.D.)	Area (km^2)	Storage (Mm^3)
16.15	10.36	11.33
16.31	20.72	13.70
16.46	25.90	17.25
16.76	51.80	29.09
17.07	93.24	51.20
17.25	145.04	72.98
17.37	266.77	98.09
17.53	336.70	144.07
17.68	406.63	200.71
17.98	520.59	340.05

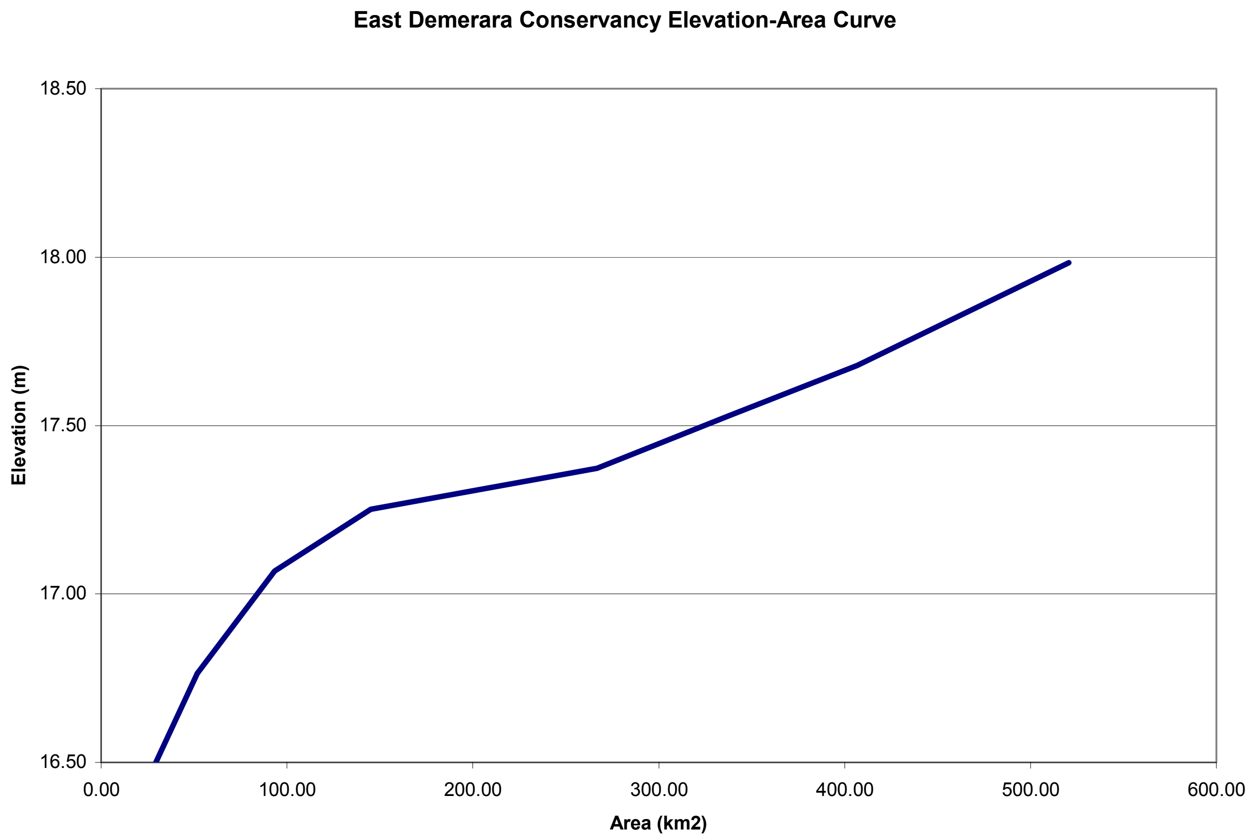

Figure 4.4 East Demerara Conservancy elevation-area curve

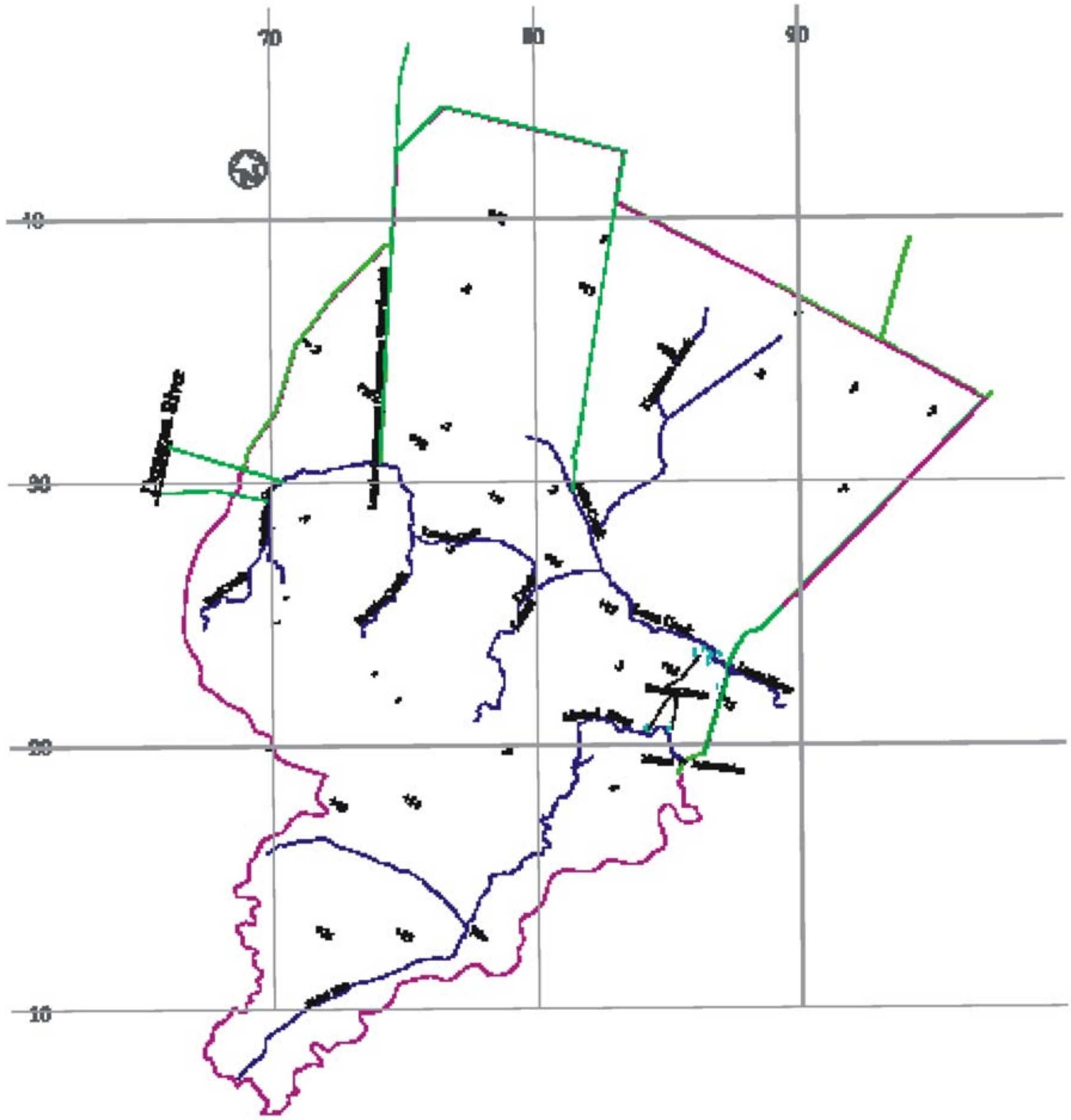

Figure 4.3 East Demerara Conservancy catchment area.

The elevation-area-storage characteristics are likely to have changed at lower elevations, with possible consequence for water resources. It is clearly very desirable that the elevation-area-storage characteristics of the conservancy are accurately re-defined.

Flood relief from the East Demerara Conservancy, at the time of the January 2005 floods, was provided through three sets of gated outlet structures:

- Land of Canaan Sluice
- Maduni Sluice
- Lama Sluice.

The locations of these structures are shown on the plan of the conservancy in Figure 4.5.

The embankment levels and freeboard around the East Demerara Conservancy are generally higher than on the Boerasirie Conservancy, but are variable. The Lands and Surveys Department carried out a topographic survey of the embankment top in 2005. A cumulative frequency plot of the embankment top levels is presented in Figure 4.6.

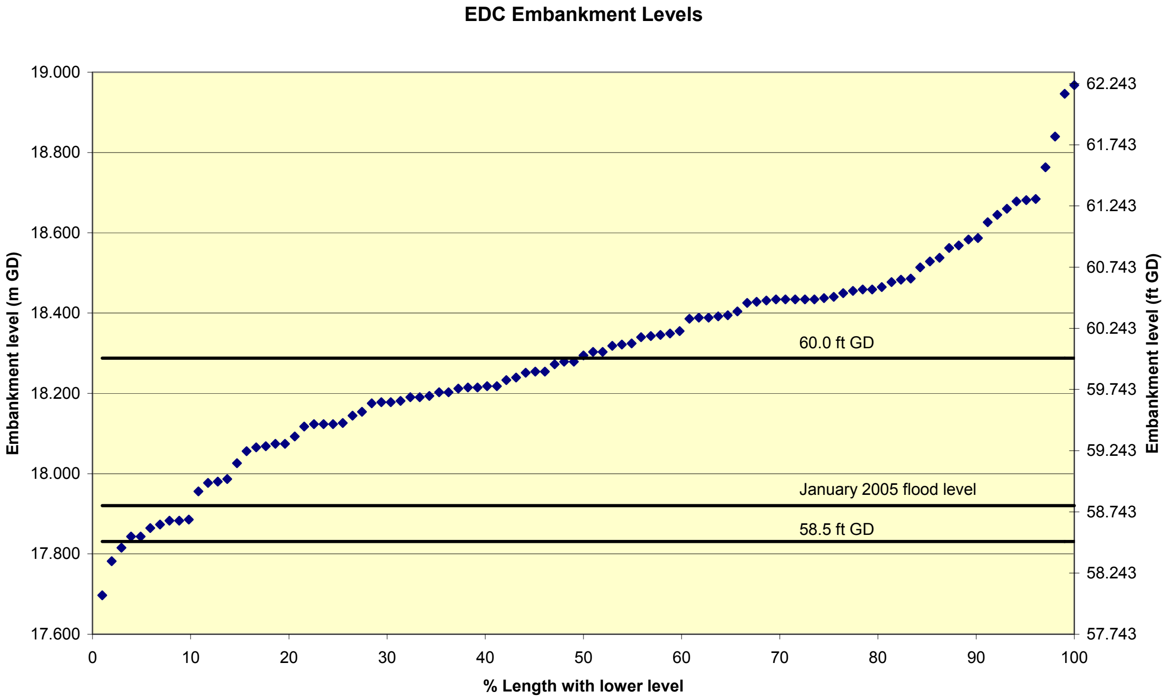

Figure 4.6 Crest elevations on EDC embankment

It would appear that about 10% of the EDC embankment was either overtopped or required sandbagging during the January 2005 flood, and that inadequate freeboard existed on over 50% of the embankment length.

4.3.2 Land of Canaan Sluice

The Land of Canaan Sluice comprises five radial gates, each of width 4.877 m. A photograph of the sluice is shown in Figure 4.7. The sill level is at 15.804 m, and historically the sluice has been fully opened when the Conservancy water level reaches 17.374 m (57 ft, discussion with Conservancy staff). The gates are lifted clear of the water surface, and a standing wave generally forms downstream. There may be tidal conditions that do result in drowning out of the structure, however, and for this reason a discharge rating table has been computed for the structure. The discharge table for the sluice is presented in Table 4.2 below. At present survey is not available for the channel downstream of the Land of Canaan sluice.

Table 4.2
Calculated discharge matrix for Land of Canaan Sluice – all gates free of water surface

	WL d/s																
WL u/s	15.6	15.7	15.8	15.9	16	16.1	16.2	16.3	16.4	16.5	16.6	16.7	16.8	16.9	17	17.1	17.2
16	3.49	3.49	3.49	3.49	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16.1	6.48	6.48	6.48	6.48	6.48	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16.2	10.03	10.03	10.03	10.03	10.03	6.72	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16.3	14.05	14.05	14.05	14.05	14.05	14.05	8.69	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16.4	18.51	18.51	18.51	18.51	18.51	18.51	18.51	10.70	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16.5	23.36	23.36	23.36	23.36	23.36	23.36	23.36	16.39	12.75	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16.6	28.57	28.57	28.57	28.57	28.57	28.57	28.57	28.57	19.12	14.84	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16.7	34.12	34.12	34.12	34.12	34.12	34.12	34.12	34.12	34.12	21.88	16.96	0.00	0.00	0.00	0.00	0.00	0.00
16.8	39.99	39.99	39.99	39.99	39.99	39.99	39.99	39.99	39.99	28.53	24.68	19.10	0.00	0.00	0.00	0.00	0.00
16.9	46.17	46.17	46.17	46.17	46.17	46.17	46.17	46.17	46.17	46.17	31.84	27.52	21.28	0.00	0.00	0.00	0.00
17	52.63	52.63	52.63	52.63	52.63	52.63	52.63	52.63	52.63	52.63	52.63	35.19	30.38	23.47	0.00	0.00	0.00
17.1	59.36	59.36	59.36	59.36	59.36	59.36	59.36	59.36	59.36	59.36	59.36	42.71	38.57	33.27	25.68	0.00	0.00
17.2	66.36	66.36	66.36	66.36	66.36	66.36	66.36	66.36	66.36	66.36	66.36	66.36	46.52	41.98	36.19	27.92	0.00
17.3	73.62	73.62	73.62	73.62	73.62	73.62	73.62	73.62	73.62	73.62	73.62	73.62	73.62	50.36	45.42	39.13	30.17
17.4	81.13	81.13	81.13	81.13	81.13	81.13	81.13	81.13	81.13	81.13	81.13	81.13	81.13	58.67	54.23	48.88	42.10
17.5	88.87	88.87	88.87	88.87	88.87	88.87	88.87	88.87	88.87	88.87	88.87	88.87	88.87	88.87	62.92	58.13	52.37
17.6	96.84	96.84	96.84	96.84	96.84	96.84	96.84	96.84	96.84	96.84	96.84	96.84	96.84	96.84	96.84	67.20	62.05
17.7	105.04	105.04	105.04	105.04	105.04	105.04	105.04	105.04	105.04	105.04	105.04	105.04	105.04	105.04	105.04	76.23	71.50
17.8	113.46	113.46	113.46	113.46	113.46	113.46	113.46	113.46	113.46	113.46	113.46	113.46	113.46	113.46	113.46	113.46	80.88
17.9	122.09	122.09	122.09	122.09	122.09	122.09	122.09	122.09	122.09	122.09	122.09	122.09	122.09	122.09	122.09	122.09	122.09
18	130.93	130.93	130.93	130.93	130.93	130.93	130.93	130.93	130.93	130.93	130.93	130.93	130.93	130.93	130.93	130.93	130.93
18.1	139.98	139.98	139.98	139.98	139.98	139.98	139.98	139.98	139.98	139.98	139.98	139.98	139.98	139.98	139.98	139.98	139.98
18.2	149.22	149.22	149.22	149.22	149.22	149.22	149.22	149.22	149.22	149.22	149.22	149.22	149.22	149.22	149.22	149.22	149.22
18.3	158.66	158.66	158.66	158.66	158.66	158.66	158.66	158.66	158.66	158.66	158.66	158.66	158.66	158.66	158.66	158.66	158.66
18.4	168.29	168.29	168.29	168.29	168.29	168.29	168.29	168.29	168.29	168.29	168.29	168.29	168.29	168.29	168.29	168.29	168.29
18.5	178.11	178.11	178.11	178.11	178.11	178.11	178.11	178.11	178.11	178.11	178.11	178.11	178.11	178.11	178.11	178.11	178.11

Figure 4.7 Land of Canaan Sluice

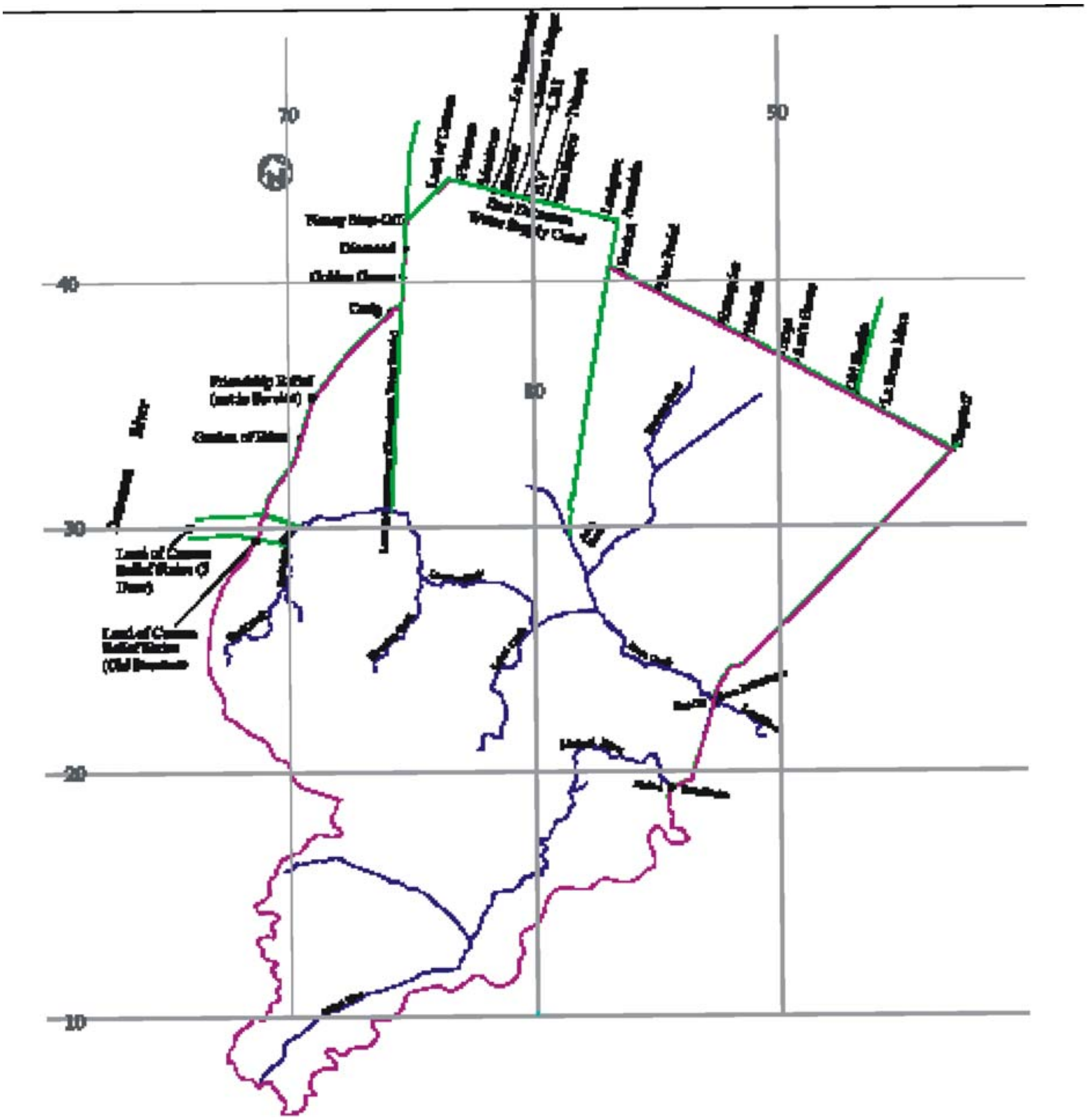

Figure 4.5 Location of structures on East Demerara Conservancy

4.3.3 Maduni Sluice

The Maduni sluice has a single vertical lift gate, of width 4.877m. A photograph of the sluice is shown in Figure 4.8. The sill level is at 14.139 m. A smaller sluice also exists at Maduni, but has been blocked off and is not in use. Conservancy staff report that the gate has historically been lifted clear of the water once the conservancy level reaches 17.465 m (57.3 ft). The sluice was visited in May 2005, when the gate was fully open. The head loss through the structure was of the order of 200 mm (Figure 4.9) and free flow conditions did not exist.

Figure 4.8 Maduni sluice

It is not clear to what extent tidal influences exist at Maduni sluice, but it is known that at low conservancy level, water can be brought into the conservancy from Lama sluice at high tide in the Mahaica River. A discharge rating table has been prepared for the Maduni sluice and is presented in Table 4.3 below.

Figure 4.9 Maduni Sluice view from downstream, May 2005

Table 4.3
Calculated discharge matrix for Maduni Sluice – gates free of water surface

	WL d/s																
WL u/s	15.6	15.7	15.8	15.9	16	16.1	16.2	16.3	16.4	16.5	16.6	16.7	16.8	16.9	17	17.1	17.2
16	13.00	11.71	10.07	7.75	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16.1	14.95	13.80	12.42	10.68	8.22	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16.2	16.88	15.82	14.59	13.13	11.29	8.69	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16.3	18.80	17.81	16.69	15.40	13.85	11.91	9.16	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16.4	27.54	19.80	18.76	17.57	16.20	14.57	12.52	9.63	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16.5	29.38	29.38	20.81	19.71	18.46	17.02	15.30	13.15	10.11	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16.6	31.26	31.26	22.86	21.82	20.66	19.35	17.83	16.03	13.77	10.59	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16.7	33.17	33.17	33.17	23.93	22.84	21.62	20.24	18.65	16.76	14.40	11.07	0.00	0.00	0.00	0.00	0.00	0.00
16.8	35.13	35.13	35.13	35.13	25.01	23.86	22.58	21.13	19.47	17.50	15.03	11.55	0.00	0.00	0.00	0.00	0.00
16.9	37.12	37.12	37.12	37.12	27.18	26.09	24.88	23.54	22.03	20.30	18.24	15.66	12.04	0.00	0.00	0.00	0.00
17	39.15	39.15	39.15	39.15	39.15	28.32	27.17	25.91	24.51	22.94	21.13	18.98	16.30	12.52	0.00	0.00	0.00
17.1	41.21	41.21	41.21	41.21	41.21	41.21	29.45	28.26	26.94	25.49	23.85	21.96	19.73	16.93	13.01	0.00	0.00
17.2	43.31	43.31	43.31	43.31	43.31	43.31	31.74	30.60	29.35	27.98	26.46	24.76	22.80	20.48	17.57	13.50	0.00
17.3	45.44	45.44	45.44	45.44	45.44	45.44	45.44	32.94	31.74	30.45	29.02	27.45	25.67	23.64	21.23	18.22	13.99
17.4	47.61	47.61	47.61	47.61	47.61	47.61	47.61	47.61	34.14	32.90	31.55	30.07	28.43	26.59	24.48	21.98	18.86
17.5	49.81	49.81	49.81	49.81	49.81	49.81	49.81	49.81	36.53	35.34	34.05	32.65	31.12	29.42	27.51	25.32	22.74
17.6	52.04	52.04	52.04	52.04	52.04	52.04	52.04	52.04	52.04	37.78	36.55	35.21	33.76	32.17	30.41	28.43	26.17
17.7	54.30	54.30	54.30	54.30	54.30	54.30	54.30	54.30	54.30	54.30	39.04	37.76	36.37	34.87	33.22	31.40	29.36
17.8	56.60	56.60	56.60	56.60	56.60	56.60	56.60	56.60	56.60	56.60	41.54	40.30	38.97	37.54	35.98	34.28	32.40
17.9	58.93	58.93	58.93	58.93	58.93	58.93	58.93	58.93	58.93	58.93	58.93	42.85	41.57	40.19	38.71	37.10	35.34
18	61.29	61.29	61.29	61.29	61.29	61.29	61.29	61.29	61.29	61.29	61.29	61.29	44.16	42.83	41.41	39.88	38.22
18.1	63.68	63.68	63.68	63.68	63.68	63.68	63.68	63.68	63.68	63.68	63.68	63.68	46.76	45.48	44.11	42.64	41.06
18.2	66.10	66.10	66.10	66.10	66.10	66.10	66.10	66.10	66.10	66.10	66.10	66.10	66.10	48.12	46.79	45.38	43.87
18.3	68.55	68.55	68.55	68.55	68.55	68.55	68.55	68.55	68.55	68.55	68.55	68.55	68.55	68.55	49.48	48.12	46.66
18.4	71.03	71.03	71.03	71.03	71.03	71.03	71.03	71.03	71.03	71.03	71.03	71.03	71.03	71.03	52.18	50.85	49.44
18.5	73.54	73.54	73.54	73.54	73.54	73.54	73.54	73.54	73.54	73.54	73.54	73.54	73.54	73.54	73.54	53.59	52.22

4.3.4 Lama Sluice

Lama sluice includes two structures with single gates referred to as the big sluice and the small sluice. These are distinguished by their sill levels and discharge capacity. Photographs of the sluices are shown in Figure 4.10 and 4.11. The small sluice is 5.233 m wide and has a sill level of 15.458 m. The big sluice is 4.877 m wide and has a sill level of 14.315 m. These structures have historically been operated when the conservancy level reaches 17.465 m (57.3 ft). At this level the gates are lifted clear of the water. It is thought that backwater influences the discharge through the sluices, and discharge ratings have been prepared for each of them. These are presented in Tables 4.4 and 4.5.

