East Demerara and Boereserie Conservancy Dams

Condition Assessment by Dams Specialist

1 Condition Assessment by Dams Specialist

2 Introduction

The visit by the Mott MacDonald Dams Specialist was carried out between 30 March and 9 April 2005. Under the consultancy TOR, the purpose of the visit was to provide technical support to the Special Advisor with respect to the emergency works proposed to safeguard the integrity of the East Demerara and Boereserie Conservancy Dams up to and including the coming rainy season expected in May and June.

However, as part of the visit, the probable requirements for Medium Term construction works to be carried out in the period up to the end of 2006 were also reviewed. The purpose of these works was to ensure the integrity of the dams in relation to possible lengths at the two Conservancies, which may be of marginal safety but require more extensive rehabilitation than can be undertaken as part of the emergency works. The design of these works will be undertaken under a separate consultancy and the purpose of the present review has been to identify and establish the required programme of site investigations. This is to enable an advance to be made in view of the time constraints and urgency of any works identified as necessary within the Medium Term.

The Dams Specialist was asked during the visit to comment on what might be necessary in the Long Term with respect to the Conservancy dams. This has been to identify possible required feasibility studies to provide the long term integrity of the Conservancies or alternative schemes.

3 Approach

The review has had a threefold approach:

- Site visits to the two Conservancies;
- Collection and review of existing reports where available with respect to the two Conservancies;
- Meetings and discussions with engineers from local consultants or Government Departments, who have had past experience of the Conservancies.

To supplement this work, there have been continual frequent discussions with members of the Task Force, which have been most useful.

The site visits have involved boat trips along the lengths of the Conservancy dams, with frequent stop-offs at locations where damage to the embankments has occurred to enable a direct inspection. This exercise included the full length of the East Demerara Water Conservancy (EDWC) dams and the larger part of the Boereserie Water Conservancy.

With a total length of dam at EDWC of 38 miles and a 28 mile length at Boereserie, it has not been possible to walk the full length of the dams. Instead the emphasis has been to invigilate the Secretary (or in other terminology “Superintendent / Maintenance and Security Officer”) responsible for the upkeep of each Conservancy. These two individuals are impressive, maintaining a very close interest and monitoring of their respective Conservancy.

The emphasis of the study has been the EDWC, as this has been known to pose the greatest risk of a possible failure. All the past studies and related reports made available have been with respect to this Conservancy. In addition the knowledge and experience of many engineers who have been involved with the problems of EDWC have been sourced.

4 Data Collection

The following reports have been made available:

Mahaica-Mahaicony-Abary (MMA) Project Stages II & III, Project Profile Report, (Camacho)

Study of the “El Nino Weather Phenomenon on the East Demerara Water Conservancy and the Mitigation of any Effects, April 1998 (Dharry, 1998);

East Demerara Water Conservancy Dam Raise and Repairs, May 2000 (Valentine, 2000);

Hydrology and Water Resources, Final Report, Guyana Drainage and Irrigation Systems Rehabilitation Project, June 2004 (Mott MacDonald (MM), 2004).

The particular relevance of the latter report are the flood estimations undertaken for extreme storm events (1 in 100 year to 1 in 1000 year return periods) with respect to EDWC and Boereserie, with corresponding flood levels in the two Conservancies provided.

5 Original Construction of Conservancy Dams

A search of their archives by the British High Commission yielded a small drawing produced in 1874 showing the intended design of the dams at EDWC. This indicated a 10 foot crest width to elevation 59 ft G.D., with side slopes of 1:1. It is expected that the Boereserie dam was of a similar design. When originally planned, it is understood that the embankments were solely intended to prevent flooding of the coastal farms and estates rather than intended to impound water over the long term.

With basic construction materials and equipment in the 1880s when EDWC was formed, the dams were constructed by hand labour with minimal compaction. The fill material was taken primarily from directly upstream as part of excavation of the canal, with probably little selection of the best soil or exclusion of pegasse (peat). This tends to be confirmed by the boreholes placed in 1998, which show the fill to be a random mixture of clay and pegasse. The foundations of the dams are soft clays or pegasse and it is not known if the original upper layer of topsoil (or pegasse) was removed prior to fill placing.

Over the 100 years plus which the dams have been in use there has been significant settlement of the crest, erosion and over significant lengths slips up to 5 ft high in the upstream slope. These slips have occurred when the water level in the Conservancies reduces post the end of the wet season and most notably during droughts when the water level is fully drawn down. A notable occasion was during the El Nino effect of 1998. These slips should be not unexpected in view of the very steep (1:1) upstream slope. In addition, at EDWC there have been significant slips (or slope failures) in the downstream slope at specific locations and this will be discussed later in this report.