Table 4.4
Calculated discharge matrix for Lama Big Sluice – gates free of water surface

	WL d/s																
WL u/s	15.6	15.7	15.8	15.9	16	16.1	16.2	16.3	16.4	16.5	16.6	16.7	16.8	16.9	17	17.1	17.2
16	11.80	10.69	9.27	7.24	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16.1	13.60	12.61	11.42	9.90	7.73	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16.2	15.40	14.49	13.43	12.16	10.54	8.23	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16.3	22.50	16.36	15.39	14.26	12.90	11.18	8.73	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16.4	24.23	24.23	17.32	16.29	15.09	13.65	11.83	9.23	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16.5	25.99	25.99	19.26	18.30	17.20	15.93	14.41	12.48	9.74	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16.6	27.79	27.79	27.79	20.29	19.27	18.12	16.78	15.17	13.14	10.24	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16.7	29.64	29.64	29.64	29.64	21.33	20.26	19.04	17.62	15.93	13.80	10.76	0.00	0.00	0.00	0.00	0.00	0.00
16.8	31.52	31.52	31.52	31.52	23.39	22.38	21.25	19.96	18.48	16.70	14.46	11.27	0.00	0.00	0.00	0.00	0.00
16.9	33.44	33.44	33.44	33.44	33.44	24.50	23.43	22.24	20.89	19.34	17.48	15.13	11.79	0.00	0.00	0.00	0.00
17	35.40	35.40	35.40	35.40	35.40	35.40	25.61	24.49	23.24	21.83	20.20	18.25	15.80	12.31	0.00	0.00	0.00
17.1	37.40	37.40	37.40	37.40	37.40	37.40	27.79	26.73	25.55	24.25	22.77	21.07	19.03	16.47	12.83	0.00	0.00
17.2	39.43	39.43	39.43	39.43	39.43	39.43	39.43	28.96	27.85	26.62	25.26	23.72	21.94	19.82	17.15	13.36	0.00
17.3	41.50	41.50	41.50	41.50	41.50	41.50	41.50	41.50	30.14	28.98	27.70	26.27	24.67	22.81	20.61	17.83	13.89
17.4	43.60	43.60	43.60	43.60	43.60	43.60	43.60	43.60	32.43	31.32	30.11	28.78	27.29	25.62	23.69	21.40	18.51
17.5	45.74	45.74	45.74	45.74	45.74	45.74	45.74	45.74	45.74	33.66	32.51	31.25	29.86	28.32	26.58	24.58	22.19
17.6	47.91	47.91	47.91	47.91	47.91	47.91	47.91	47.91	47.91	47.91	34.90	33.70	32.39	30.95	29.35	27.54	25.46
17.7	50.12	50.12	50.12	50.12	50.12	50.12	50.12	50.12	50.12	50.12	37.30	36.15	34.90	33.54	32.04	30.38	28.51
17.8	52.35	52.35	52.35	52.35	52.35	52.35	52.35	52.35	52.35	52.35	52.35	38.60	37.40	36.10	34.69	33.14	31.42
17.9	54.62	54.62	54.62	54.62	54.62	54.62	54.62	54.62	54.62	54.62	54.62	54.62	39.90	38.65	37.31	35.84	34.24
18	56.92	56.92	56.92	56.92	56.92	56.92	56.92	56.92	56.92	56.92	56.92	56.92	42.40	41.20	39.91	38.52	37.00
18.1	59.26	59.26	59.26	59.26	59.26	59.26	59.26	59.26	59.26	59.26	59.26	59.26	59.26	43.75	42.51	41.17	39.73
18.2	61.62	61.62	61.62	61.62	61.62	61.62	61.62	61.62	61.62	61.62	61.62	61.62	61.62	61.62	45.11	43.82	42.44
18.3	64.01	64.01	64.01	64.01	64.01	64.01	64.01	64.01	64.01	64.01	64.01	64.01	64.01	64.01	47.71	46.47	45.14
18.4	66.44	66.44	66.44	66.44	66.44	66.44	66.44	66.44	66.44	66.44	66.44	66.44	66.44	66.44	66.44	49.12	47.83
18.5	68.89	68.89	68.89	68.89	68.89	68.89	68.89	68.89	68.89	68.89	68.89	68.89	68.89	68.89	68.89	68.89	50.53

Figure 4.10 Lama Big Sluice

Table 4.5
Calculated discharge matrix for Lama Small Sluice – gates free of water surface

	WL d/s																
WL u/s	15.60	15.70	15.80	15.90	16.00	16.10	16.20	16.30	16.40	16.50	16.60	16.70	16.80	16.90	17.00	17.10	17.20
16.00	3.45	3.45	3.45	2.06	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16.10	4.44	4.44	4.44	3.21	2.50	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16.20	5.52	5.52	5.52	5.52	3.79	2.94	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16.30	6.67	6.67	6.67	6.67	6.67	4.37	3.39	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16.40	7.89	7.89	7.89	7.89	7.89	5.74	4.97	3.85	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16.50	9.18	9.18	9.18	9.18	9.18	9.18	6.45	5.58	4.31	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16.60	10.54	10.54	10.54	10.54	10.54	10.54	10.54	7.16	6.19	4.78	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16.70	11.95	11.95	11.95	11.95	11.95	11.95	11.95	8.73	7.88	6.80	5.25	0.00	0.00	0.00	0.00	0.00	0.00
16.80	13.42	13.42	13.42	13.42	13.42	13.42	13.42	13.42	9.54	8.61	7.43	5.73	0.00	0.00	0.00	0.00	0.00
16.90	14.95	14.95	14.95	14.95	14.95	14.95	14.95	14.95	14.95	10.36	9.35	8.06	6.21	0.00	0.00	0.00	0.00
17.00	16.53	16.53	16.53	16.53	16.53	16.53	16.53	16.53	16.53	12.10	11.19	10.09	8.69	6.70	0.00	0.00	0.00
17.10	18.17	18.17	18.17	18.17	18.17	18.17	18.17	18.17	18.17	18.17	13.01	12.02	10.83	9.33	7.19	0.00	0.00
17.20	19.85	19.85	19.85	19.85	19.85	19.85	19.85	19.85	19.85	19.85	19.85	13.92	12.86	11.59	9.97	7.68	0.00
17.30	21.59	21.59	21.59	21.59	21.59	21.59	21.59	21.59	21.59	21.59	21.59	15.82	14.84	13.71	12.34	10.62	8.18
17.40	23.37	23.37	23.37	23.37	23.37	23.37	23.37	23.37	23.37	23.37	23.37	23.37	16.82	15.77	14.56	13.10	11.27
17.50	25.20	25.20	25.20	25.20	25.20	25.20	25.20	25.20	25.20	25.20	25.20	25.20	25.20	17.82	16.70	15.41	13.87
17.60	27.07	27.07	27.07	27.07	27.07	27.07	27.07	27.07	27.07	27.07	27.07	27.07	27.07	19.86	18.82	17.64	16.27
17.70	28.99	28.99	28.99	28.99	28.99	28.99	28.99	28.99	28.99	28.99	28.99	28.99	28.99	28.99	20.94	19.83	18.58
17.80	30.95	30.95	30.95	30.95	30.95	30.95	30.95	30.95	30.95	30.95	30.95	30.95	30.95	30.95	30.95	22.02	20.85
17.90	32.95	32.95	32.95	32.95	32.95	32.95	32.95	32.95	32.95	32.95	32.95	32.95	32.95	32.95	32.95	24.20	23.10
18.00	34.99	34.99	34.99	34.99	34.99	34.99	34.99	34.99	34.99	34.99	34.99	34.99	34.99	34.99	34.99	34.99	25.35
18.10	37.08	37.08	37.08	37.08	37.08	37.08	37.08	37.08	37.08	37.08	37.08	37.08	37.08	37.08	37.08	37.08	37.08
18.20	39.20	39.20	39.20	39.20	39.20	39.20	39.20	39.20	39.20	39.20	39.20	39.20	39.20	39.20	39.20	39.20	39.20
18.30	41.37	41.37	41.37	41.37	41.37	41.37	41.37	41.37	41.37	41.37	41.37	41.37	41.37	41.37	41.37	41.37	41.37
18.40	43.57	43.57	43.57	43.57	43.57	43.57	43.57	43.57	43.57	43.57	43.57	43.57	43.57	43.57	43.57	43.57	43.57
18.50	45.81	45.81	45.81	45.81	45.81	45.81	45.81	45.81	45.81	45.81	45.81	45.81	45.81	45.81	45.81	45.81	45.81

Figure 4.11 Lama Small Sluice

4.3.5 Kofi and Cunha Sluices

The Kofi and Cunha sluices are of similar dimensions to the Maduni Sluice. The have not been operable for a very long time, but following the January 2005 floods are being re-habilitated. Survey information for the sluice sill levels was not available, and for modelling purposes the characteristics of Maduni Sluice have been assumed. Dimensions of the proposed channel downstream of Kofi sluice were available from tender documents, and the same section has been assumed for both Kofi and Cunha outfall channels.

4.3.6 Tidal Boundary Conditions

Each of the sluices providing flood relief from the EDC are influenced tidally under certain water level and flow conditions. Tidal records for Georgetown and Timehri were obtained from the data base of the Coastal Defences section of the Ministry of Public works. Data from Georgetown were available from November 2003 to March 2005. The Timehri records were from mid-May 2004 to mid-January 2005.

For modelling purposes it was important to establish boundary conditions downstream of each of the sluices. At Land of Canaan this could be based on the Georgetown and Timheri records, but for the Maduni and Lama sluices, assumptions were made on the basis of observations in May 2005, and adjustments made to Georgetown records to synthesise a record for use downstream of these sluices.

For design purposes it was also considered necessary to develop typical spring and neap tidal profiles for use with model runs. To do this the available records were reviewed and typical periods of neap and spring tides extracted. The neap tidal profiles are shown in Figure 4.12, and the spring tidal profiles in Figure 4.13. To determine a profile at Maduni, the Timehri profile was adjusted by reducing the amplitude by 50%, and setting the maximum tidal level to 16.5 m GD. This profile is shown in Figure 4.14. Clearly the Maduni profile will have to be established more objectively in any future work.

For the January 2005 flood, tidal records were not available for Timehri past 16th January. In modelling the January 2005 flood the Georgetown tidal record has been used for the Land of Canaan outfall, and the boundary levels of Figure 4.14 for the Maduni and Lama outfalls.

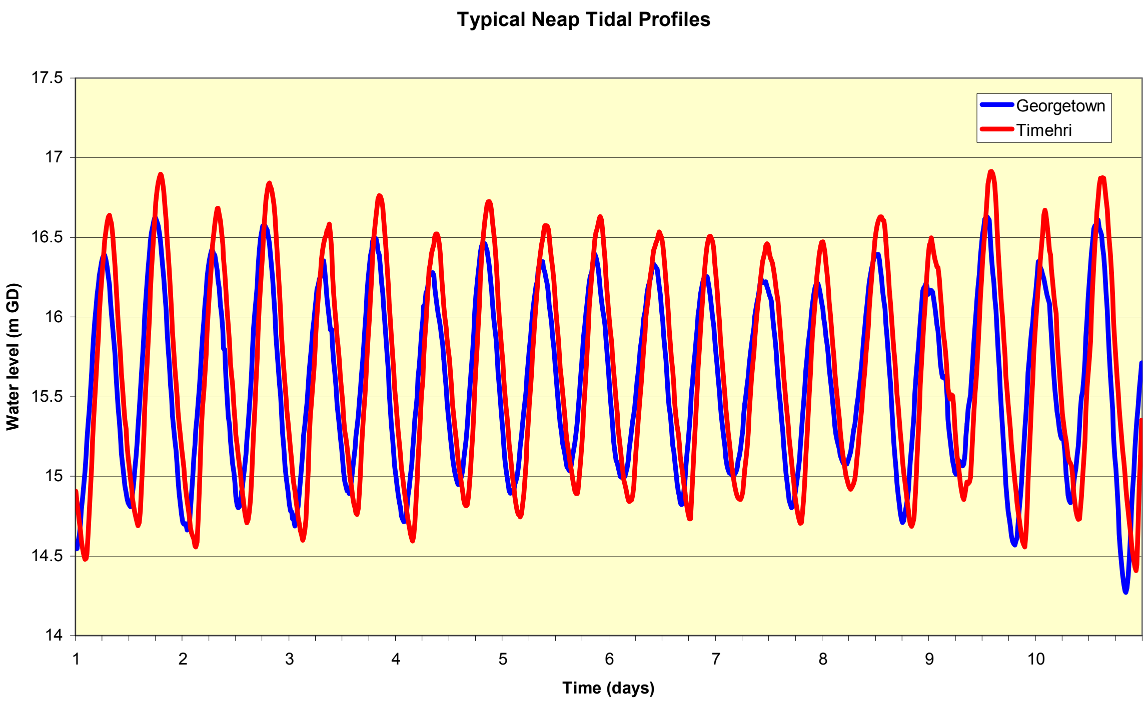

Figure 4.12 Adopted Neap tidal profile

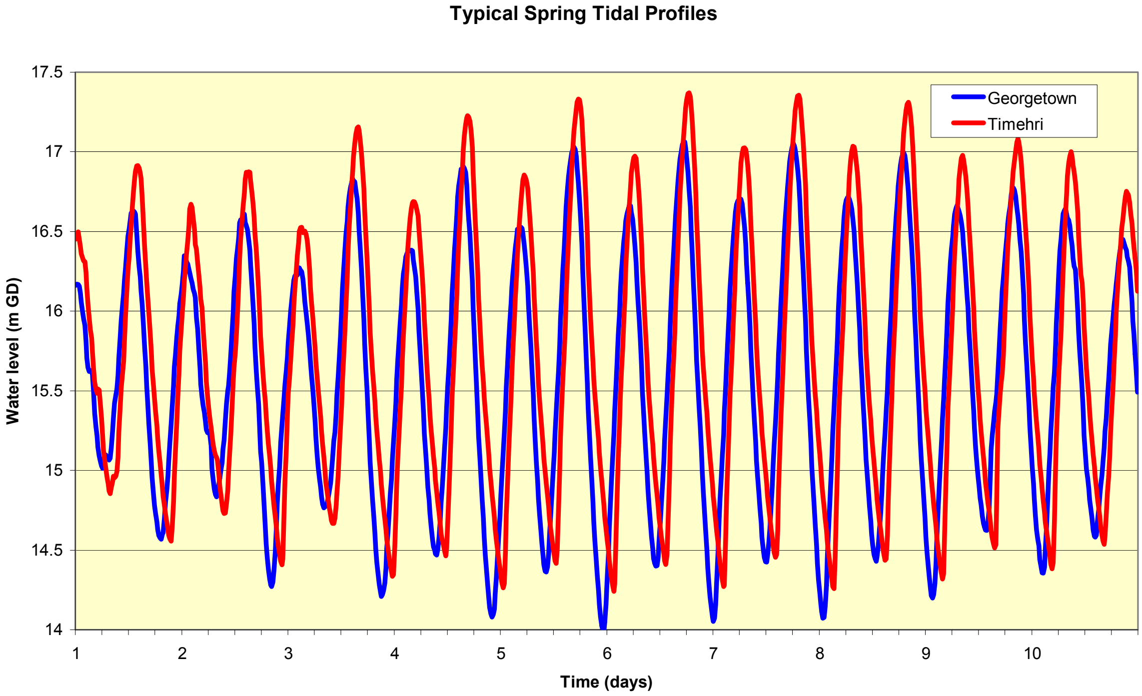

Figure 4.13 Adopted spring tidal profile.

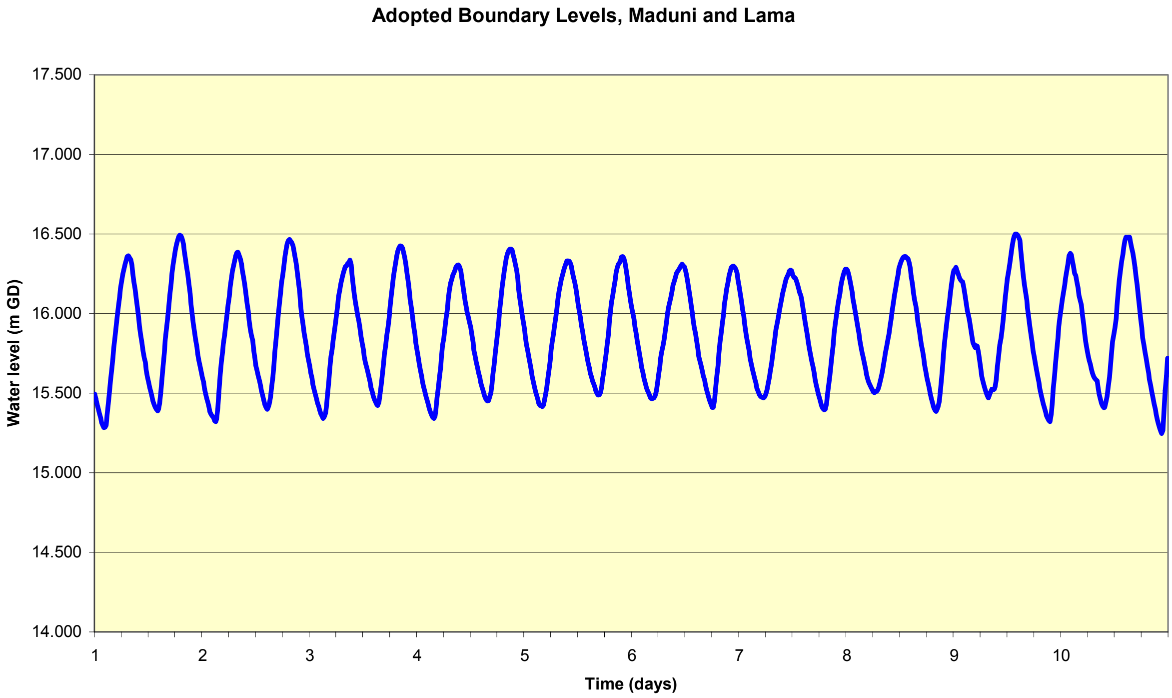

Figure 4.14 Boundary levels adopted for Maduni and Lama

4.3.7 HYDRO1D Model Set Up

The HYDRO1D model network for the conditions that existed during the January 2005 flood is shown in Figure 4.15. The network is very simple, with an outfall channel represented only for the Land of Canaan sluice.

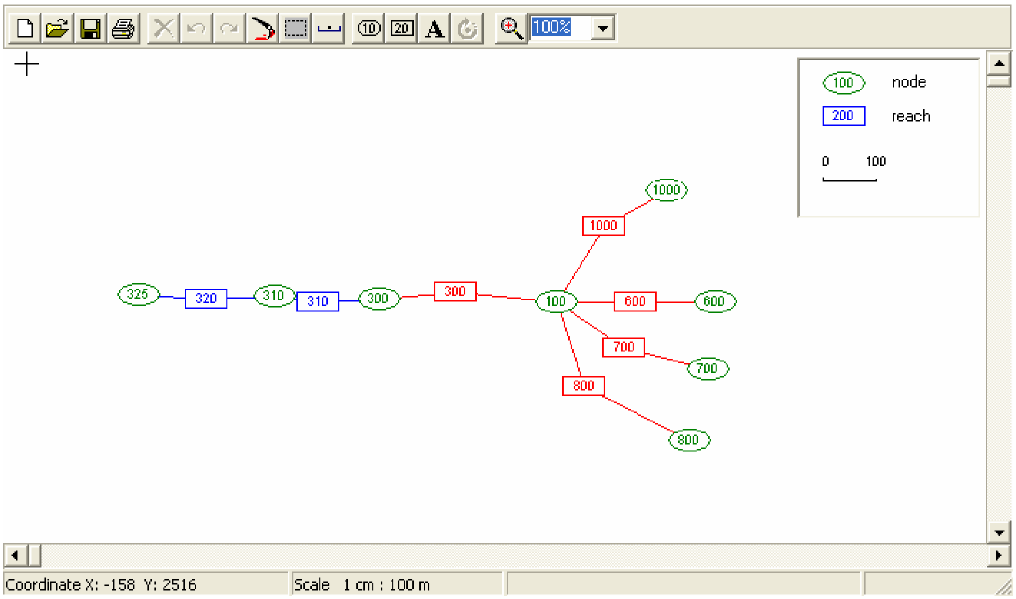

Figure 4.15 HYDRO1D network for the January 2005 flood

Structural reaches are shown as red rectangles, normal reaches by blue rectangles and nodes by green ovals. Node 100 represents the conservancy. Node 325 is the tidal boundary for Land of Canaan, and nodes, 800, 700 and 600 are the boundaries for the Maduni and Lama sluices respectively. Reach 300 represents the Land of Canaan sluice, reach 800 Maduni sluice, and reaches 700 and 600 the two Lama sluices. Reach 1000 represents a potential overspill and is set at a level of 17.92 m (58.82 ft). This level was exceeded at a number of locations along the embankment where sandbags were placed during the January 2005 flood, and is representative the section in the vicinity of Anns Grove. Although no breaching occurred in 2005, the levels do indicate that overtopping certainly did occur.

4.3.8 Simulation of the January 2005 Flood

The January 2005 flood has been simulated using the inflow hydrographs generated by the HEC-HMS model, and adopting the physical characteristics outlined above for the conservancy. The gates on the conservancy were modelled as being fully opened on the following dates:

Land of Canaan Sluice	fully opened on 7th January
Maduni Sluice	fully opened on 17th January
Lama Big Sluice	fully opened on 18th January
Lama Small Sluice	fully opened on 19th January

Figure 4.16 below shows simulated and observed water levels.

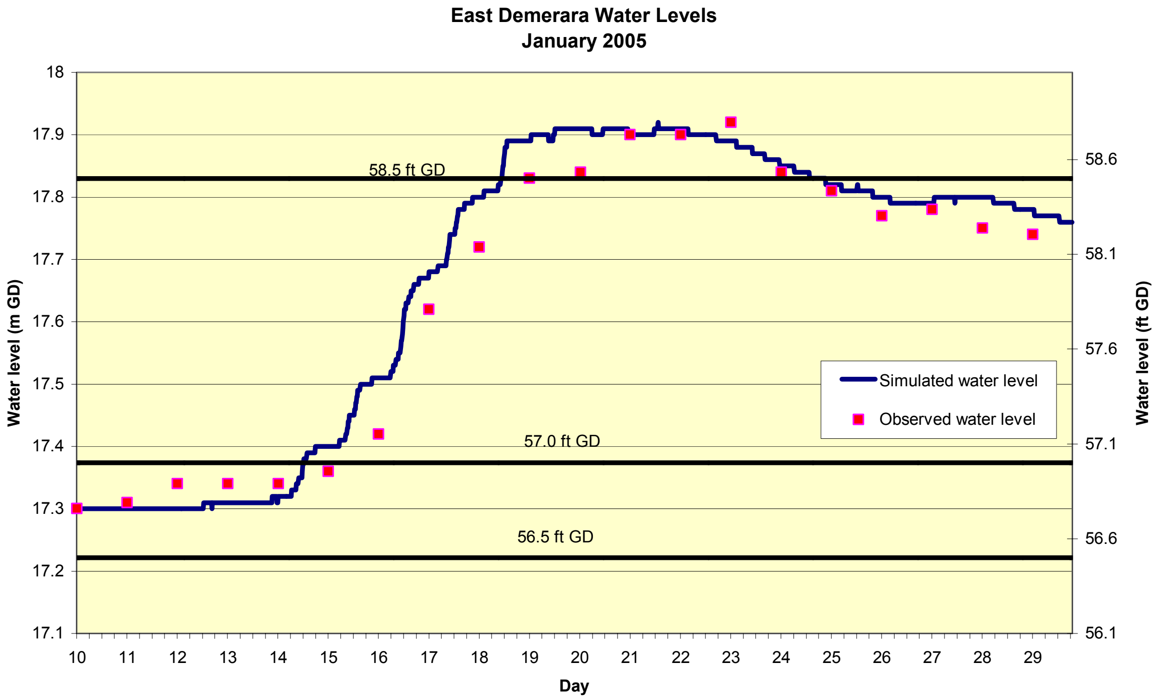

Figure 4.16 Simulated and observed water levels for the January 2005 flood.

The rise in water levels is well simulated, although the peak occurs earlier in the simulation than it does in the prototype. The peak is, however, well simulated although it should be noted that in both the model and the prototype spill has been occurring, and certainly in the case of the model, this has limited the peak water level. It is expected that the same is true of the prototype and there has been suggestion that spill was occurring around the south-eastern end of the embankment. The recession of the water level hydrograph is also well simulated and there may be reasonable confidence in the ability of the model to predict conservancy response to extreme rainfall.

4.4 Modelling the Boerasirie Conservancy

4.4.1 General

The following description of Boerasirie Conservancy draws heavily on the GDISRIP (Mott MacDonald, 2004).

The Boerasirie Conservancy has a total catchment area of some 436 km^2. This estimate has been made from 1:50,000 scale topographic mapping. The estimate of the catchment area made by Hutchinson in the 1950s was 404 km^2. Figure 4.17 shows the catchment area, and the extent of the conservancy itself. At spillway crest elevation, the area of the conservancy is some 254 km^2, and is thus over half of its own catchment area. The remaining natural catchment is heavily vegetated, and underlain by white sand deposits. Relief is very low, and the stream slope along the longest water course is of the order of 0.00023. The primary flood response will come from precipitation falling on the reservoir area itself, rather than from the natural catchment area.

The elevation-area curve for the Boerasirie Conservancy is shown in Figure 4.18, and elevation-area-storage characteristics are summarised in Table 4.6. It should be noted that these data are those prepared by Hutchinson in 1951, and no updating has been carried out since that time. This may not be of significance for flood control, as the area above the spill level is unlikely to have changed significantly since the data were originally produced. It is likely that storage characteristics at lower elevations will have changed, however, with potential implications for water resources. It is clearly unsatisfactory to be using data that are now over 50 years old in an environment that is under change. For flood control the part of the elevation-area curve of key importance is above an elevation of 18.684 m (61.3 ft), the spillway crest elevation.

Table 4.6
Elevation-area-storage characteristics for Boerasirie Conservancy

Elevation (m G.D.)	Area (km^2)	Storage (Mm^3)
17.069	20.7	4.2
17.374	44.0	15.6
17.678	88.1	34.3
17.983	132.1	65.4
18.288	191.7	113.3
18.593	235.7	178.4
18.684	253.8	220.9
18.898	269.4	283.2

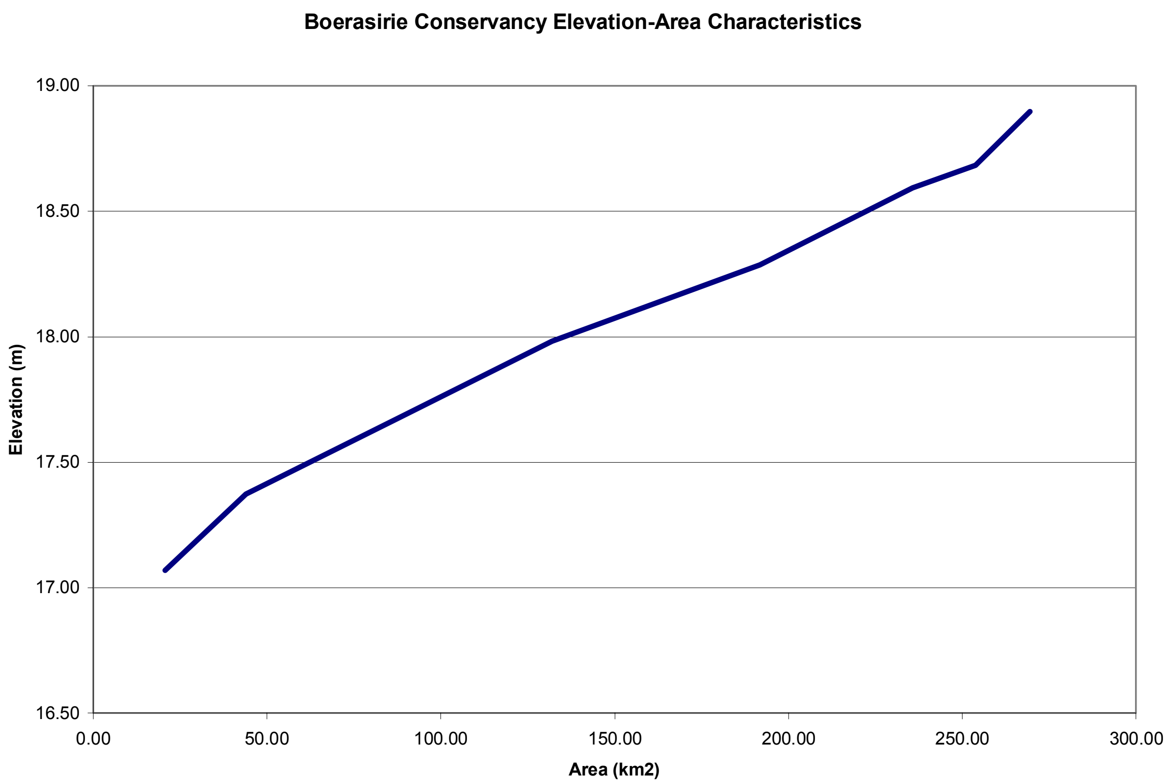

Figure 4.17 Elevation-area curve for Boerasirie Conservancy (after Hutchinson, 1951)

There are four flood relief structures on the Boerasirie conservancy. These are:

- Waramia Sluice
- The 8000 ft relief weir
- Naamryck Sluice
- Potosi Sluice

The locations of these structures are shown in Figure 4.19. All flood relief sluices have sill levels set at 16.916 m. The 8000 ft weir has a crest elevation of 18.684 m (61.3 ft), and it is when this level is reached that the flood relief sluice gates are opened.

Embankment levels around the conservancy are not consistent, and in a number of areas there is very little freeboard. The lowest point on the embankment is reportedly 18.745 m (61.5 ft), and the highest point of the order of 18.898 m (62.5 ft). At its lowest point there is only 60 mm freeboard above the spillway crest, and at its highest point only 214 mm (figures as reported by the Secretary to the Conservancy Board). The current freeboard is quite inadequate. There was extensive overtopping in January 2005, and overtopping in certain sections has been common in the past. No topographic survey exists at present for the crest of the embankment. It is essential that a survey be carried out as soon as possible.

The Conservancy Board record water levels daily at each of the above structures. The water levels recorded in January 2005 are shown in Figure 4.20.

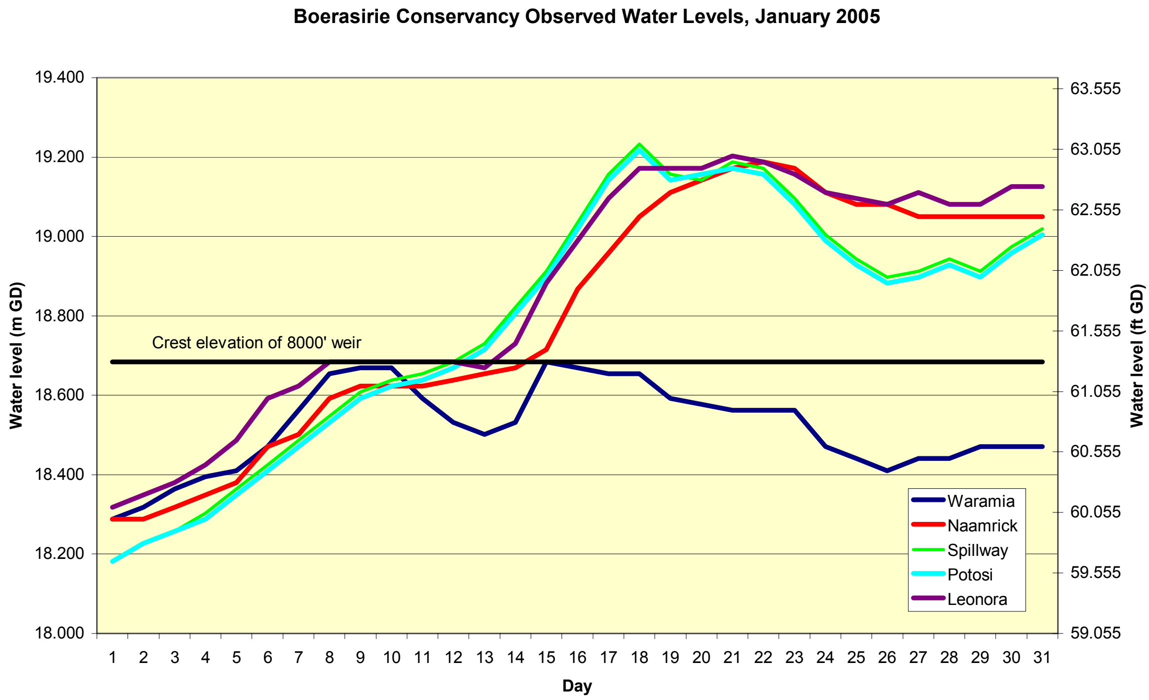

Figure 4.20 Boerasirie Conservancy water levels in January 2005

Of particular note is the significant water level difference between Naamrick and Waramia. The boundary channel between these locations is very restricted, and clearly prevents a significant body of water reaching the main flood relief works of the 8000' weir, and Waramia sluice.

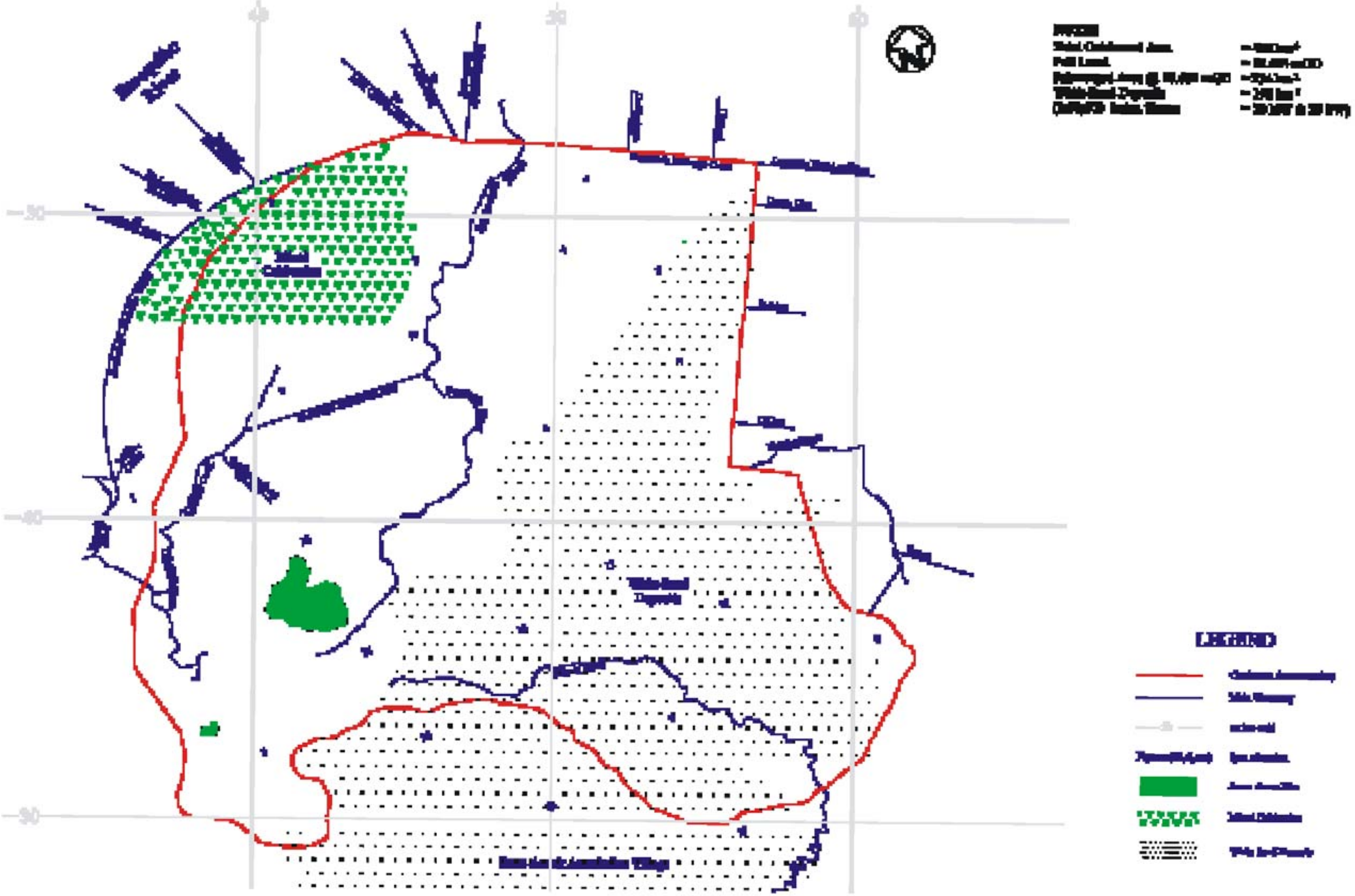

Figure 4.18 Boerasiria Conservancy catchment area

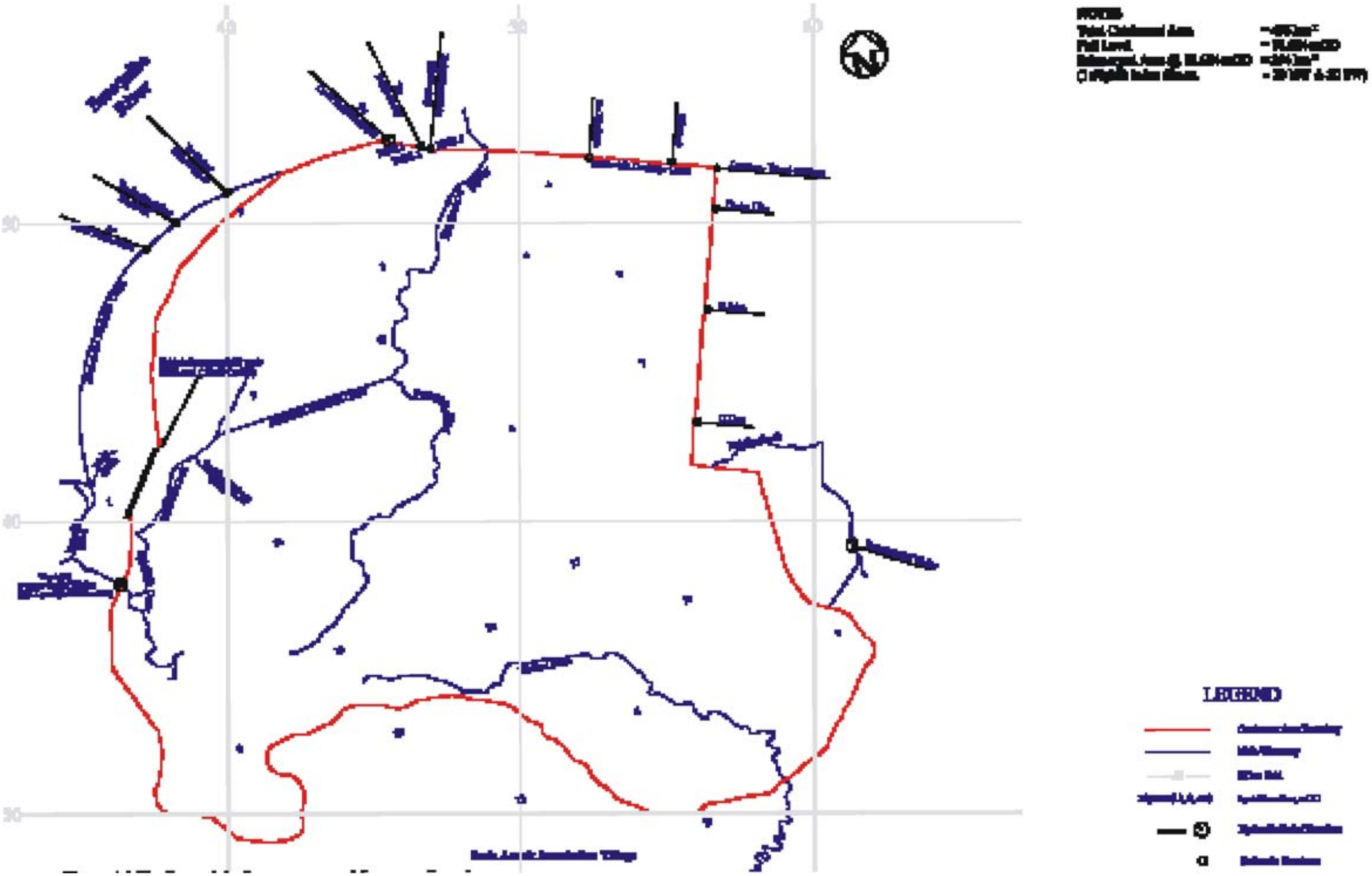

Figure 4.19 Boerasiria Conservancy structures

4.4.2 Waramia Sluice

Waramia sluice comprises 5 gates, each of which is 4.877 m wide. Figure 4.21 shows the structure viewed from downstream. The design capacity of Waramia sluice at a water level of 18.684 m is only 56.6 m^3/s (the figure on a plaque at the structure is 2000 cfs). With a free discharge through this structure, one could expect a discharge of about 90 m^3/s (3200 cfs), but there is a complicated energy dissipation system downstream and a tail weir. Discussion with conservancy staff indicates that additional backwater effects from the River Bonasika do not occur, and water marks tend to confirm this. The water marks on the structure indicate a head loss through the structure of about 0.35 m. When this head loss is used with a sluice equation (i.e. gates not fully removed) and a normal discharge coefficient, the corresponding discharge is 58.6 m^3/s (2068 cfs). It is believed that the structure has been designed on this basis.

A rating table has been prepared for Waramia sluice with coefficients derived on the basis of the design discharge. The rating table is presented in Table 4.7. It may be noted with the water levels in Figure 4.20 that Waramia Sluice was not providing significant discharge during the January 2005 flood.

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Figure 4.21 Waramia sluice from downstream

Table 4.7
Calculated discharge matrix for Waramia Sluice – coefficients based on design data

	WL d/s																
WL u/s	17.5	17.6	17.7	17.8	17.9	18	18.1	18.2	18.3	18.4	18.5	18.6	18.7	18.8	18.9	19	19.1
17.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
17.6	7.58	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
17.7	11.39	8.84	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
17.8	20.27	13.06	10.12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
17.9	23.80	17.05	14.76	11.42	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
18	27.52	27.52	19.06	16.47	12.74	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
18.1	31.42	31.42	31.42	21.08	18.20	14.06	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
18.2	35.48	35.48	35.48	25.61	23.13	19.95	15.40	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
18.3	39.70	39.70	39.70	39.70	27.91	25.19	21.72	16.76	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
18.4	44.08	44.08	44.08	44.08	44.08	30.24	27.27	23.50	18.12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
18.5	48.61	48.61	48.61	48.61	48.61	35.25	32.58	29.37	25.30	19.50	0.00	0.00	0.00	0.00	0.00	0.00	0.00
18.6	53.29	53.29	53.29	53.29	53.29	53.29	37.82	34.95	31.49	27.10	20.88	0.00	0.00	0.00	0.00	0.00	0.00
18.7	58.11	58.11	58.11	58.11	58.11	58.11	58.11	40.41	37.32	33.61	28.92	22.28	0.00	0.00	0.00	0.00	0.00
18.8	63.06	63.06	63.06	63.06	63.06	63.06	63.06	45.86	43.02	39.71	35.75	30.76	23.68	0.00	0.00	0.00	0.00
18.9	68.15	68.15	68.15	68.15	68.15	68.15	68.15	68.15	48.68	45.64	42.12	37.91	32.60	25.09	0.00	0.00	0.00
19	73.36	73.36	73.36	73.36	73.36	73.36	73.36	73.36	73.36	51.51	48.28	44.54	40.08	34.46	26.51	0.00	0.00
19.1	78.70	78.70	78.70	78.70	78.70	78.70	78.70	78.70	78.70	57.37	54.36	50.93	46.98	42.26	36.32	27.94	0.00
19.2	84.17	84.17	84.17	84.17	84.17	84.17	84.17	84.17	84.17	84.17	60.41	57.22	53.60	49.42	44.45	38.19	29.38
19.3	89.76	89.76	89.76	89.76	89.76	89.76	89.76	89.76	89.76	89.76	89.76	63.46	60.09	56.28	51.88	46.65	40.08
19.4	95.47	95.47	95.47	95.47	95.47	95.47	95.47	95.47	95.47	95.47	95.47	69.71	66.53	62.98	58.97	54.35	48.86
19.5	101.29	101.29	101.29	101.29	101.29	101.29	101.29	101.29	101.29	101.29	101.29	101.29	72.95	69.61	65.88	61.68	56.84
19.6	107.23	107.23	107.23	107.23	107.23	107.23	107.23	107.23	107.23	107.23	107.23	107.23	107.23	76.21	72.70	68.80	64.40
19.7	113.27	113.27	113.27	113.27	113.27	113.27	113.27	113.27	113.27	113.27	113.27	113.27	113.27	82.81	79.48	75.81	71.73
19.8	119.43	119.43	119.43	119.43	119.43	119.43	119.43	119.43	119.43	119.43	119.43	119.43	119.43	119.43	86.25	82.76	78.93
19.9	125.70	125.70	125.70	125.70	125.70	125.70	125.70	125.70	125.70	125.70	125.70	125.70	125.70	125.70	125.70	89.70	86.06
20	132.07	132.07	132.07	132.07	132.07	132.07	132.07	132.07	132.07	132.07	132.07	132.07	132.07	132.07	132.07	96.66	93.17

4.4.3 The 8000 ft Weir

The 8000 ft weir is 2438.4 m long, and has a crest width of 152 mm (6 inches). A photograph of the weir is shown in Figure 4.22.