6 Principal Issues with Respect to Integrity of Earthfill Dams

6.1 General

Prior to discussing the specific aspects at the respective Conservancy dams, a brief explanation is provided on the general issues with respect to the integrity of earthfill embankment dams. The three principal issues are overtopping, piping and embankment instability.

6.2 Overtopping

The risk of failure is obviously dependent on the depth of water over the crest (or discharge per unit length) and failure starts by erosion at the downstream toe, which then progresses into the body of the embankment. Full failure is generally rapid, but this is also dependent on the cohesiveness of the fill material, with a sand eroding much more rapidly. In the case of the Conservancy dams, the clay fill, particularly if a high plasticity clay, is less erodable and this has doubtless prevented greater damage up to now.

6.3 Piping

"Piping" is internal erosion due to an excessive hydraulic gradient in relation to the type of fill (or foundation) material. Piping is generally a gradual process, first starting at the downstream slope or foundation and then progressing through the embankment or its foundation at an accelerating rate, as the hydraulic gradient increases. If observed in time, it can be countered by the provision of a downstream filter(s) system to stop the migration of material.

"Piping" is one of the most common causes of the failure of embankment dams, usually occurring at structures such as culverts within the dam body, with seepage occurring along the interface between the fill and the concrete. Failures due to "piping" have also occurred due to the presence of dispersive clays and it is generally standard practice for new embankment dams to test for possible dispersivity by laboratory tests. As in the case of overtopping, unless there are dispersive clays, cohesionless sands are much more potentially erodable than cohesive clays.

In the context of both the EDWC and Boereserie dams, there is no evidence of piping in terms of the internal erosion process described above. Similarly there is no indication that dispersive clays are present. Prior to laboratory testing, the first initial clue to the presence of dispersive clays is deep erosion or gullying in the surrounding area or at the dams themselves. There is no evidence of this over the 120 year life of the Conservancy, including under the frequent intense rainfall to which the area is subjected.

In clay fill dams such as at the Conservancies, "pipes" may be present due to decay of tree roots or branches left within the original construction. This is what occurred at one of the EDWC dams in January 2005 and this is discussed later. It is unlikely that such a pipe would cause failure of these low height embankments, but of course should be rectified without delay as occurred. The treatment adopted of placing sand filled bags at the exit of the pipe, to act as a filter, is correct.

6.4 Embankment Instability

6.4.1 Upstream Slope

Upstream slope instability is caused by a drawdown of the water level in a reservoir, taking away the supporting water loading against the slope before the internal water pressures (pore pressures) which reduce the shear strength within the fill have had time to dissipate. This is referred to as the “rapid drawdown” loading for conventional dams. However where the fill is poorly compacted and/or at an unduly steep slope and/or the clay is virtually impervious, as at the Conservancy dams, this slope failure can occur under quite normal drawdown of the storage level. Generally the slips are quite shallow, but gradually progress into the embankment until the fill reaches a long term stable slope. The danger here is if eventually this results in a failure of the crest resulting in overtopping.

At the Conservancy dams, there are considerable lengths where slips have occurred in the upstream slope. These could have been worse at some locations, but this was prevented by the binding effect of tree and scrub root zones. It is understood that the majority of the slips occurred during drought periods, such as occurred during 1998 due to El Nino, and were caused by a combination of low storage levels and tension cracks forming at the crest due to drying out and subsequent shrinkage.

Examples of slips in the upstream slope which have occurred are shown in Photo Nos. 5 and 6.

6.4.2 Downstream Slope

These are normally deep seated failures and hence are serious. The cause is a combination of an excessively steep downstream slope and high water table (or phreatic surface) within the downstream shoulder of the dam embankment. This is countered in the design of conventional embankment dams by the provision of an internal drainage system, comprising sand and/or gravel layers. Where there is no internal drainage such as at very low dams (less than 15 ft high), a shallower downstream slope is adopted, so that the phreatic surface does not exit at the exposed surface. Clearly the level of the phreatic surface in the dam embankment is dependent on the storage level upstream and hence the risk of instability can be reduced by maintaining a lower water level.

In the case of the Conservancy dams, the original downstream slope of 1:1 and correspondingly the existing slopes are excessively steep. Failure of slopes has only occurred at specific locations where there is a combination of higher embankment (or deep excavation downstream) and less cohesive clay fill. Elsewhere slope failure has not occurred either due to low height of embankment and/or the presence of more cohesive medium to high plasticity clays.