Figure 4.22 The 8000 ft weir

There is significant weed growth on the upstream face of the weir that will influence its hydraulic performance. There is also significant vegetation growth on the downstream side of the weir that could restrict its performance at higher discharges. At a maximum head over the weir of 215 mm, the head to width ratio is 1.4, and the expectation is therefore that the weir behaves as a broad crested weir throughout this range (USGS, 1968). The distinction between broad crested and sharp crested behaviour is a very important one. For a broadcrested weir operating under ideal conditions,

$$q = 1.7 \times h^{1.5} \qquad \ldots\ldots\ldots..1$$

and for a sharp crested weir under ideal conditions,

$$q = 2.95 \times h^{1.5} \qquad \ldots\ldots\ldots..2$$

where, q is the discharge per unit width (m^3/s) and h is the total head available over the crest (m).

Sharp crested behaviour is approached when the head to width ratio exceeds 1.5. For a sharp crested weir, the discharge coefficient is influenced by the head to depth ratio upstream of the weir, and actual behaviour varies significantly from that given by equation 2. For the head to depth ratios existing upstream of the 8000 ft weir, were it possible to apply a sharp crested equation, the coefficient of discharge C_d would be 1.82.

The coefficient of discharge for a broad crested weir increases as the head to width ratio increases. At the lowest head to width ratio (i.e., when the weir just starts to spill), the coefficient of discharge would be 1.45, and with a ratio of 1.4 (i.e. a head over the crest of 215 mm), the coefficient of discharge would be 1.78, which is approaching the lower end of sharp crested behaviour (USGS, 1968). The discharge coefficient for the 8000 ft weir would, under conditions of no weed growth upstream of the weir, vary with head in the range of 1.45 – 1.78.

A discharge table has been prepared for the 8000 ft weir, using a discharge coefficient of 1.45, and ignoring the influence of vegetation upstream and downstream of the weir. Table 4.8 below presents the discharge matrix. Taking the average water level between Naamrick and Waramia as being representative of an average water level over the 8000 ft weir, a discharge well in excess of 300 m^3/s should have been possible during the January 2005 flood. It is very unlikely that the actual discharge got anywhere close to this figure.

Table 4.8
Calculated discharge matrix for the 8000 ft weir

WL u/s	WL d/s 17.5	17.6	17.7	17.8	17.9	18	18.1	18.2	18.3	18.4	18.5	18.6	18.7	18.8	18.9	19	19.1
17.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
17.6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
17.7	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
17.8	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
17.9	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
18.1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
18.2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
18.3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
18.4	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
18.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
18.6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
18.7	7.16	7.16	7.16	7.16	7.16	7.16	7.16	7.16	7.16	7.16	7.16	7.16	0.00	0.00	0.00	0.00	0.00
18.8	139.69	139.69	139.69	139.69	139.69	139.69	139.69	139.69	139.69	139.69	139.69	139.69	139.69	0.00	0.00	0.00	0.00
18.9	354.94	354.94	354.94	354.94	354.94	354.94	354.94	354.94	354.94	354.94	354.94	354.94	354.94	354.94	0.00	0.00	0.00
19	628.06	628.06	628.06	628.06	628.06	628.06	628.06	628.06	628.06	628.06	628.06	628.06	628.06	628.06	455.80	0.00	0.00
19.1	948.67	948.67	948.67	948.67	948.67	948.67	948.67	948.67	948.67	948.67	948.67	948.67	948.67	948.67	948.67	624.77	0.00
19.2	1310.53	1310.53	1310.53	1310.53	1310.53	1310.53	1310.53	1310.53	1310.53	1310.53	1310.53	1310.53	1310.53	1310.53	1310.53	1310.53	798.49
19.3	1709.40	1709.40	1709.40	1709.40	1709.40	1709.40	1709.40	1709.40	1709.40	1709.40	1709.40	1709.40	1709.40	1709.40	1709.40	1709.40	1251.62
19.4	2142.11	2142.11	2142.11	2142.11	2142.11	2142.11	2142.11	2142.11	2142.11	2142.11	2142.11	2142.11	2142.11	2142.11	2142.11	2142.11	2142.11
19.5	2606.20	2606.20	2606.20	2606.20	2606.20	2606.20	2606.20	2606.20	2606.20	2606.20	2606.20	2606.20	2606.20	2606.20	2606.20	2606.20	2606.20
19.6	3099.68	3099.68	3099.68	3099.68	3099.68	3099.68	3099.68	3099.68	3099.68	3099.68	3099.68	3099.68	3099.68	3099.68	3099.68	3099.68	3099.68
19.7	3620.87	3620.87	3620.87	3620.87	3620.87	3620.87	3620.87	3620.87	3620.87	3620.87	3620.87	3620.87	3620.87	3620.87	3620.87	3620.87	3620.87
19.8	4168.40	4168.40	4168.40	4168.40	4168.40	4168.40	4168.40	4168.40	4168.40	4168.40	4168.40	4168.40	4168.40	4168.40	4168.40	4168.40	4168.40
19.9	4741.04	4741.04	4741.04	4741.04	4741.04	4741.04	4741.04	4741.04	4741.04	4741.04	4741.04	4741.04	4741.04	4741.04	4741.04	4741.04	4741.04
20	5337.73	5337.73	5337.73	5337.73	5337.73	5337.73	5337.73	5337.73	5337.73	5337.73	5337.73	5337.73	5337.73	5337.73	5337.73	5337.73	5337.73

4.4.4 Naamryck Sluice

Naamryck sluice comprises a single gate of width 4.572 m, with the sill set at 16.916 m. A photograph of the structure is shown in Figure 4.23, and of the channel downstream in Figure 4.24. Apparently when this structure is discharging at full capacity, there are complaints from farmers downstream about backing up through their drains. However, the consequences of an embankment failure would clearly be worse than any damage caused by backing up from the channel. The gates at this structure can be lifted clear of the upstream water level, and there are apparently no backwater influences from the discharge channel, although the tidal gates at the downstream end are thought to be in poor repair. A discharge matrix has been prepared for the structure and is presented in Table 11 below. There is no significant capacity at Naamrick.

Table 4.9
Calculated discharge matrix for the Naamrick Sluice

WL u/s	WL d/s 17.5	17.6	17.7	17.8	17.9	18	18.1	18.2	18.3	18.4	18.5	18.6	18.7	18.8	18.9	19	19.1
17.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
17.6	2.34	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
17.7	3.52	2.74	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
17.8	6.27	4.04	3.13	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
17.9	7.36	5.27	4.56	3.53	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
18	8.51	8.51	5.90	5.10	3.94	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
18.1	9.72	9.72	9.72	6.52	5.63	4.35	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
18.2	10.98	10.98	10.98	7.92	7.16	6.17	4.77	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
18.3	12.28	12.28	12.28	12.28	8.64	7.79	6.72	5.18	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
18.4	13.64	13.64	13.64	13.64	13.64	9.36	8.44	7.27	5.61	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
18.5	15.04	15.04	15.04	15.04	15.04	10.90	10.08	9.09	7.83	6.03	0.00	0.00	0.00	0.00	0.00	0.00	0.00
18.6	16.49	16.49	16.49	16.49	16.49	16.49	11.70	10.81	9.74	8.39	6.46	0.00	0.00	0.00	0.00	0.00	0.00
18.7	17.98	17.98	17.98	17.98	17.98	17.98	17.98	12.50	11.55	10.40	8.95	6.89	0.00	0.00	0.00	0.00	0.00
18.8	19.51	19.51	19.51	19.51	19.51	19.51	19.51	14.19	13.31	12.29	11.06	9.52	7.33	0.00	0.00	0.00	0.00
18.9	21.08	21.08	21.08	21.08	21.08	21.08	21.08	21.08	15.06	14.12	13.03	11.73	10.09	7.76	0.00	0.00	0.00
19	22.70	22.70	22.70	22.70	22.70	22.70	22.70	22.70	22.70	15.93	14.94	13.78	12.40	10.66	8.20	0.00	0.00
19.1	24.35	24.35	24.35	24.35	24.35	24.35	24.35	24.35	24.35	17.75	16.82	15.76	14.53	13.07	11.24	8.64	0.00
19.2	26.04	26.04	26.04	26.04	26.04	26.04	26.04	26.04	26.04	26.04	18.69	17.70	16.58	15.29	13.75	11.82	9.09
19.3	27.77	27.77	27.77	27.77	27.77	27.77	27.77	27.77	27.77	27.77	27.77	19.63	18.59	17.41	16.05	14.43	12.40
19.4	29.53	29.53	29.53	29.53	29.53	29.53	29.53	29.53	29.53	29.53	29.53	21.56	20.58	19.48	18.24	16.82	15.12
19.5	31.33	31.33	31.33	31.33	31.33	31.33	31.33	31.33	31.33	31.33	31.33	31.33	22.57	21.53	20.38	19.08	17.58
19.6	33.17	33.17	33.17	33.17	33.17	33.17	33.17	33.17	33.17	33.17	33.17	33.17	33.17	23.58	22.49	21.28	19.92
19.7	35.04	35.04	35.04	35.04	35.04	35.04	35.04	35.04	35.04	35.04	35.04	35.04	35.04	25.62	24.59	23.45	22.19
19.8	36.95	36.95	36.95	36.95	36.95	36.95	36.95	36.95	36.95	36.95	36.95	36.95	36.95	36.95	26.68	25.60	24.42
19.9	38.89	38.89	38.89	38.89	38.89	38.89	38.89	38.89	38.89	38.89	38.89	38.89	38.89	38.89	38.89	27.75	26.62
20	40.86	40.86	40.86	40.86	40.86	40.86	40.86	40.86	40.86	40.86	40.86	40.86	40.86	40.86	40.86	29.90	28.82

4.4.5 Potosi Sluice

Potosi sluice has the same characteristics as Naamryck sluice. There are constraints posed by the downstream channel. Discharge characteristics have been assumed to be the same as at Naamryck, however, with structure functioning normally.

Figure 4.23 Naamryck sluice

Figure 4.24 Channel downstream of Naamryck Sluice

4.4.5 Tidal Boundary Conditions

Boerasirie Conservancy is at a higher level than EDC, and it is not thought that tidal variations affect the discharge capacity of any of the structures. However, for modelling purposes, Georgetown tidal profiles have been used for downstream boundary conditions. They have no effect on any of the discharge matrices, but are required for modelling completeness.

4.4.6 The HYDRO1D Model Setup

The HYDRO1D model network for Boerasirie Conservancy is shown in Figure 4.25 below. The network is very simple. The conservancy is represented by storage node 100. Reaches 200, 300, 400 and 500 represent Potosi sluice, Naamrick sluice, the 8000 ft weir, and Waramia sluice respectively.

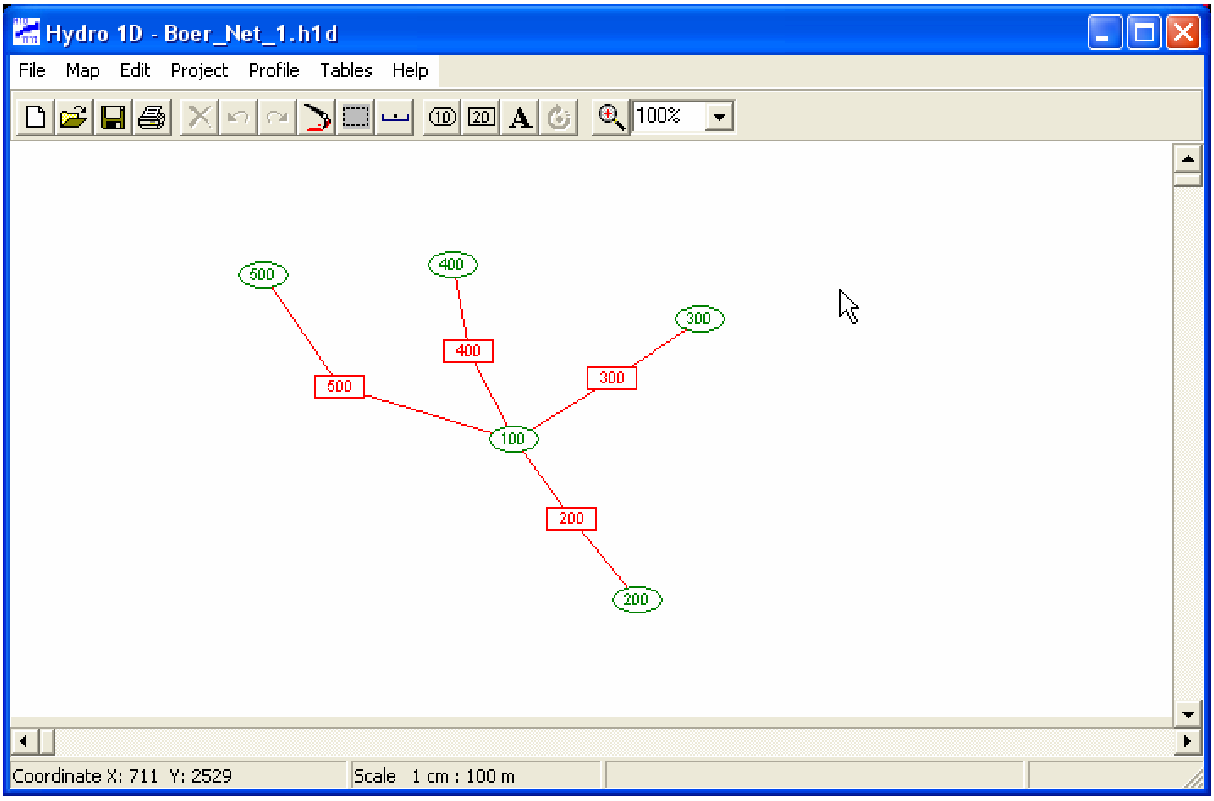


Figure 4.25 The HYDRO1D model setup for Boerasirie Conservancy.

4.4.7 Simulation of the January 2005 Flood

The January 2005 flood in Boerasirie conservancy has been simulated using inflow hydrographs generated through the HEC-HMS model, and adopting the discharge matrixes given in the preceding section for each of the structures. The simulated and observed water levels for the January 2005 flood are presented in Figure 4.26.

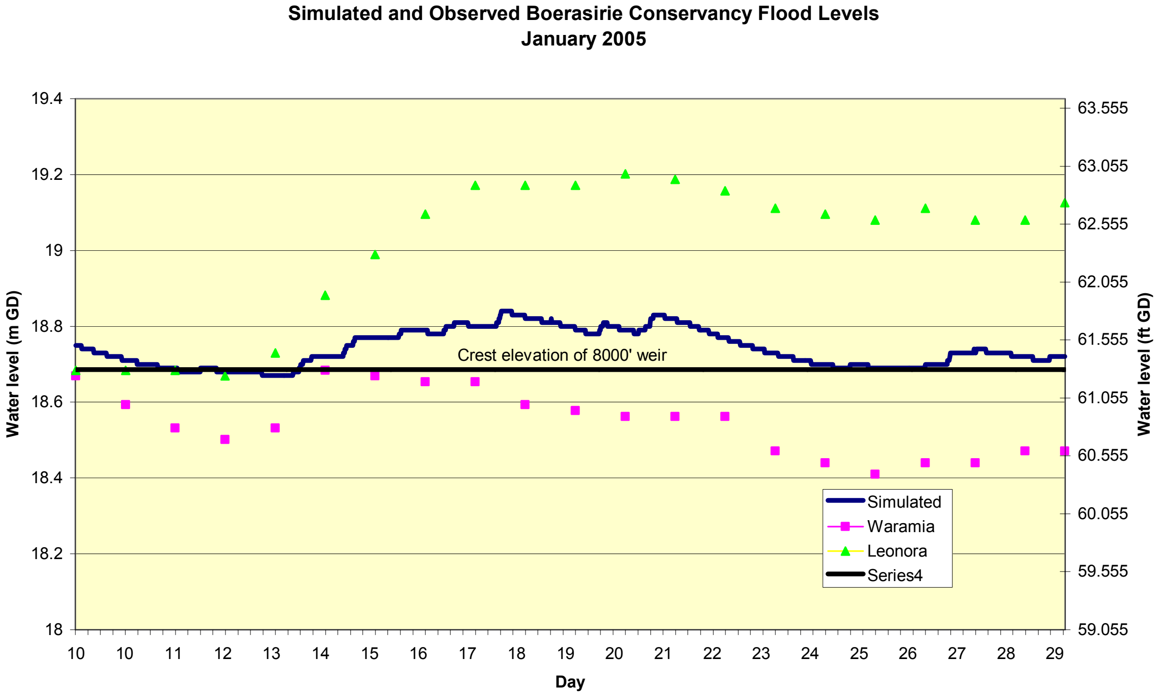


Figure 4.26 Simulated and observed Boerasirie Conservancy water levels for January 2005

In view of the water surface gradients that existed across the conservancy during the 2005 flood, there is no possibility of modelling the observed behaviour with a model that assumes level pool behaviour, and that assumes free flow over a the 8000 ft weir. The indications from Figure 4.26 are that if the conservancy was behaving as a level pool, and if the conditions at the 8000 ft weir were close to design assumptions, then there would not be problems of over-topping of the conservancy embankments.

It is clear that the waterway between Naamrick and the 8000 ft weir and Waramia needs to be significantly improved to permit adequate flow to the main flood relief structure that is the 8000 ft weir. A photograph of part of the waterway is shown in Figure 4.27.

In order to properly determine the required sizes to which waterways are excavated and maintained within the conservancy, it is strongly recommended that a pseudo two-dimensional hydraulic model be developed for the conservancy. Construction of this model will require hydrographic survey of the conservancy. Hydrographic survey of the conservancy is a priority need.

Figure 4.27 Part of the waterway on Boerasirie conservancy between Naamrick and Waramia

5. Operational Flood Management of the EDC

5.1 Post-Emergency Works Situation

As has been indicated in earlier sections, the disused sluice structures at Kofi and at Cunha are being brought back into operation. These structures, along with representation of their outfall channels and structures at the Demerara river have been included in the model. The model network with the additional structures included is shown in Figure 5.1.

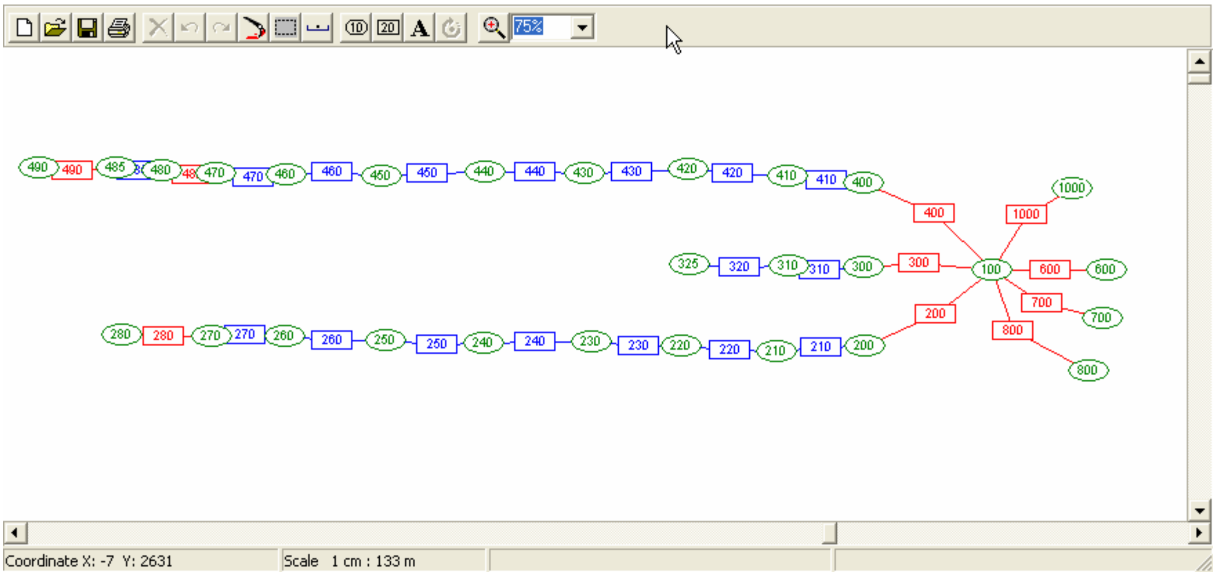


Figure 5.1 The HYDRO1D network with the Kofi and Cunha outfall channels and their respective sluices included.

A preliminary model run was carried out with the post-emergency network using the January 2005 flood to assess how the conservancy would have responded, had these two structures been opened at the same time as the Land of Canaan sluice. The resulting water level hydrographs are shown in Figure 5.2. Had the Kofi and Cunha sluices been operational, then the peak water level during the January 2005 flood could have been reduced by about 90 mm. this reduction is sufficient, in model terms, to eliminate any spill from the conservancy. It would not, however, have been sufficient to bring the peak water level below the level of 17.83 m (58.5 ft), which has been indicated to be the maximum safe operating level. If the embankment is formed uniformly to a level of 18.29 m (60.0 ft) then a maximum water level of 17.83 m provides freeboard of 460 mm (1.5 ft). It is considered desirable that outlet works capacities and operating levels are developed to ensure that the 2005 flood could have been passed within this maximum level of 17.83 m (58.5 ft).

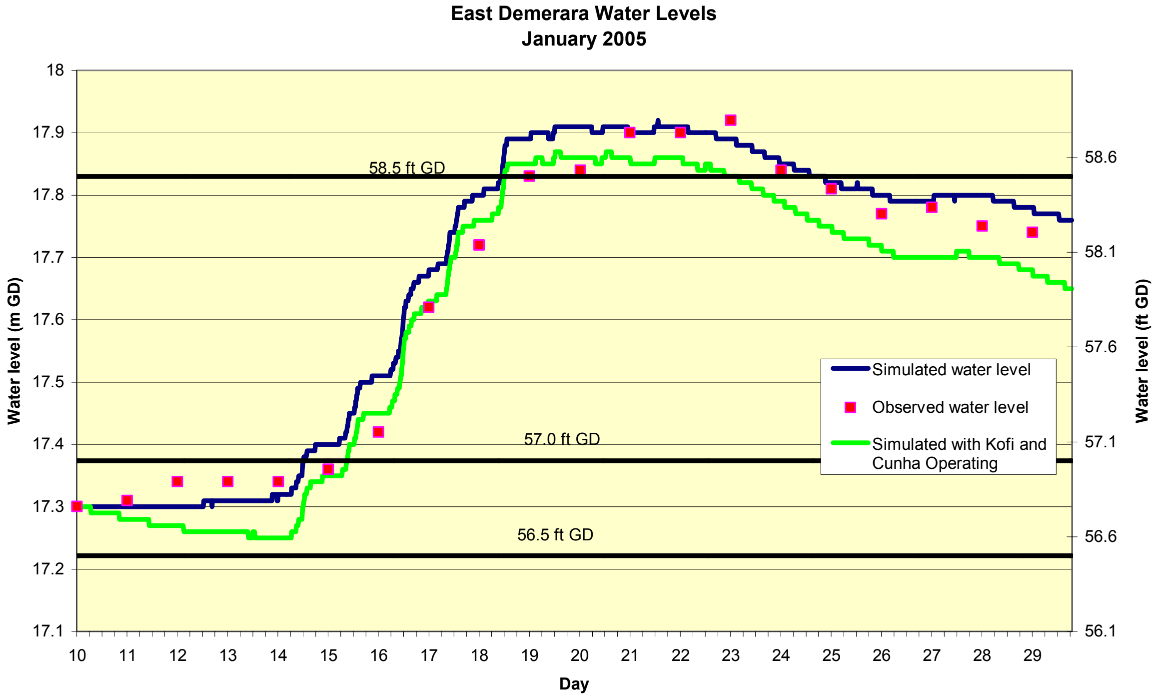


Figure 5.1 Simulated EDC water levels for the 2005 flood showing the influence that the Kofi and Cunha sluices could have had on peak levels.

5.2 Assessment of the Impact of Storm Duration

An assessment has been made of the water level profiles associated with storm durations of 5, 7 and 10 days at return periods of 100, 200, 500, 1000 and 10000 years, assuming that all gates were opened at a level of 17.37 m (57 ft). The results are shown in Figures 5.2, 5.3 and 5.4 for durations of 5, 7 and 10 days respectively.

It is clear that the highest flood levels are associated with the 10-day duration storm, and in subsequent production runs with the model, only the 10-day duration was used. It is apparent that if all gates were opened at a level of 17.73 m, then floods of up to 1,000 year return period could be kept within desired upper level limit for the conservancy. The 10,000 year flood could not, however, and it should be noted that the model has in fact permitted some spill above a level of 17.92 m. It is desirable that even the 10,000 year event be kept with the desired upper level of 17.83 m (58.5 ft). The model was not re-run to determine what the maximum level with the 10,000 year flood would be assuming no spill, as further runs have been carried out to determine the requirements to provide safe passage of the 10,000 year flood.

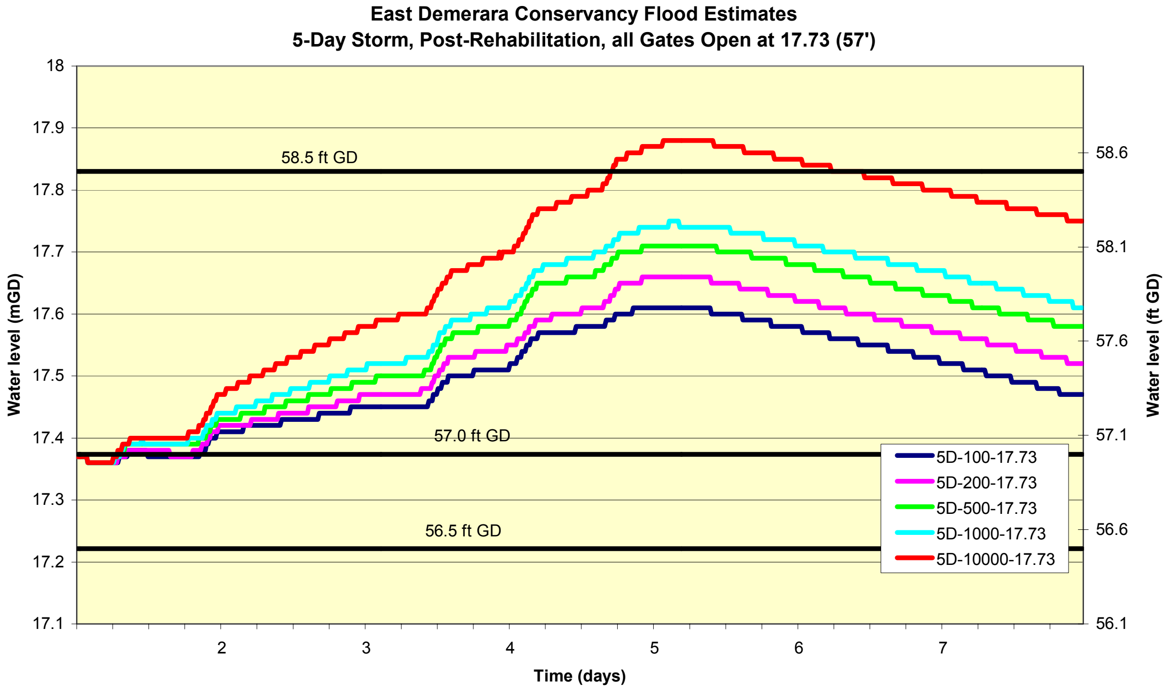


Figure 5.2 Response of EDC to a 5-day storm with different return periods, and assuming that all gates, including Kofi and Cunha are opened at 17.37 m.

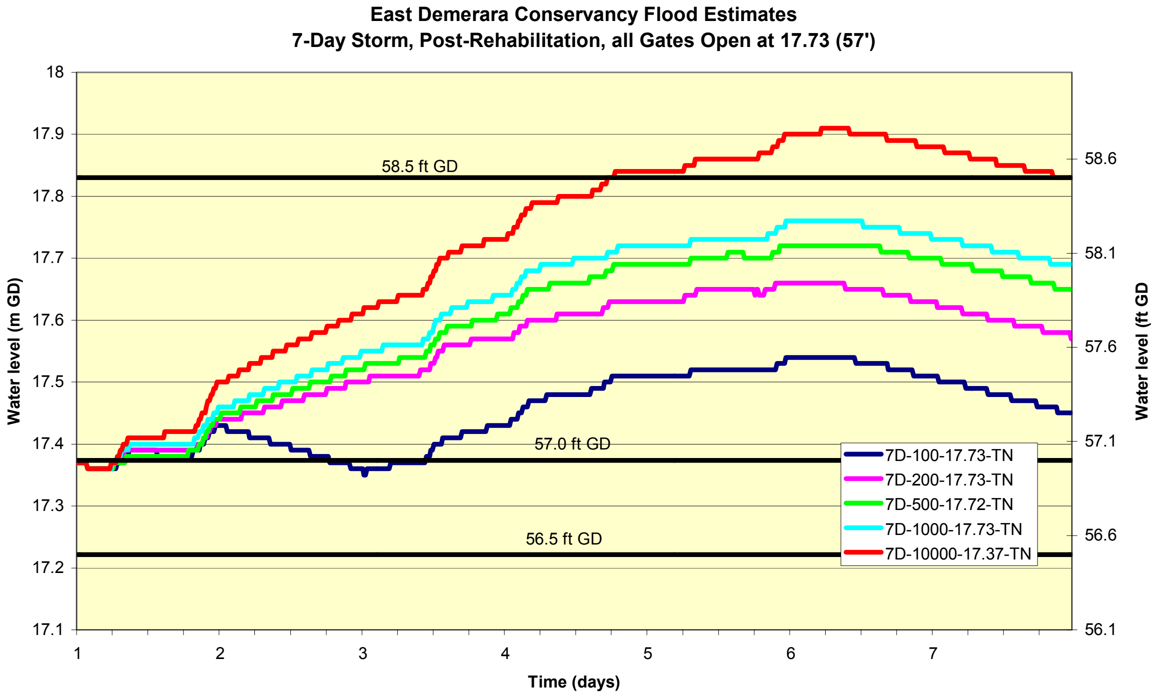


Figure 5.3 Response of EDC to a 7-day storm with different return periods, and assuming that all gates, including Kofi and Cunha are opened at 17.37 m.

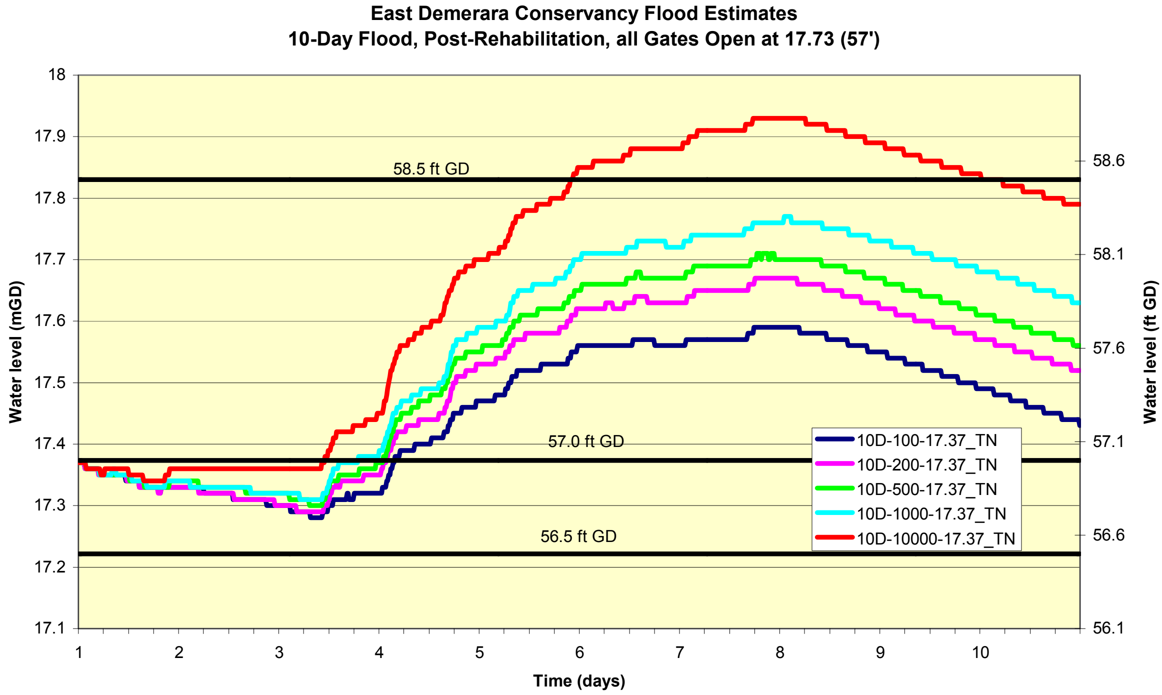


Figure 5.4 Response of EDC to a 10-day storm with different return periods, and assuming that all gates, including Kofi and Cunha are opened at 17.37 m.

5.2 Evaluation of SEEC Committee Schedule for Gate Operations

A meeting was held with SEEC committee on 10th May 2005, at which a schedule of gate operation levels was produced with the request that these be modelled for the conditions of Kofi and Cunha in operation, and Kofi and Cunha not in operation. The levels at which SEEC propose that gates be operated are given in Table 5.1 below.

Table 5.1
SEEC Committee schedule for gate operations

Water level at Maduni (ft GD)	**Gate Status**					
	Land of Canaan	**Maduni**	**Lama Big**	**Lama Small**	**Cunha**	**Kofi**
55.50	Closed	Closed	Closed	Closed	Closed	Closed
56.00	**Open**	Closed	Closed	Closed	Closed	Closed
56.25	**Open**	**Open**	Closed	Closed	**Open**	**Open**
57.00	**Open**	**Open**	Closed	Closed	**Open**	**Open**
57.50	**Open**	**Open**	Closed	Closed	**Open**	**Open**
58.00	**Open**	**Open**	**Open**	Closed	**Open**	**Open**
58.25	**Open**	**Open**	**Open**	**Open**	**Open**	**Open**

A series of model runs has been set up starting with the SEEC schedule given above. The runs were carried out for storm durations of 5, 7 and 10 days, and for return periods of 100, 200, 500, 1000 and 10000 years. The naming convention used for the production runs comprises three parts: 5D, 7D or 10D, indicating duration; A, B, C, D or E, representing return periods or 100, 200, 500, 1000, and 10000 years; and the letters SEEC, SEEC2, SEEC3 etc. representing further variations that are described below. In fact only a 10-day duration was considered for the SEEC series of runs. The TN in run references refers to Timehri neap tides, which are likely to provide conservative drainage conditions.

The simulated water levels following the SEEC schedule of gate openings given in Table 5.1 is presented in Figures 5.5.

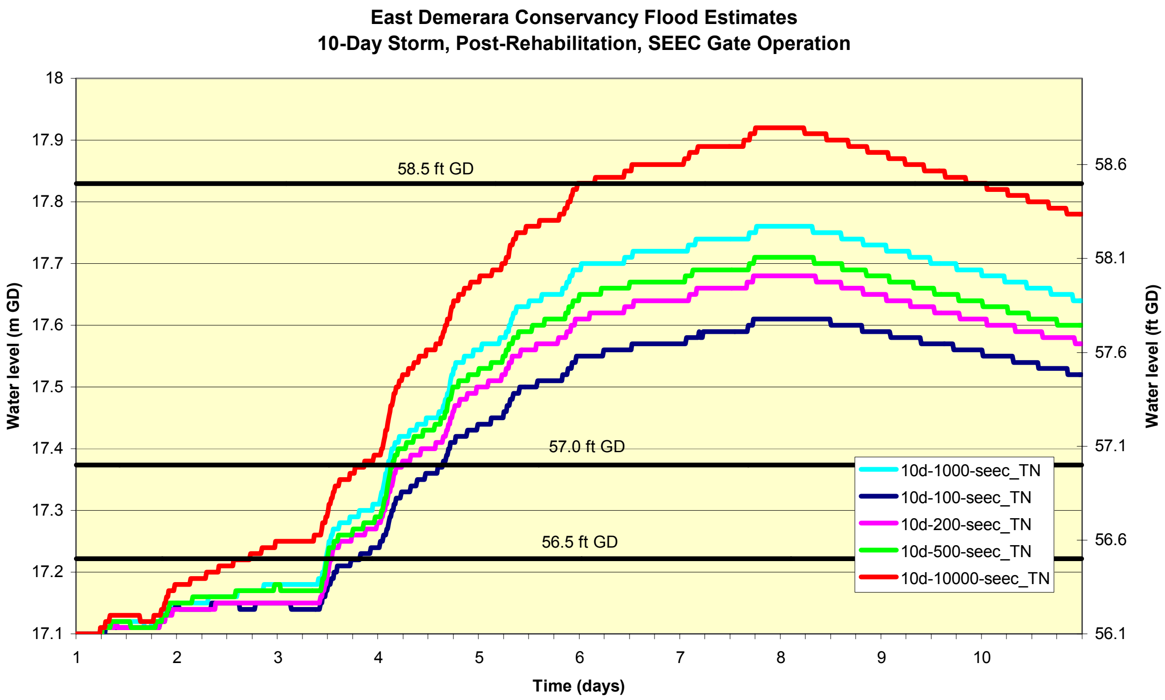


Figure 5.5 Response of EDC to a 10-day storm with different return periods, and assuming that gates are operated according to the schedule given in Table 8

With the SEEC schedule, storms of up to 1000 years return period could be passed through the conservancy without encroaching upon the freeboard. The 10000 year flood could not be accommodated however.

SEEC had requested that their gate schedule also be tested for the case of Kofi and Cunha sluices not operating. The results for these runs are shown in Figure 5.6. In this case the 1000 year event would still be held below the freeboard limit, but the 10,000 year event would take levels significantly above the freeboard limit. Note that the levels indicated for the 10,000 year event were computed assuming that spill occurs at 17.92 m. Following embankment repair, levels would in fact be higher than those indicated in Figure 5.6.

A further variation of the runs represented by Figure 5.6, was to have gates operated as outlined in Table 5.1, but delaying the opening of Land of Canaan sluice to a level of 17.22 m (56.5 ft). The results of these runs are presented in Figure 5.7. Again the 1000 year event will be retained below the freeboard limit, but the 10,000 year even would encroach on freeboard by a significant margin.

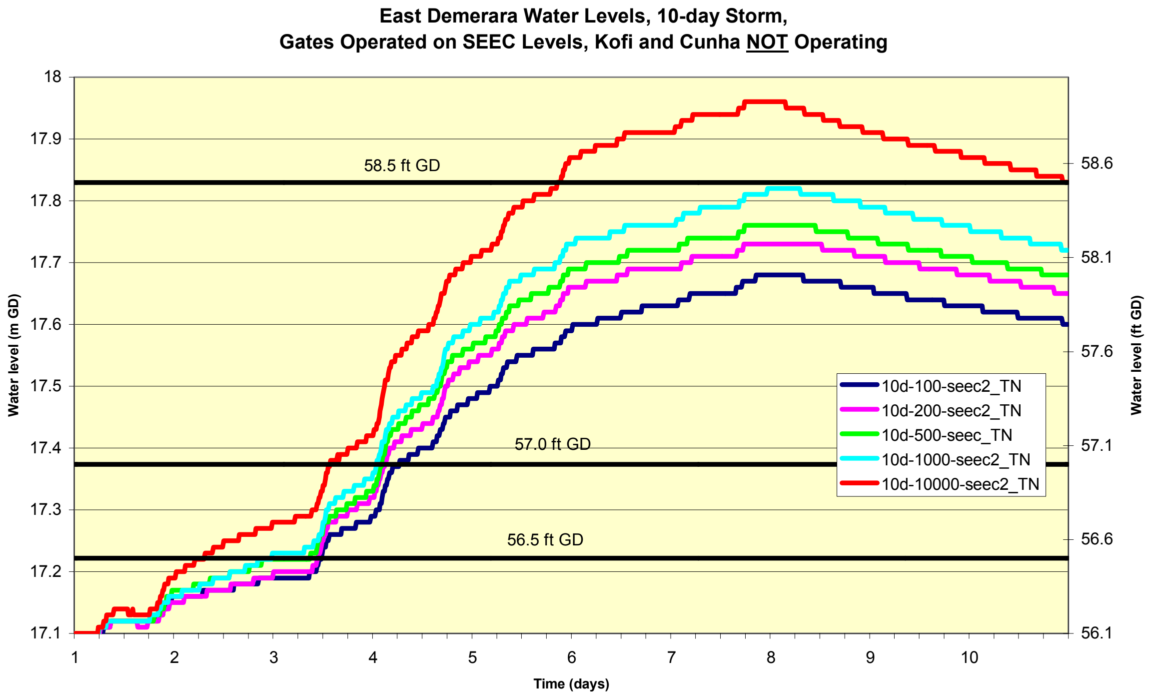


Figure 5.6 Response of EDC to a 10-day storm with different return periods, and assuming that gates are operated according to the schedule given in Table 8, but without Kofi and Cunha. (N.B. 10,000 year levels with spill at 17.92 m)

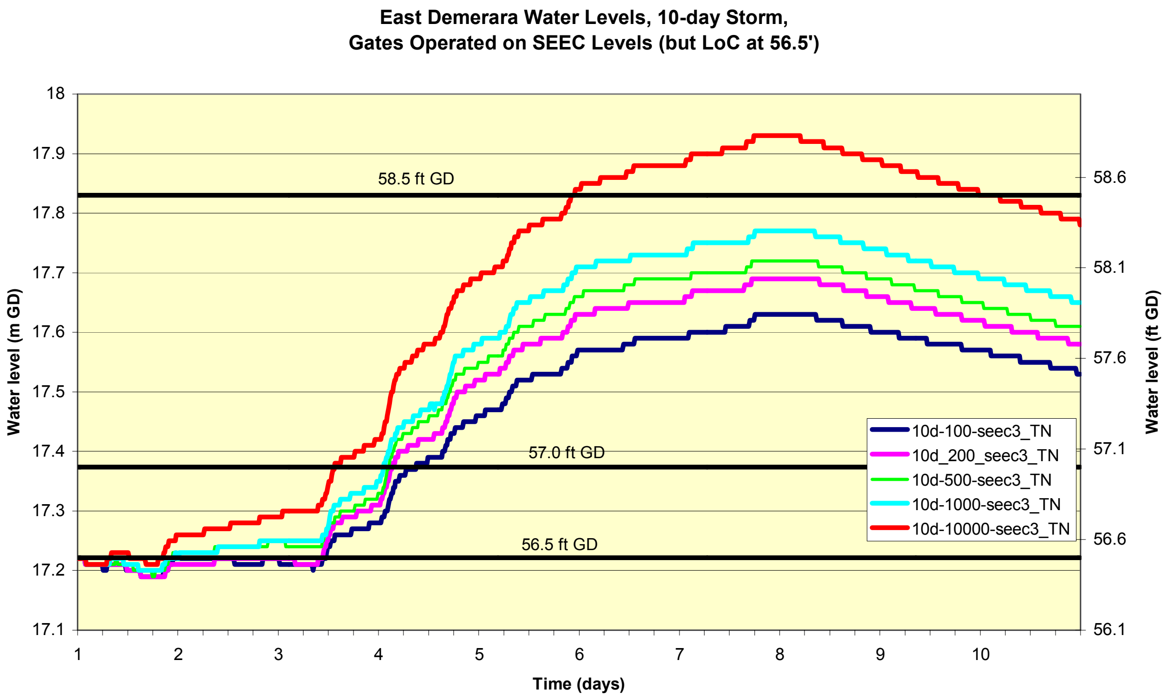


Figure 5.7 Response of EDC to a 10-day storm with different return periods, and assuming that gates are operated according to the schedule given in Table 8, but with Land of Canaan starting at 17.22. (N.B. 10,000 year levels with spill at 17.92 m)

Further production runs were carried out using the 1000 year event. The runs carried out were as follows:

SEEC4	Land of Canaan, Maduni, Kofi and Cunha opened at 17.37 m (57 ft).
SEEC5	Land of Canaan, Maduni, Kofi and Cunha opened at 17.22 m (56.5 ft) and Lama sluices kept closed.
SEEC6	Assumed Land of Canaan outfall channel increased in width by 10 m.
SEEC7	Kofi and Cunha outfall channels increased in width by 10 m.