7 Treatment of Trees in Conservancy Dams

There is extensive tree, bamboo or scrub growth at the crest and downstream slope of specific lengths of the dams at both EDWC and Boereserie WC. These are at:

- The western dam, 11 miles long, at EDWC (see Photo No. 1);
- The northern dam, 11.5 miles long, at Boereserie WC

The question of treatment of trees, including bamboo, has been discussed in Progress Report No. 2, describing the advantages and disadvantages of retaining the trees. It is therefore considered worthwhile to clarify this issue at this stage.

Typically trees are not allowed at embankment dams, as toppling of the trees could cause a localised instability due to the root zone. Similarly growth of trees and shrubs are prevented, as they impede a proper inspection of the downstream slope. This is conventional practice at all recent and new embankment dams.

The problem arises at old dams, with established trees, as at the Conservancies. If the trees are removed, then the roots have also to be grubbed out. This could require quite deep excavation in low height dams if the root zone is extensive. In the case of bamboo, the indications are that the root zone has contributed to the stability of the upstream slope.

Typical UK practice with respect to large trees at old embankment dams is to leave these in place, unless there is a clear reason in terms of the dam's integrity for their removal. However, if such trees are retained, then the typical practice is for regular (annual) inspection by an experienced forester to check if there is any risk of toppling. If there is risk, then the tree is cut down and the roots grubbed out but often removal of specific large branches is adequate. It is suggested that this should be the practice at the Conservancy dams.

In the case of bamboos, these should also be retained, but if dying should then be removed, the roots grubbed out and the hole carefully backfilled. Obviously this work should be carried out outside periods when high storage levels are occurring.

8 East Demerara Water Conservancy

8.1 General Description

A plan of the Conservancy showing the various dams and offtake locations is shown in Figure 7.1. For ease of reference, the dams are described as West, North, North East and East, as shown in the figure. The lengths of the various dams are as indicated in Table 7.1.