The results of these runs are shown in Figure 5.8. Increasing the Land of Canaan outfall channel width had no significant effect (note that the modelled channel probably does not match with that existing at present as no survey was available).

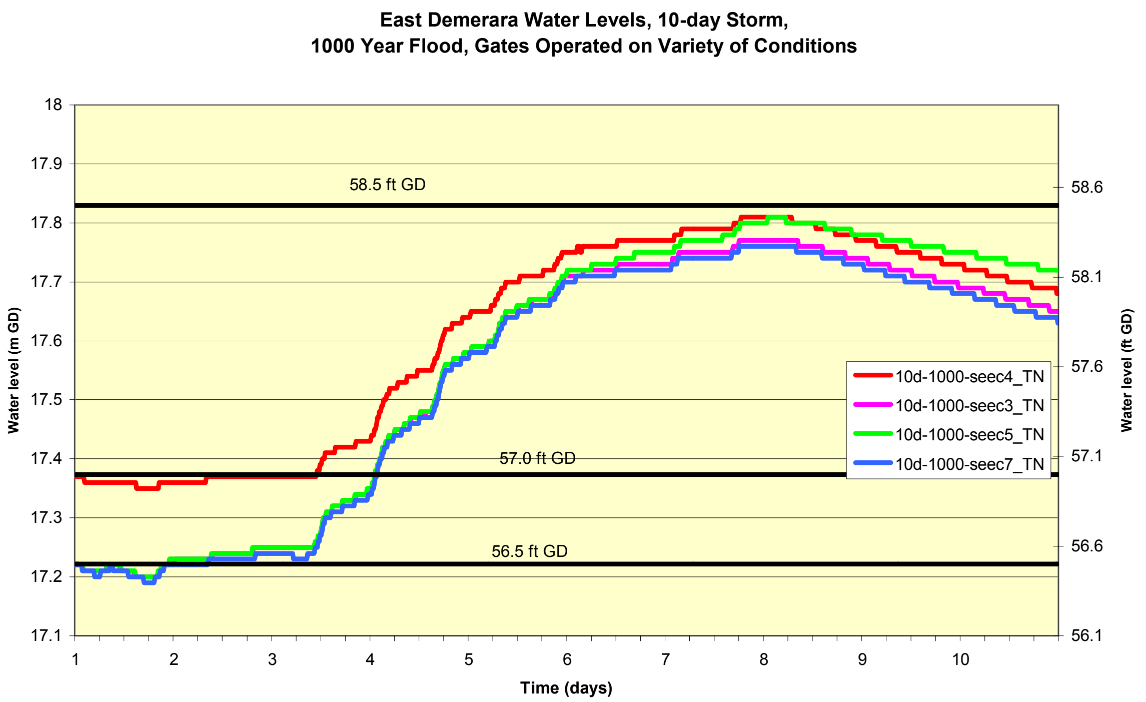


Figure 5.8 Response of EDC to a 10-day storm, 1000 year return period, and assuming that gates are operated according to the schedule given in Table 8, but with a range of operational criteria as outlined above.

With all gates operational, the 1000 year flood can be accommodated even with a starting level as high as 17.37 m (57 ft) (SEEC4). With Lama sluices closed, the 1000 year flood can be accommodated if the other sluices are operated from a level 17.22 m (56.5 ft) (SEEC5). Enlarging the outfall channels of the Kofi and Cunha sluices does not have a significant impact on peak levels, but does help (SEEC7). The SEEC3 run is included above as a frame of reference back to Figure 5.7, from which it is observed that the 10,000 year flood which cannot be kept below freeboard.

5.3 Accommodating the 10,000 Year Flood

Further model runs were carried out in order to determine what additional sluice gate capacity would be required to accommodate the 10,000 year flood without encroaching on freeboard. An additional structure was introduced with a total gated width of 30 m, and with a sill set at the same level as that at Land of Canaan. Kofi, Cunha and Land of Canaan sluices, and the modelled additional sluice, were set to be opened at a level of 17.22 m (56.5 ft), with the Lama sluices being opened at the levels indicated in Table 5.1. The results of this run are presented in Figure 5.9 below. The introduction of another sluice with a minimum of 30 m gated width would permit water levels during a 10,000 year flood to be accommodated without encroaching on freeboard. Construction of such a structure ought to be given the highest priority. Further modelling work could be carried out to refine dimensions, and possible to remove the need for operation of the Lama sluices at all.

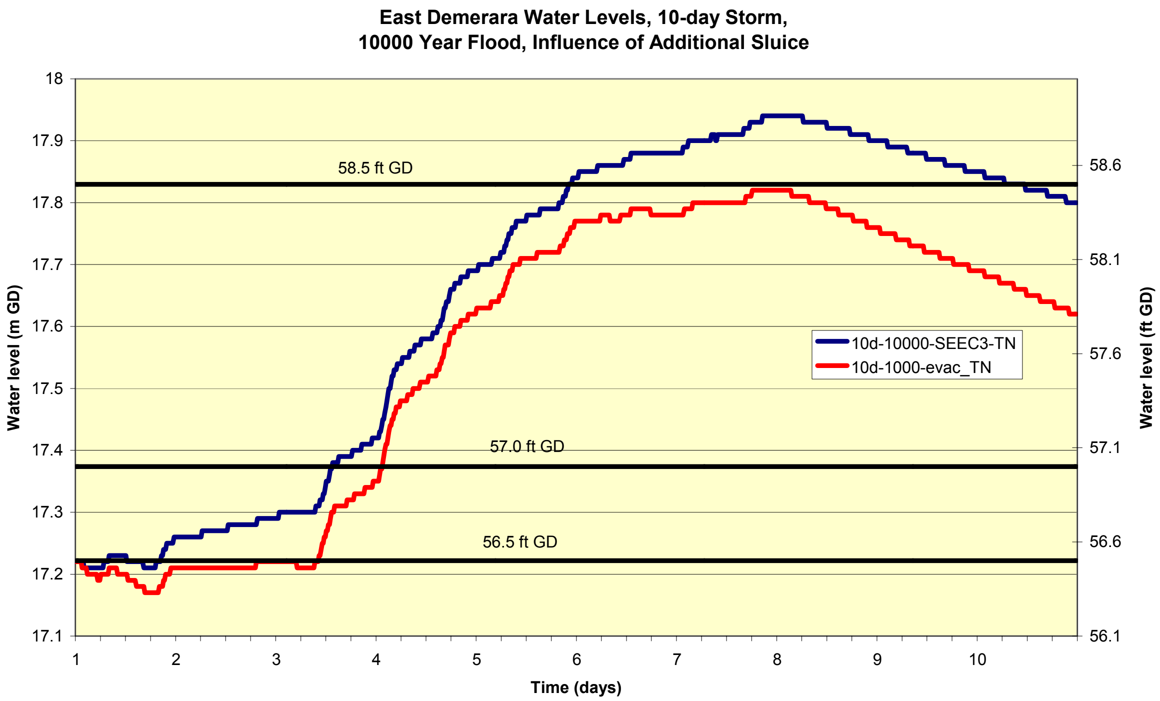


Figure 5.9 Influence of an addition sluice with a gated width of 30 m on water levels during a 10,000 year event.

5.4 Impact on Water Resources of Lower Conservancy Water Levels

A monthly catchment model and reservoir yield assessment model was set up as part of the GDISRP. That model indicated that the reliability of water supply from the East Demerara conservancy was low, and this is confirmed by the frequent use of pumps to bring water in from the Mahaica River.

The model has been re-run to assess the impact of adopting different top water levels for the conservancy. Runs were carried out on the basis of a rainfall year equalled or exceeded in 80% of years, and with irrigation demands calculated on the basis of reduced navigation losses. Navigation losses were assumed to be cut by 30%, and details of the irrigation demand calculations may be found in the GDISRP Hydrology and Water Resources report (Mott MacDonald, 2004). Figure 5.10 shows simulated conservancy water levels, assuming maximum conservancy water levels controlled to 17.465 m GD, 17.37 m GD (57 ft GD), 17.22 m GD (56.5 ft GD) and 17.07 m GD (56 ft GD). Interestingly there is in fact very little difference in the frequency of supply failure associated with operating the conservancy at a lower top water level. In a 34 year simulation period, there were four supply failures when the top water level was set to 17.465 m GD, four supply failures when the top water level was set to 17.22 m, and five supply failures when the top water level was set to 17.07 m GD. Operating to a level of 17.22 m GD (56.5 ft GD) does not increase the risk of supply failure, although will increase the severity of failures. The indications are that because of the nature of the droughts experienced the risk of supply failure is not significantly increased by reducing the top water level. The cumulative supply deficit over the 34 year simulation period does of course increase as the top water level is lowered. Table 5.2 summarises the simulated cumulative deficits.

Table 5.2
Simulated cumulative water supply deficits for EDC, 34 year simulation

Top water level (m GD)	Cumulative deficit (Mm^3)
17.465	28.2
17.37	30.3
17.22	34.5
17.07	38.4

At present it is frequently necessary to pump water into the conservancy from the Mahaica River during drought conditions. Rehabilitation of leaking outlet structures, and improved water use efficiency would offset any problems caused by

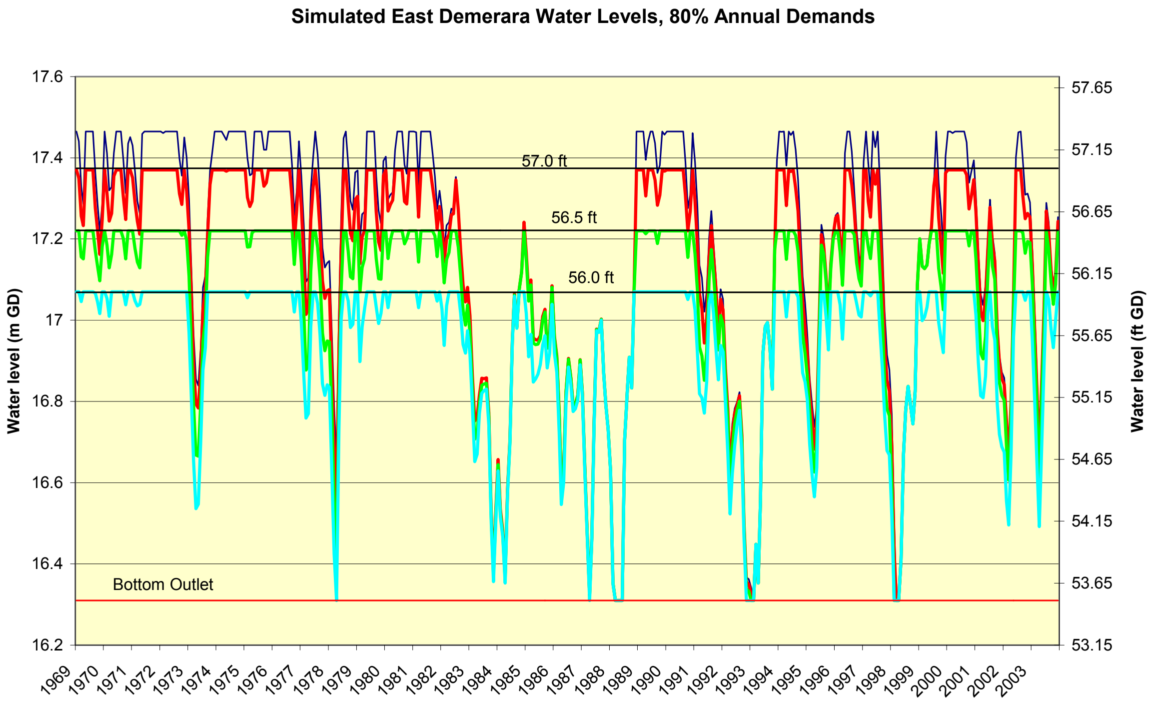


Figure 5.10 Simulated impact of different top water levels on EDC operation

6. Conclusions and Recommendations

A number of conclusions and recommendations have been drawn from the results of the brief investigation

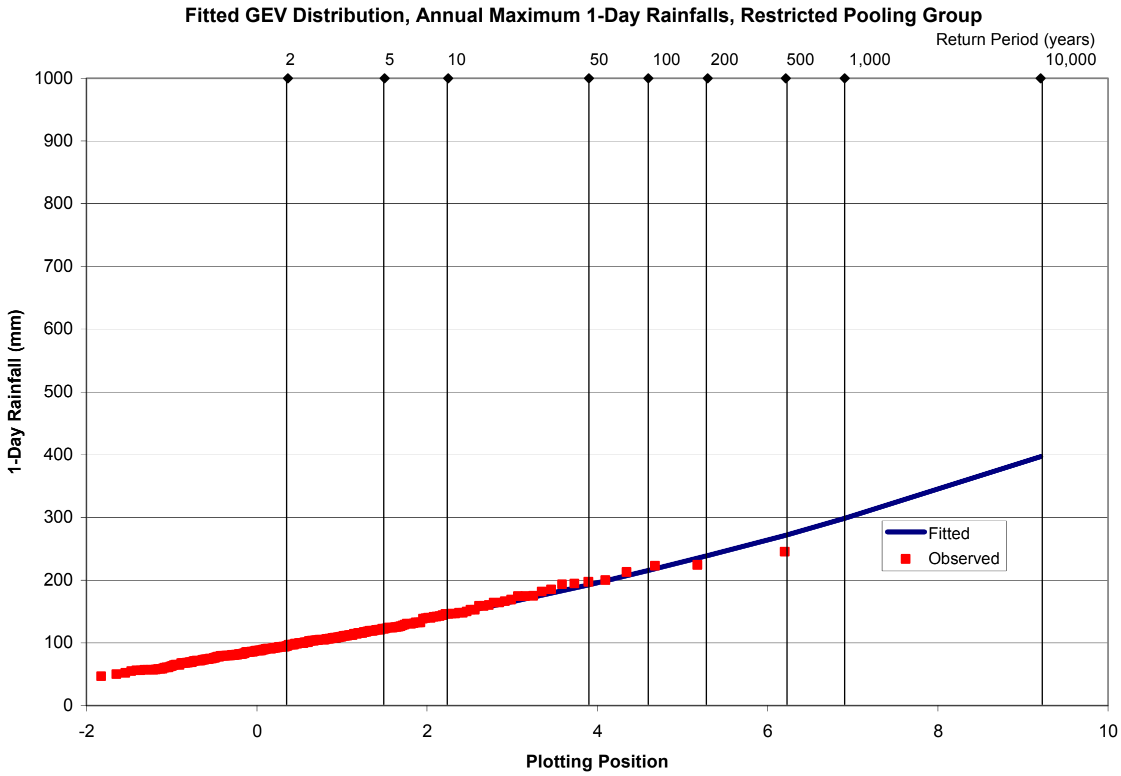
- Hydrographic survey is required for both conservancies. The survey should provide waterway cross sections at regular intervals as well as transects across the entire conservancy. The survey of both conservancies should be considered as a priority work.
- Following hydrographic survey, modelling should be extended to a pseudo two-dimensional mode. This will permit the effectiveness of new structures and approach channels to be properly assessed and designed, and will permit more accurate assessments of conservancy behaviour during flood to be made. It is possible that the current model over-estimates the potential the Cunha, Kofi and Land of Canaan outfalls on the EDC, although the good simulation of the January 2005 flood levels is encouraging. Pseudo two-dimensional models could be set up for both conservancies with modest staff inputs following survey, and should also be considered a priority work.
- The model indicates that there is a number of combinations of gate openings that will permit the EDC to accommodate floods of up to 1000 years return period following rehabilitation of the Kofi and Cunha sluices and outlet channels. These are not sufficient to permit the conservancy to safely accommodate a 10,000 year flood. The more detailed pseudo two-dimensional modelling referred to above will help to confirm that waterways leading to these outlets have sufficient capacity.
- It is strongly recommended that an additional outlet structure be provided on EDC. It has been shown that a further gated width of 30 m, would, when operated in conjunction with the existing gates, permit evacuation of a 10,000 year flood. This would be a modest investment when considered in terms of the economic consequences of an embankment failure. Proper design of additional outlet facilities will require the use the pseudo two-dimensional model referred to above.
- The EDC cannot be operated in any sort of real time or responsive mode to rainfall. The storms that produce serious damage and flooding cannot be forecast with any accuracy. The conservancy should be operated on the basis of a pre-defined schedule of gate openings.
- The SEEC committee schedule of gate openings for the EDC are appropriate and will provide an additional margin of safety while further rehabilitation works are carried out, and until a new and large additional outlet structure can be provided. Operating at a level of 56 ft GD permits greater flexibility with regard to Lama sluices operation.
- The impacts of lower top water levels on water resources does not result in significant increase in the risk of supply failure from the Conservancy. Operating to a top water level of 56.5 ft GD results in the same risk of failure as operating to a level of 57 ft GD, although there is of course some increase in supply deficits.
- Embankment levels around the Boersarie Conservancy must be surveyed as a matter of urgency.
- The 8000 ft weir is the major flood relief facility on the Boerasirie conservancy. This cannot function anywhere close to its design capacity because of vegetation growth upstream and downstream of the weir. Were the weir functioning properly, the

Boerasirie conservancy would have been able to accommodate the January 2005 flood without overtopping.

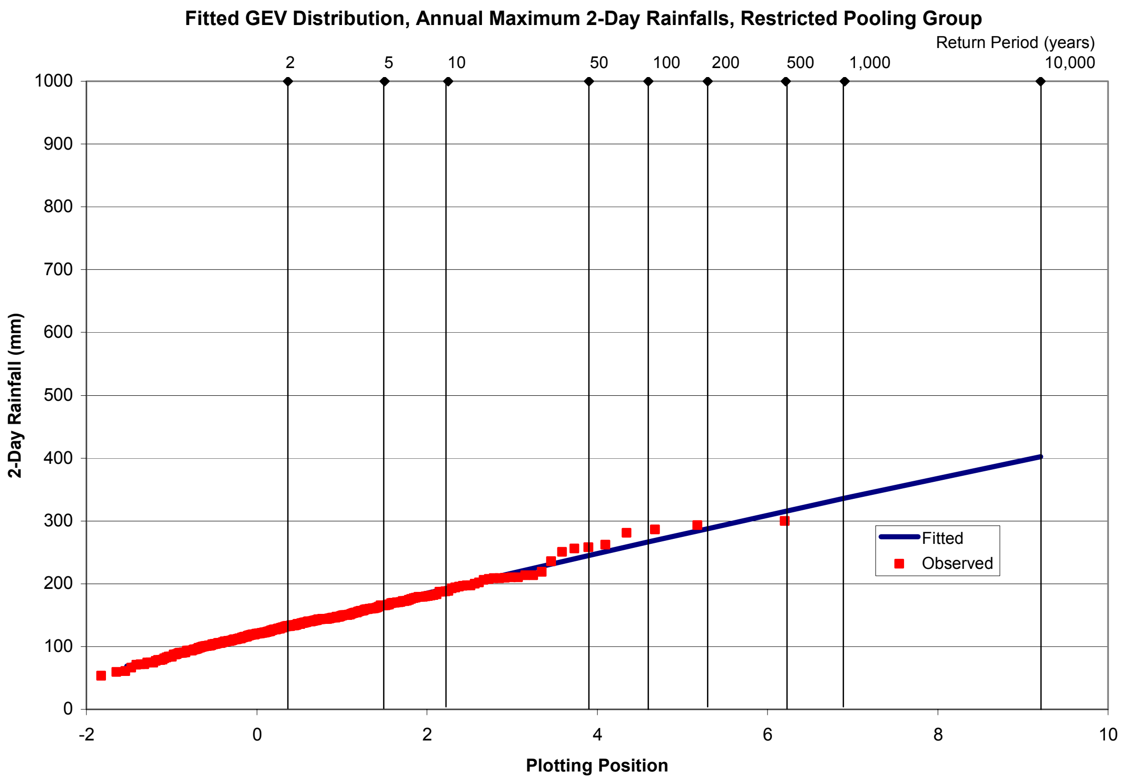
- An assessment needs to be made of the sustainability of adequately maintaining the 8000 ft weir – the present maintenance regime clearly cannot keep up with vegetation growth. If it cannot be maintained, then an additional high capacity safe outlet must be provided. A strategic decision is required on this.
- Waterways in the Boerasirie conservancy need to be cleared to permit better flow conditions to the existing flood relief facilities. Preparation of a pseudo two-dimensional model for the conservancy would help establish the size to which the channels should be maintained.

APPENDIX A

Fitted GEV Frequency Distributions to Pooled Data



Fitted GEV Distribution, Annual Maximum 1-Day Rainfalls, Restricted Pooling Group
Return Period (years)
2
5
10
50
100
200
500
1,000
10,000
1-Day Rainfall (mm)
0
100
200
300
400
500
600
700
800
900
1000
-2
0
2
4
6
8
10
Plotting Position
Fitted
Observed



Fitted GEV Distribution, Annual Maximum 2-Day Rainfalls, Restricted Pooling Group
Return Period (years)
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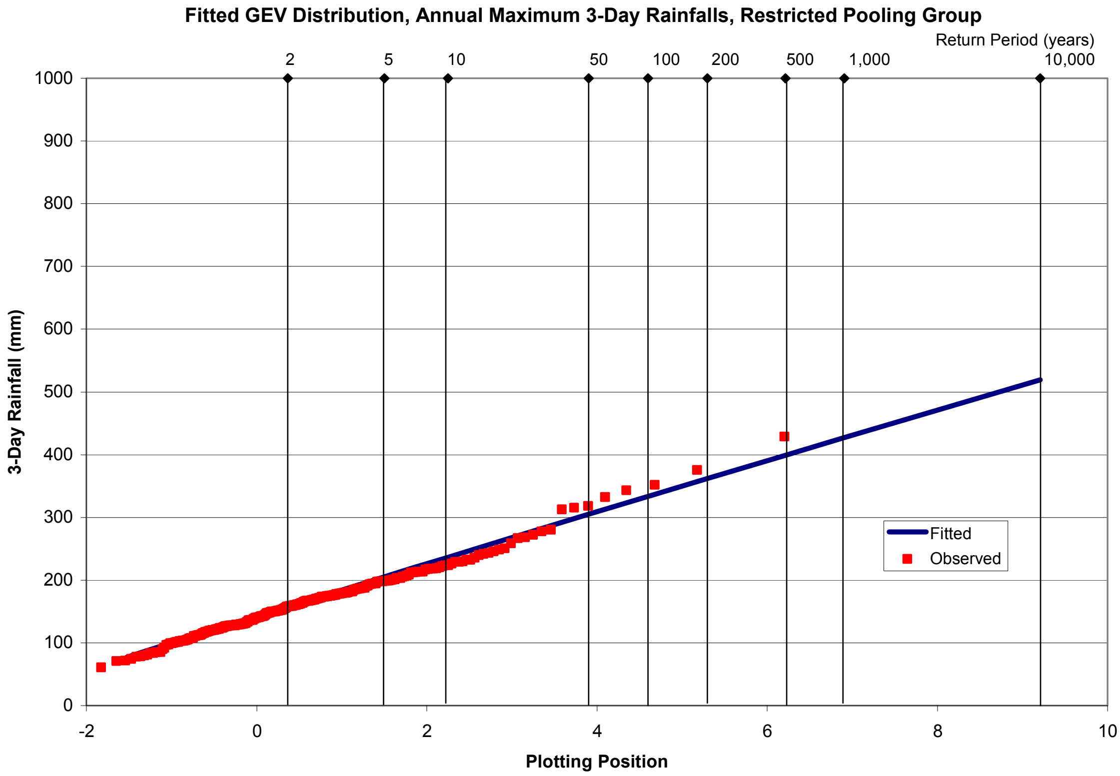
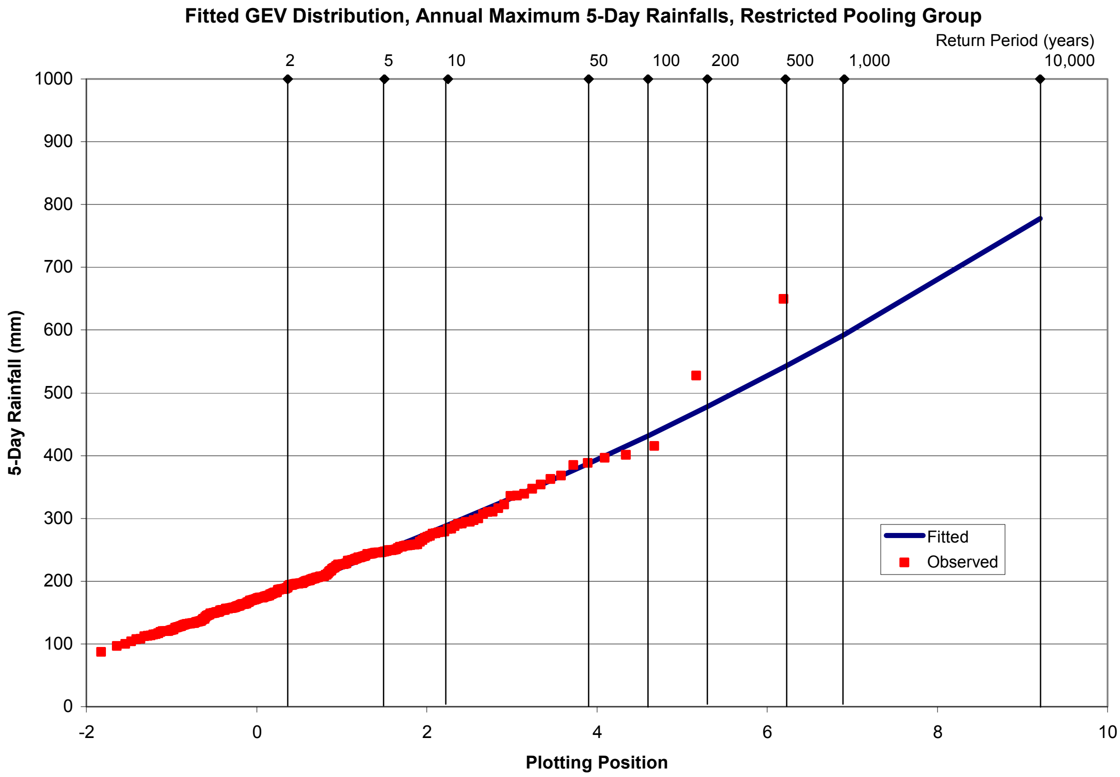
10
50
100
200
500
1,000
10,000
2-Day Rainfall (mm)
0
100
200
300
400
500
600
700
800
900
1000
-2
0
2
4
6
8
10
Plotting Position
Fitted
Observed


Fitted GEV Distribution, Annual Maximum 3-Day Rainfalls, Restricted Pooling Group
Return Period (years)
2
5
10
50
100
200
500
1,000
10,000
3-Day Rainfall (mm)
1000
900
800
700
600
500
400
300
200
100
0
-2
0
2
4
6
8
10
Plotting Position
Fitted
Observed


Fitted GEV Distribution, Annual Maximum 5-Day Rainfalls, Restricted Pooling Group
Return Period (years)
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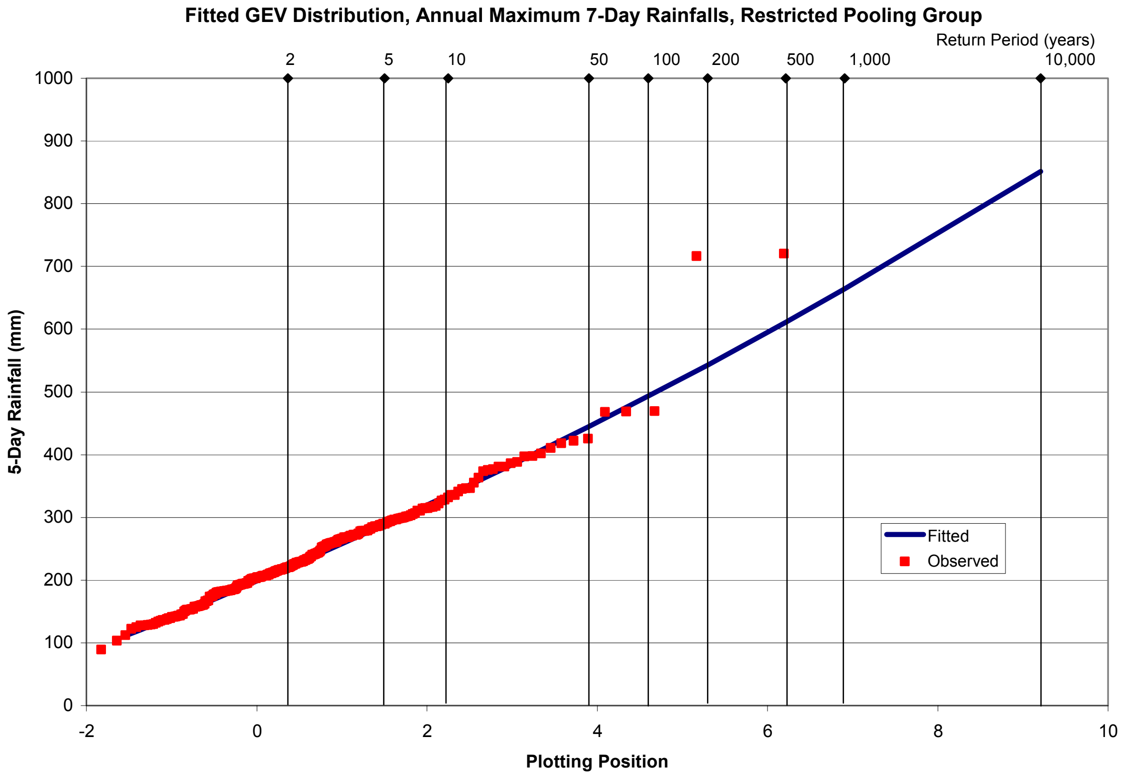
50
100
200
500
1,000
10,000
5-Day Rainfall (mm)
1000
900
800
700
600
500
400
300
200
100
0
-2
0
2
4
6
8
10
Plotting Position
Fitted
Observed



Fitted GEV Distribution, Annual Maximum 7-Day Rainfalls, Restricted Pooling Group
Return Period (years)
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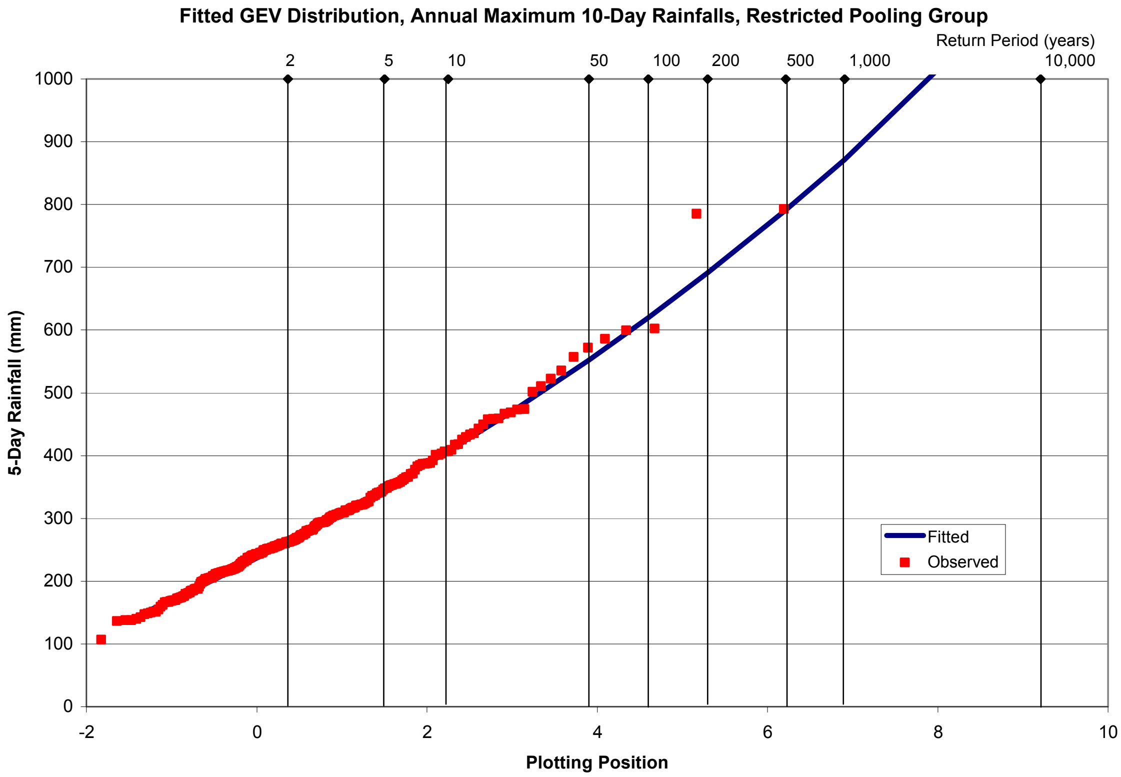
100
200
500
1,000
10,000
5-Day Rainfall (mm)
Plotting Position
Fitted
Observed



Fitted GEV Distribution, Annual Maximum 10-Day Rainfalls, Restricted Pooling Group
Return Period (years)
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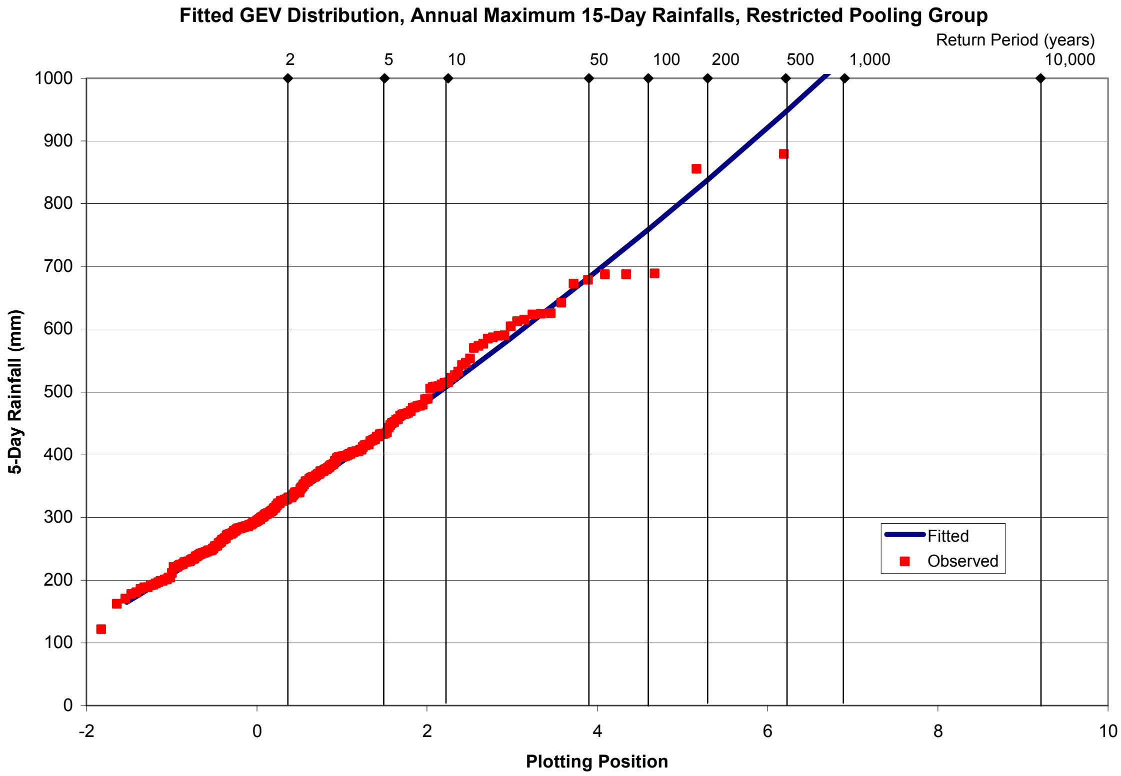
10
50
100
200
500
1,000
10,000
5-Day Rainfall (mm)
Plotting Position
Fitted
Observed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Return Period (years)
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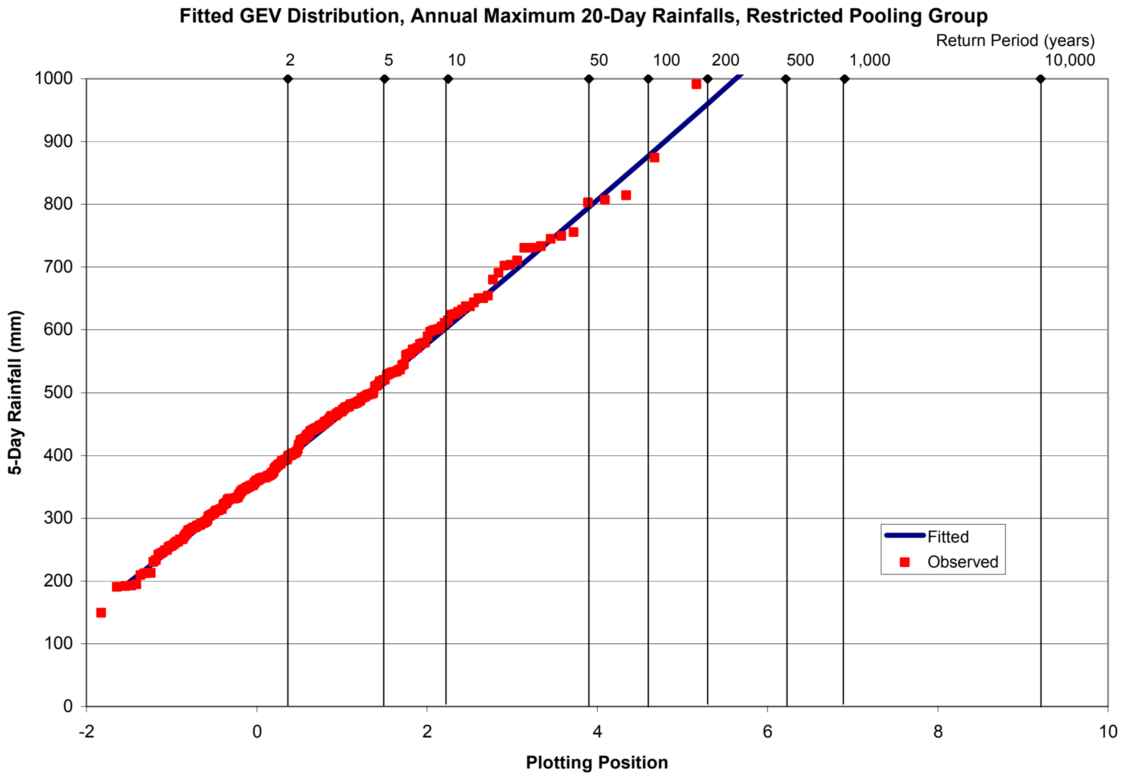
200
500
1,000
10,000
5-Day Rainfall (mm)
Plotting Position
Fitted
Observed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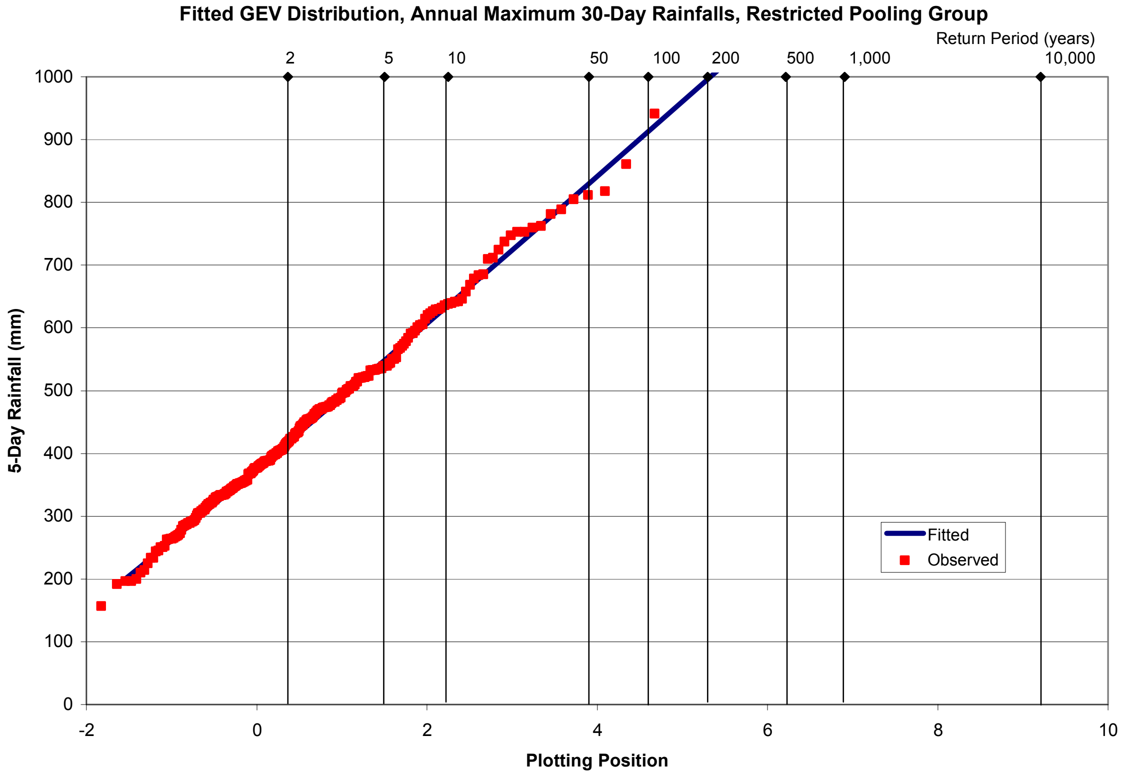
100
200
500
1,000
10,000
5-Day Rainfall (mm)
Plotting Position
Fitted
Observed


Fitted GEV Distribution, Annual Maximum 30-Day Rainfalls, Restricted Pooling Group
Return Period (years)
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10
50
100
200
500
1,000
10,000
5-Day Rainfall (mm)
0
100
200
300
400
500
600
700
800
900
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-2
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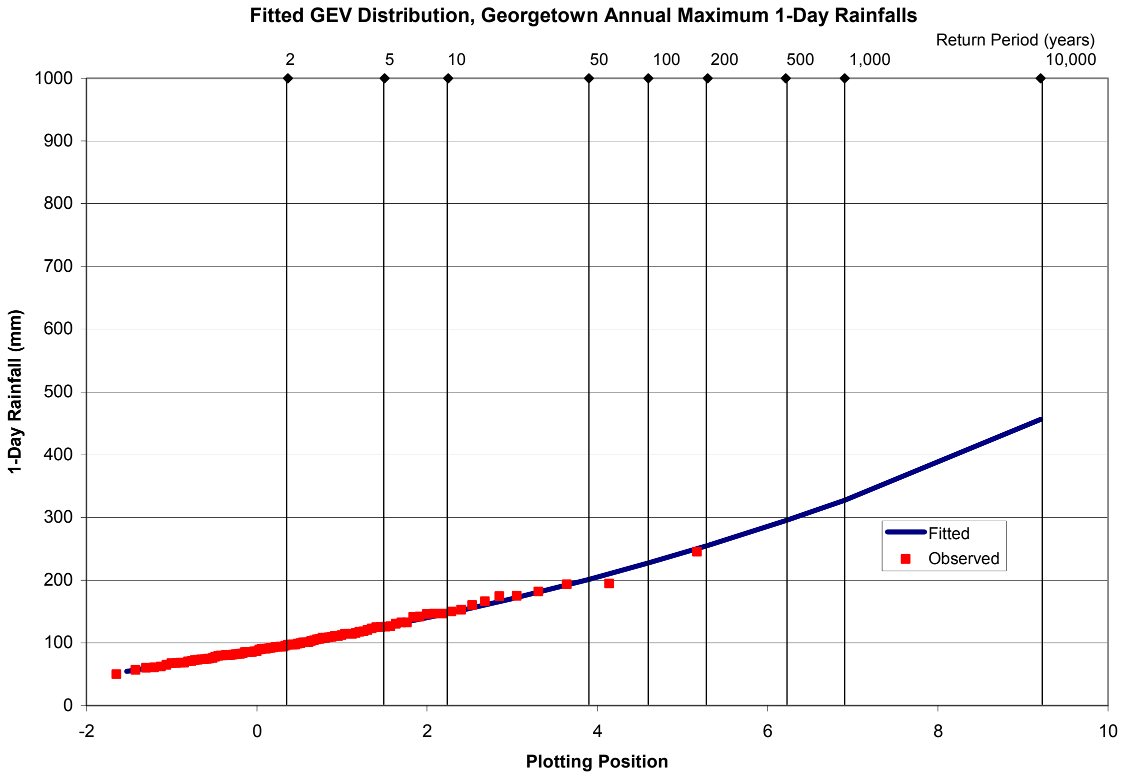
Plotting Position
Fitted
Observed