FIGURE 7.1: East Demerara Conservancy

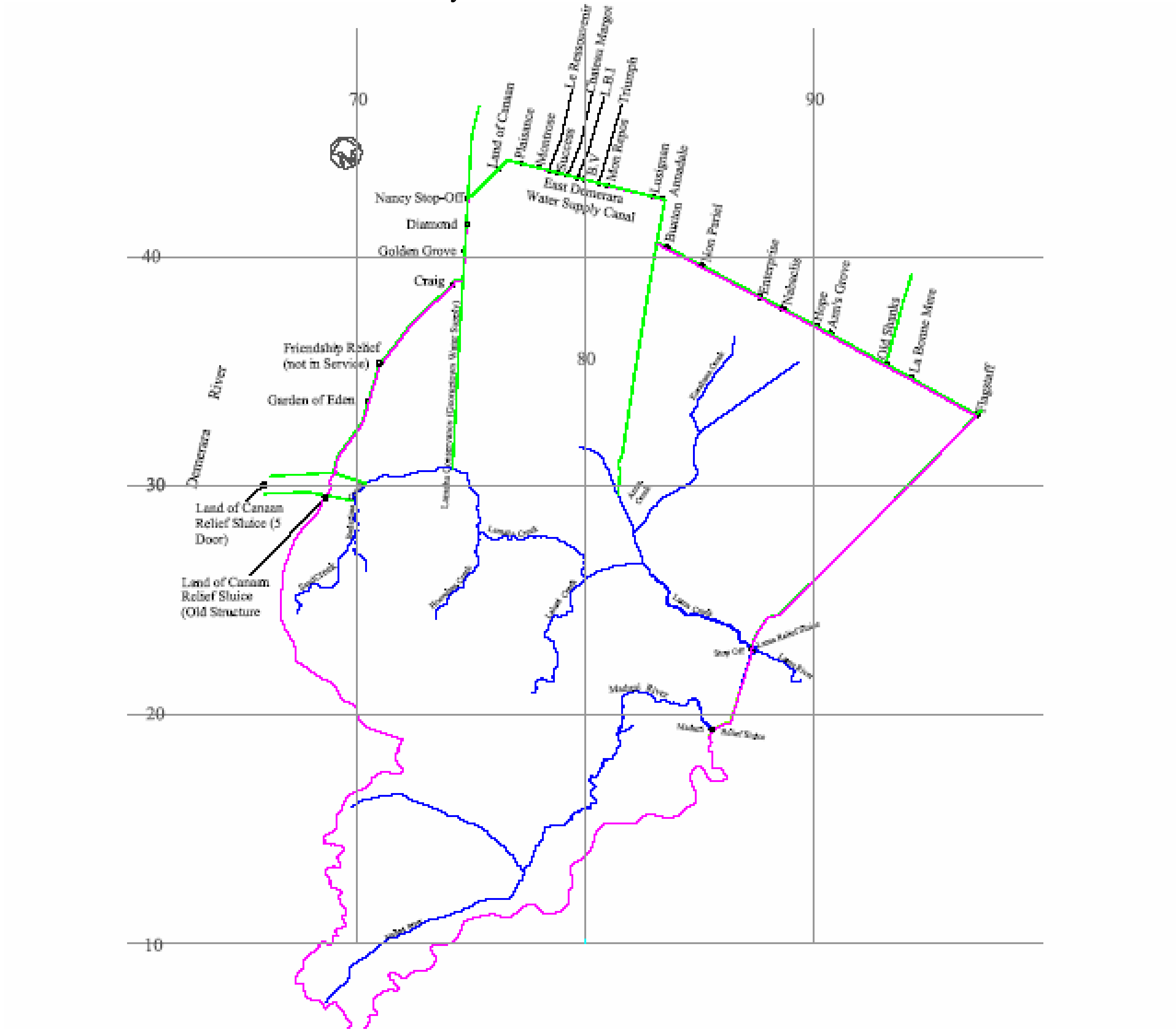


Table 7.1

Lengths of Conservancy Dam

Dam	Location	Length (miles)
West	Land of Canaan Relief Sluice to Nancy Stop-Off	9
North	Nancy Stop-Off to Buxton	6
North East	Buxton to Flagstaff	8
East	Flagstaff to Maduni Relief Sluice	9

8.2 Present Condition of EDWC Dams

8.2.1 West Dam

This dam is under heavy vegetation, including large trees and bamboos. Due to settlement and erosion over time, it has been necessary to restore the crest, mainly by the placing by hand of clay fill from inside the Conservancy. This has resulted in a narrow crest over large length of this dam, often only 4 to 5 ft wide. A typical view of this dam is shown in Photo 3

This area of the Conservancy has cohesive medium to higher plasticity clays, which were used to form the dam embankment originally and for the repair works. Similarly there is from a visual impression less evidence of the presence of pegasse (peat). Due to these factors, the dam embankment has generally remained stable during its life, apart from some localised small slips at the upper part of the upstream slope. However the dam embankment has been subject to settlement and hence the additional fill placing at the crest referred to above. The present crest level and the variation in this level along the length of the dam is not known, but the impression is that it may be adequate apart from some localised sections.

8.2.2 North Dam

This embankment is founded on and constructed on weaker clays than the West Dam. There are no trees or bamboo on the embankment, but a good heavy grass vegetation except where additional fill has had to be placed at the crest. The latter has been necessary due to settlement of the crest over time, generally by hand. There was no evidence of significant slips on the downstream slope. Similarly the slips on the upstream side were limited to a relatively shallow depth from the crest. A typical view of this embankment is shown in Photo Nos. 2 and 3.

These photos show the additional fill placed at the crest or the use of sand-bags, depending on the width of crest available. Much of the fill placed is of not very good quality and has had to be placed rapidly last January; hence the uneven appearance.

Generally the appearance of the larger length of the dam, in terms of the fill placing which has had to be undertaken over time and the varying cross-section of the embankment, does not inspire confidence in its long term integrity, and site investigations to establish its true stability are recommended later in this report. Restoration of crest levels has been done visually and the actual crest levels at present are not known.

8.2.3 North East Dam

This dam has historically caused the most problems and has been the subject of several studies in the last ten years plus major repair works. The cause has been primarily the poor fill used for the original dam construction and correspondingly the suspected weak foundation soils, which would have been sourced for the dam construction. Most particularly the dam fill contains significant quantities of pegasse, as evidenced by the borehole logs for the boreholes placed at the dam crest in 1998 (Amary, 1998).

The weak construction has caused slips at the upstream slope along much of its length, and quite extensive in some places. The cause has been due to the normal drawdown of the reservoir from GD 57.5 ft to GD 54.0 during each dry season, but further aggravated by a deeper drawdown at times of drought. This was most critical in 1998, when the storage level was drawdown to approximately GD 51.0 ft and possibly lower.

Due to significant settlement of the crest over time, once more reflecting the poorer construction, reconstruction works were carried out in 1995. Unfortunately the Contractor sourced much of his fill material directly downstream of the downstream toe of the dam, to depths of up to 10ft, and this caused breaches or major slips in several locations. At one location this necessitated urgent repair works using sheet piling. This location is shown in Photo No. 4. The poor condition of this embankment is also shown in Photo Nos. 5, 6 and 7. The crest re-instatement undertaken in January/February of this year and the proximity of the trench excavated for the borrow material, as referred to above, are shown in Photo Nos. 8 and 9.