APPENDIX B

Fitted GEV Frequency Distributions to Georgetown Data


Fitted GEV Distribution, Georgetown Annual Maximum 1-Day Rainfalls
Return Period (years)
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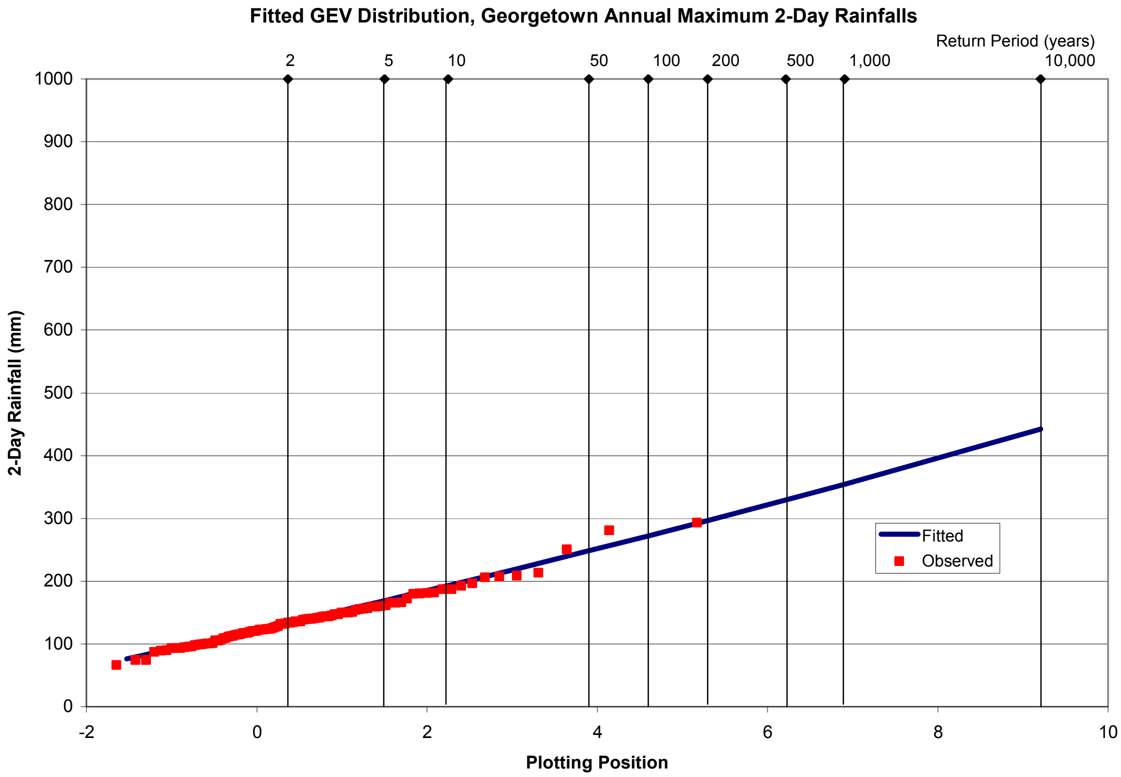
10
50
100
200
500
1,000
10,000
1-Day Rainfall (mm)
Plotting Position
Fitted
Observed


Fitted GEV Distribution, Georgetown Annual Maximum 2-Day Rainfalls
Return Period (years)
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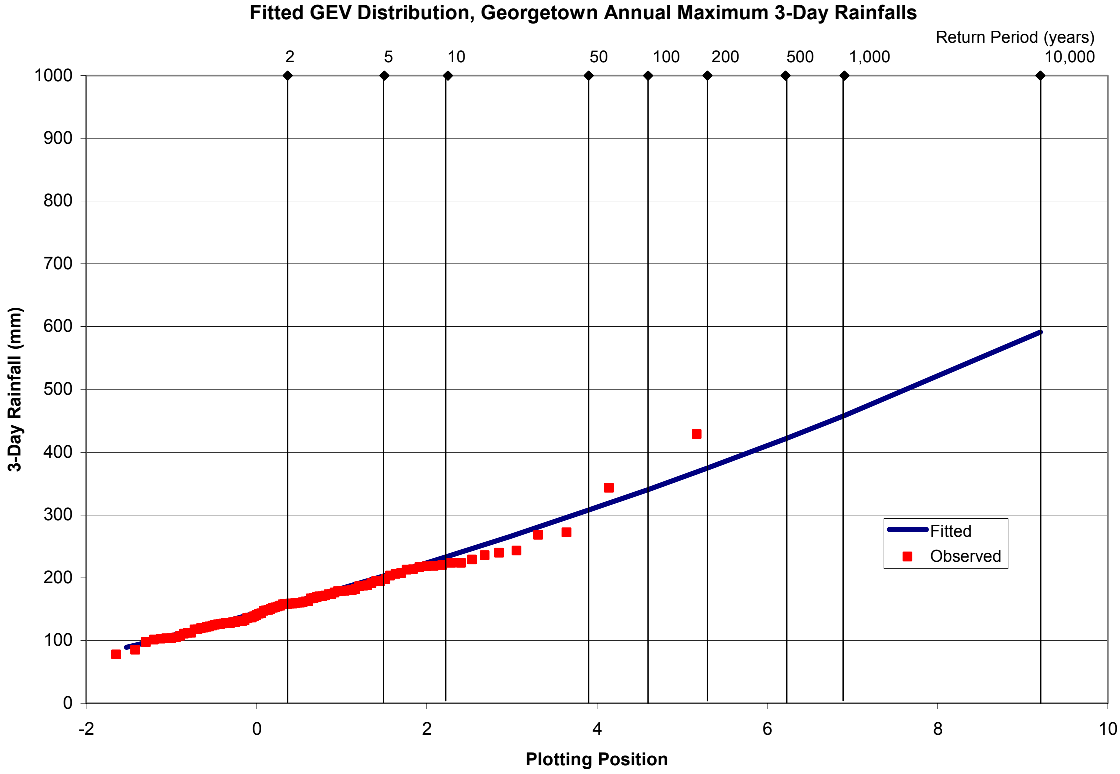
5
10
50
100
200
500
1,000
10,000
2-Day Rainfall (mm)
Plotting Position
Fitted
Observed


Fitted GEV Distribution, Georgetown Annual Maximum 3-Day Rainfalls
Return Period (years)
2
5
10
50
100
200
500
1,000
10,000
3-Day Rainfall (mm)
1000
900
800
700
600
500
400
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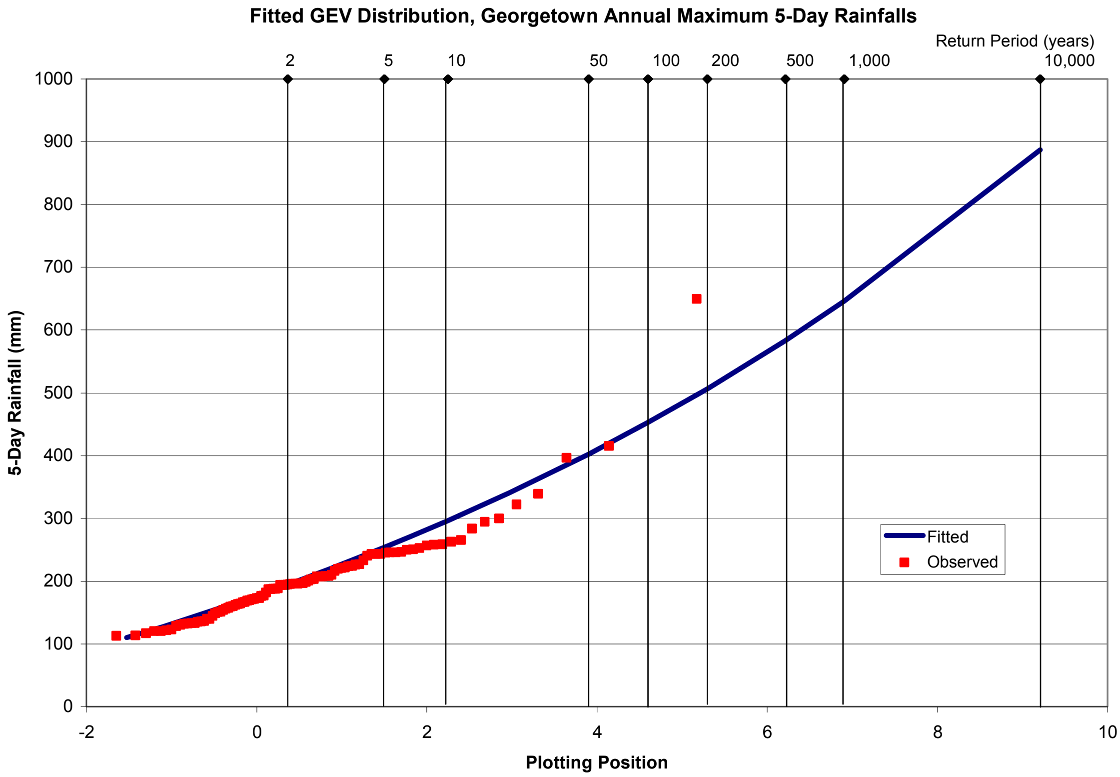
200
100
0
-2
0
2
4
6
8
10
Plotting Position
Fitted
Observed


Fitted GEV Distribution, Georgetown Annual Maximum 5-Day Rainfalls
Return Period (years)
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10
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100
200
500
1,000
10,000
5-Day Rainfall (mm)
1000
900
800
700
600
500
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300
200
100
0
-2
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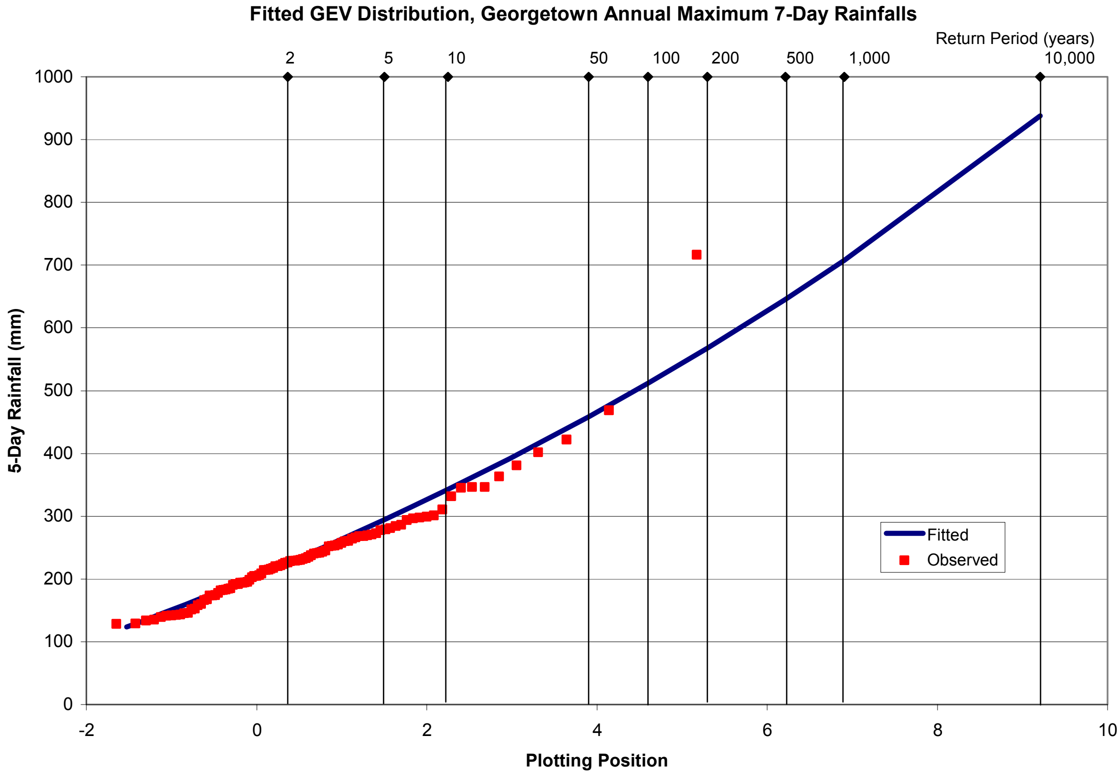
4
6
8
10
Plotting Position
Fitted
Observed


Fitted GEV Distribution, Georgetown Annual Maximum 7-Day Rainfalls
Return Period (years)
2
5
10
50
100
200
500
1,000
10,000
5-Day Rainfall (mm)
1000
900
800
700
600
500
400
300
200
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-2
0
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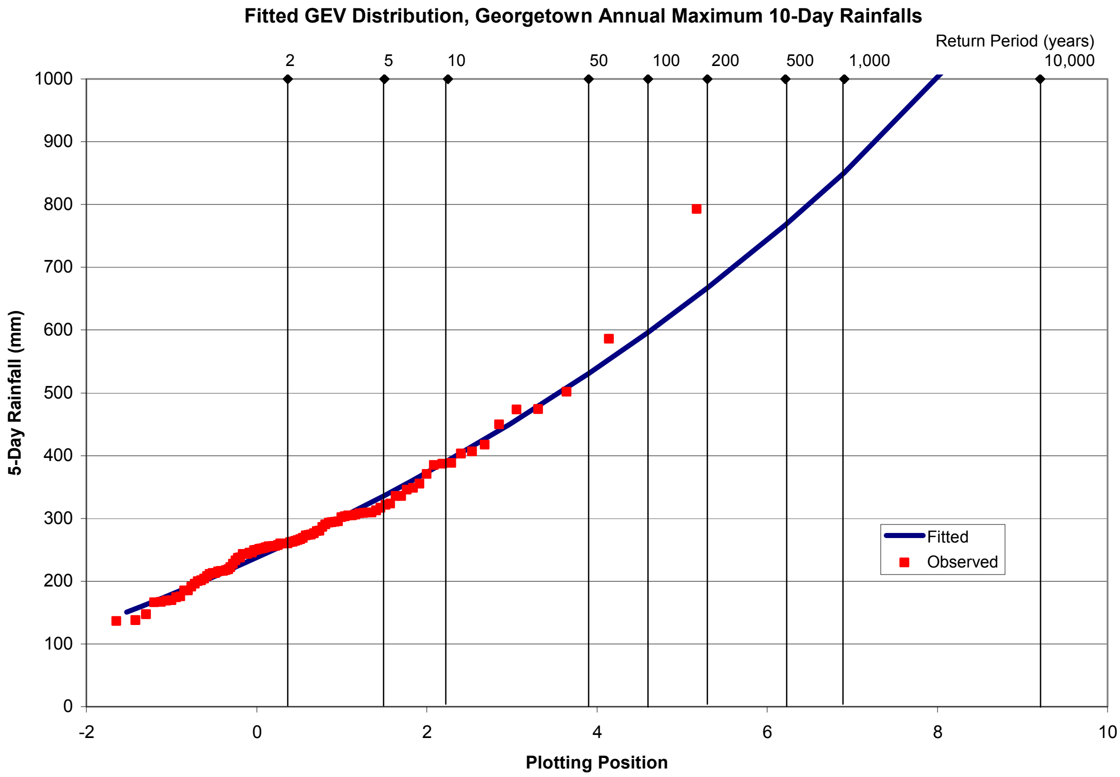
4
6
8
10
Plotting Position
Fitted
Observed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Return Period (years)
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10
50
100
200
500
1,000
10,000
5-Day Rainfall (mm)
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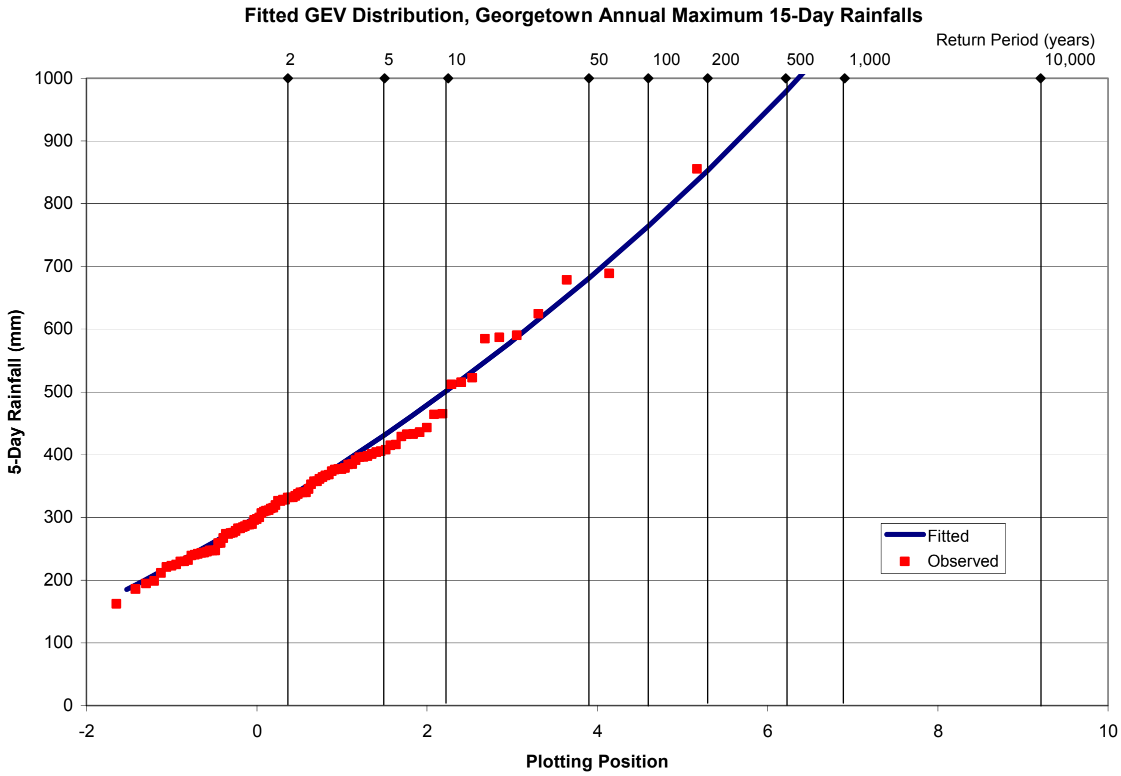
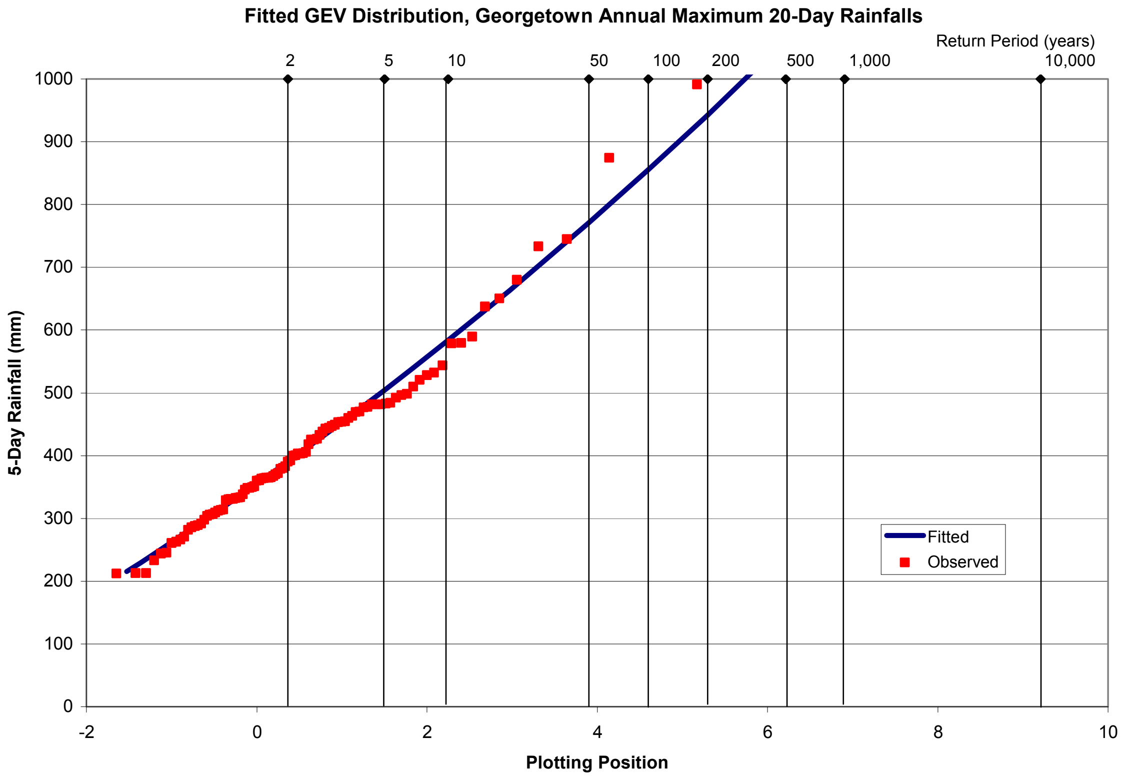
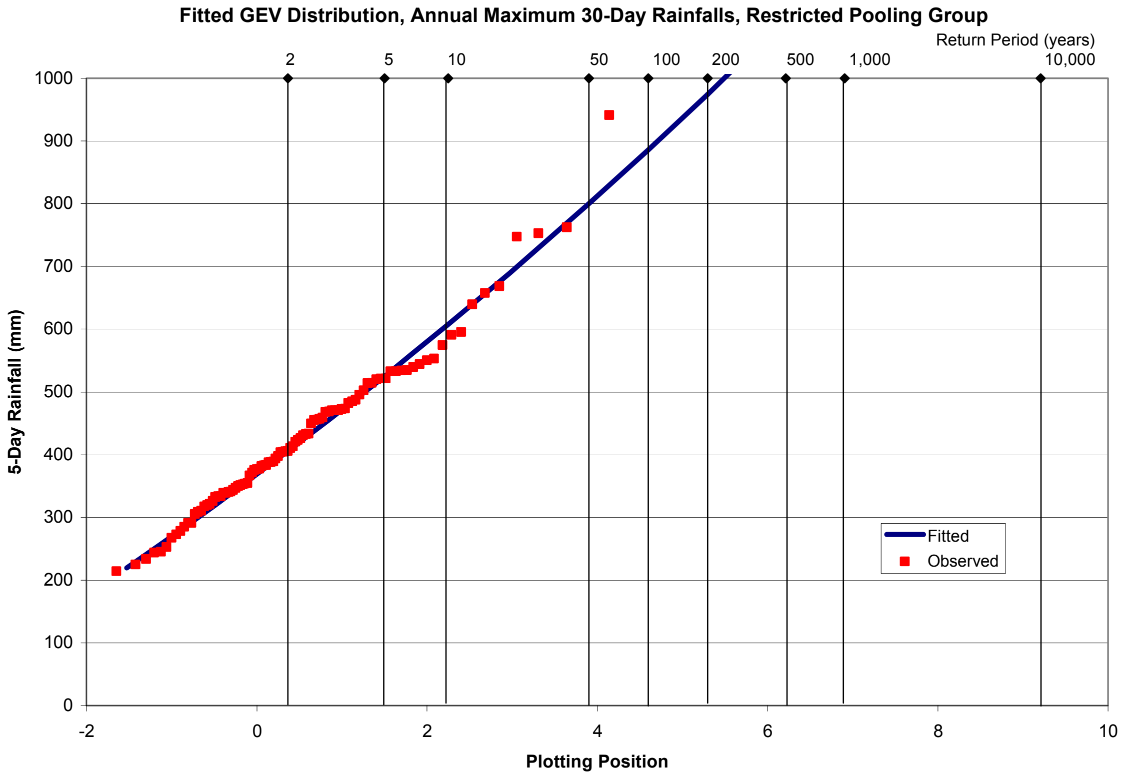
800
700
600
500
400
300
200
100
0
-2
0
2
4
6
8
10
Plotting Position
Fitted
Observed


Fitted GEV Distribution, Georgetown Annual Maximum 15-Day Rainfalls
Return Period (years)
2
5
10
50
100
200
500
1,000
10,000
5-Day Rainfall (mm)
Plotting Position
Fitted
Observed


Fitted GEV Distribution, Georgetown Annual Maximum 20-Day Rainfalls
Return Period (years)
2
5
10
50
100
200
500
1,000
10,000
5-Day Rainfall (mm)
Plotting Position
Fitted
Observed


Fitted GEV Distribution, Annual Maximum 30-Day Rainfalls, Restricted Pooling Group
Return Period (years)
2
5
10
50
100
200
500
1,000
10,000
5-Day Rainfall (mm)
0
100
200
300
400
500
600
700
800
900
1000
-2
0
2
4
6
8
10
Plotting Position
Fitted
Observed