A suspected piping occurred at one location along this dam during the high storage levels last January. Rather than internal erosion, the cause was found to be a tree trunk and some branches, which had been placed in the embankment fill. It is not known if this was a reflection of poor original construction or placed during more recent repair work. This was repaired satisfactorily by removal of the wood and the placing of sand filled bags as a toe weight/filter at the downstream toe. However, due to the quality of the bags (refer to Section 7.3.2), this can only be a temporary measure.

Due to poor foundations and original construction, including the presence of pegasse in the fill, it is considered that the larger length of this dam is at marginal stability (factor of safety approximating to 1.0). This is also reflected by the 1995 repair works and associated studies. Similarly there must be some concern that there could be other locations where old tree trunks or branches have been left within the embankment. As for the other dams, the actual present crest levels are not known, with the most recent crest restoration carried out by the Conservancy Maintenance and Security Officer by visual assessment only.

8.2.4 East Dam

There is no extensive vegetation at this dam, apart from random grass growth.

There are two locations along this dam where slope failures have occurred in the downstream slope. These are shown in Photo Nos. 10 and 11. The latter shows the emergency sheet piling which had to be placed. These two locations are in the northern part of this dam and, according to the Conservancy Maintenance and Security Officer, are at areas where there were originally stream courses. This cannot be confirmed. Elsewhere along much of this dam, there has been a need to raise the crest level due to settlement. This has been done by vegetation debris or occasionally sand bags, as shown in Photo Nos. 12 and 13. These can only be temporary measures.

Due to these slips, the overall long term stability of this dam is questionable. It is proposed later in this report that site investigations are carried out at this dam to establish the true present stability.

8.2.5 Outlet Structures (Kokers) at Dams

This assessment only covers their integrity in terms of the safety of the dams rather than hydraulic capacity or mechanical condition.

Seepage during the high storage levels in January and February has been reported by observers from Guysuco occurring at the downstream sides (concrete/fill contact) at several of the outlet structures. This is not considered to be a problem, and there was no evidence of any damage at the outlet structures on the down stream slope or toe actually visited.

However there is a major seepage through the fill or structure at the offtake structure at Enmore in the North East Dam, which supplies the Guysuco sugar factory. This is an uncontrolled "koker", which has a steel pipe in the foundation starting in the upstream canal. In addition there is clear evidence of a major leak from flow patterns visible directly upstream at the left hand side revetment. This is shown in Photo No 14. Unfortunately it is not possible to establish visually if the leak is through the fill or foundation or some original structure. At this stage the worst, i.e. through the fill, must be assumed.

This structure is unsatisfactory in many respects. Apart from the leakage referred to above, the release from the Conservancy is excessive due to its uncontrolled nature and beyond Guysuco's requirements. As a result there is an unnecessary waste of water from the Conservancy and also a significant cost to Guysuco to pump this water out of the sugar estates into the sea.

Although the leakage referred to above has been occurring over some period of time, it is a significant risk and needs to be rectified as soon as possible. This is discussed later in this report in Section 7.4.5.

8.3 Emergency Works

8.3.1 General

Emergency works have already been instituted as indicated in Progress Report Nos. 1, 2 and 3. These include the provision of sand-bags and the stockpiling of a significant number of already filled sand-bags at the Conservancy headquarters at Flagstaff. In addition sheet piling should be stockpiled at Flagstaff as recommended by SEEC. This is all satisfactory, but some observations on the use and type of sand-bags is given in Section 7.3.2.

In addition the Maintenance and Security Officer has a very good understanding of both what should be monitored on a daily basis and what repair works should be undertaken if an emergency occurs. This was evident from detailed discussions with the superintendent during the site visit. The Maintenance and Security Officer confirmed that he had an adequate number of rangers for the daily inspections required during the emergency period and they understood what to look for in terms of seepage or overtopping.

8.3.2 Sand-Bags

The quality of the sand bag material being used is poor- just local rice bags. These are weak and degrade rapidly in sunlight. The latter is partly countered by covering with soil, where possible, but the bags break easily as witnessed during the site visits. Although it could be argued that they are only required to be sound up to the end of the rainy season, the reality is that they may be needed for much longer. It is therefore preferable that proper geotextile bags, particularly produced for this purpose, are procured as soon as possible. These can be fabricated to any particular required size by the geotextile supplier or, possibly better in this situation, the geotextile sheets or rolls are supplied along with stitching machines to enable the bags to be produced locally as demand requires. These stitching machines require only a small power supply and hence the bags can be produced to the required size (depending on the proposed handling) "on site".

The sand bags have to date been filled with clay imported from a distance of about 10-12 miles. Whereas normal practice is to fill bags with sand, this may not be appropriate in this case in view firstly of the haul distance for the sand and, secondly, the need to institute the repairs locally along each dam. However it is recommended that the priority for sand-filled bags should be at the downstream slope or toe, if seepage is identified. Where it is needed to restore the crest, clay filled bags, packed down by hand, will be adequate. The bags should not be totally filled.

8.3.3 Assurance Against Overtopping

At present the crest of the dams has been restored with clay fill bags on a visual basis only. Whereas this is reasonable, it would be much preferable if the full crest length of the dams is surveyed, with marker pegs placed at, say, 300 ft intervals. The clay fill bags should then be placed throughout to a designated level, based on these marker pegs.

In terms of the designated crest level required, the following flood levels are indicated:

January/February 2005:	El 58.8 ft
1 in 500 year return period flood	El 58.5 ft
1 in 1000 year return period flood	El 58.6ft

The two return period flood levels are based on the June 2004 "Hydrology and Water Resources" Report by MM in 2004.

On the basis of these levels and pending any information from the modelling studies for SEEC, it is recommended that all the crests are placed to GD 60.0 ft minimum prior to the coming wet season. This should be achieved using clay fill bags.

8.3.4 Offtake Structure at Enmore

The leakage occurring at the left revetment for this structure is of concern. Unless this can be rectified at the short term, which is unlikely, this leakage needs to be very closely monitored especially if the storage level increases during the coming few months.

Unfortunately it is not possible to see where the leakage is exiting on the downstream side, most probably directly into the downstream channel. It is recommended that a dye is used, introduced at the entry point, to try and establish where the leakage is exiting.

Irrespective of this, any deterioration will be evidenced by slumping of the crest or downstream slope at this location and/or by evidence of an increased discharge. If this does occur, there will be an immediate need to drive sheet piling at the centreline of the dam. However this will first require exposure of the tie rods supporting the existing sheet pile whalings, so as to avoid breaking these.

8.4 Medium Term Requirements (Until End of 2006)

8.4.1 General

As discussed in Section 1, the present visit has been required to review and comment on the emergency works required. However at the same time there is a need to institute a site investigation programme for the medium term works, so that these can be advanced ahead of the required design studies. This is essential in view of the time constraints for the design and construction over this one and a half year period.

The Medium Term works will be examined in detail under a separate consultancy. These will be targeted on those existing works, which are considered to pose a significant risk which cannot await more longer-term solutions. In the case of some of the dams, the degree of present risk cannot be established without site investigations, including laboratory testing, which will enable slope stability analyses to determine the actual present stability.

In the Medium Term, all measures possible should be taken to prevent over-stressing the dam in terms of water loadings. Ideally the maximum storage level should not exceed GD 57.5 ft and hopefully the advance warning measures to be initiated under SEEC should help this. In order to reduce the risk of slips in the upstream slopes, the storage level during the dry season should not reduce below GD 53.5 ft and preferably higher.

8.4.2 North East Dam

It is apparent, as discussed in Section 7.2.3, that this dam is at marginal stability. It therefore needs to be rectified in the Medium Term. However, due to the deep flooded excavations existing at the downstream toe, strengthening by building up the downstream slope and widening and heightening the crest, will be complicated and costly. Similarly there are lengths of the existing dam where the body of the dam has been compromised by the previous slips and the temporary placing of pegasse.

In this situation it would be preferable to construct an entirely new dam embankment downstream of the existing. Ideally it would be located as close as possible to the existing dam, say 400 to 500 ft downstream, but this will depend on the foundation conditions. Similarly the length of the new dam required may not have to extend over the full 9 mile length. This will depend primarily on the extent of the deep excavations at the downstream toe and this should be established by a detailed survey along the full length of the dam.

The programme of site investigations proposed would be in two stages. Firstly a series of hand auger holes would be placed over a 500 x 500 ft grid to identify areas where pegasse can be avoided or partly avoided. Once the preferred location is established, more detailed site investigations followed by laboratory testing would be initiated.

From observations provided by local engineers, who know the Conservancy area well, it may be difficult to avoid areas of pegasse unless the dam alignment is moved a significant distance downstream. This unfortunately will result in the loss of good farming land. In this context it may be necessary to locate the dam on foundations with pegasse. Whilst dams practice is to avoid organic soils in the foundation or fill, it should be noted that the present embankments, which are probably partly founded on pegasse, have stood up to a degree over more than 100 years.

A further example of where dam embankments have been constructed on deep peat is the Fens area of eastern England, where embanking on such soils is unavoidable. Mott MacDonald have had specific experience of this, having from their Cambridge office been closely involved over a number of years in the rehabilitation and strengthening of the original Barrier Banks. These were constructed to a similar height as the Conservancy dams.

8.4.3 East and North Dams

It is recommended that the present stability of these dams is checked as soon as possible. The priority should be firstly the East Dam and then the North Dam, for the reasons given in Sections 7.2.2 and 7.2.4. The programme of site investigations would involve boreholes within the existing dam and its foundation, placed from the present crest, and further single row of boreholes directly downstream of the dam. The results from the laboratory testing of samples from the former will enable the stability of the existing embankments to be checked. The boreholes at the downstream toe will also help towards this purpose and in addition will be used for the design of any strengthening works, which in the case of these two dams can be achieved by building up the downstream slope with fill placing from ground level. A decision on the need to strength these two dams will depend on the results of slope stability analysis.

8.4.4 Site Investigation Programme

The suggested site investigation programme, including laboratory testing is given in Appendix A. The field work would comprise firstly hand auger holes, as explained earlier, in-situ vane tests and then conventional boreholes with undisturbed and disturbed sampling.

The main strength testing of the soils for the stability analyses would be carried out by the consolidated undrained triaxial test (CU-triaxial). This is a complicated test, with only a few laboratories world-wide capable of undertaking the testing to the right standards to achieve the correct results. A review of the geotechnical studies for the 8th EDF Sea Defence Project in 2001 show that there is one laboratory in Guyana capable of carrying out this test and to the required standard. This is Ground Structures Engineering Consultants Inc. and they will need to be used for this, although some corroborative CU-triaxial testing should still be carried out overseas.

8.4.5 Offtake at Enmore

It is recommended that, on dam safety grounds, this structure should be rebuilt in the Medium Term programme. This could solely involve driving light duty sheet piles at the dam crest as described in Section 7.3.4 or a completely new structure.

Siphons could be used as a solution to provide the long term regulated flow. However these can be difficult to operate satisfactorily and a completely new structure with regulation by gates may be a better solution.

8.4.6 Increased Spillway Capacity

The flood studies undertaken in 2004 (MM, 2004) and the subsequent flooding in January and February have identified the need, as a priority, of increased outlet capacity at the Conservancy. This may take the form of additional gated structures, or a long properly engineered overtoppable embankment to ensure that at critical floods there is no risk of overtopping of the dams. The use of an overtoppable embankment takes away the risk of human error in the event of an extreme flood.

9 Long Term

The present Conservancy dams do not meet internationally recognised standards with respect to safety of impounding reservoirs. Although the dams are of low height, the risks they pose in the event of failure are considerable. The dams would fail to meet the required standards of integrity under any one of three risks- potential risk to life, major economic disruption and loss of a primary existing source of domestic water supply to Georgetown and surrounding areas.

In view of this, it is recommended that feasibility studies are initiated without delay identifying and examining alternative long-term solutions. These alternatives would include:

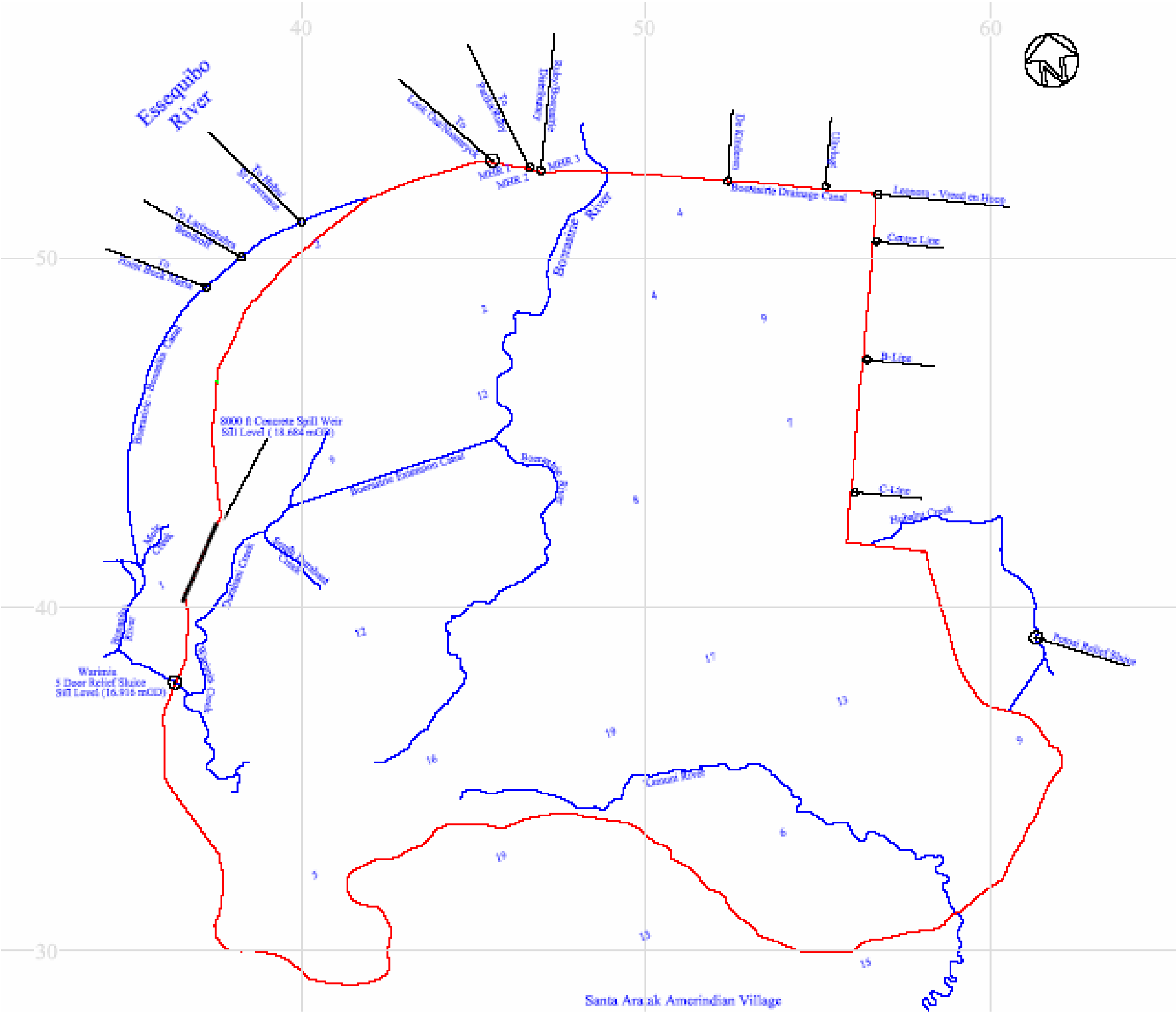
1. Strengthening of all remaining dams not undertaken under the Medium Term works;
2. Construction of new dams downstream of, and as a replacement for, those dams not undertaken under the Medium term works. This could also include the possible use of the existing downstream Crown Dam;
3. Implementation of the Mahaica-Mahaicony-Abary Stage II & III Project (MMA II), which it is understood will enable a lower storage level to be retained at the EDWC Conservancy. This will achieve the requirement of a reduced water loading on the dam, particularly of benefit if the reduction is more than 3 ft.

10 Boereserie Water Conservancy

10.1 General Description

A plan view of the Conservancy is shown on Figure 9.1

FIGURE 9.1: Boersarie Water Conservancy



For convenience the respective dams are called North West, North, East and South East. All the dams have been visited with the exception of the East Dam. The locations of the dams are shown in the figure and are detailed in Table 9.1

Table 9.1

Boereserie Conservancy Dams

Dam	Location	Length (miles)
West	8000 ft Weir to Naamryck	10.5
North	Naamryck to Leonora	6.2
East	Leonora to C-Line	5.5
South East	C-Line to Potosi Relief Sluice	5.6

10.2 Present Condition of Dams Visited

10.2.1 General

The two site visits to this Conservancy and the detailed discussions with the Conservancy superintendent, Mr Ramesh, indicated that the dams at this Conservancy do not have the same problems as at East Demerara Water Conservancy. The embankments are constructed of a good clay material, which is readily available within the Conservancy, generally at short distance from the dams. As a result slips in the dams have generally been limited to a small height on the upstream slopes, mainly it is understood due to wave action from boats, most particularly jetskis (which are actually forbidden on the Conservancy), and boat moorings. The latter is specifically along the East Dam. However, due to the cohesive nature of the clay fill, these slopes still remain relatively stable.

The main risk to the dams is therefore due to possible overtopping and indeed this did occur to shallow depth at some locations in January and February.

10.2.2 North Dam

This is densely vegetated including large trees and bamboos. Due to settlement and long term erosion the crest width has reduced considerably to less than 5 ft over long lengths of this dam. It is also evident that the crest is lower than the other dams and needs to be heightened. Otherwise, due to the good clays and vegetation there appear to be no particular concerns as regards stability and the long-term integrity of the dam.

10.2.3 East Dam

Typical views of this dam are shown in Photo Nos. 15 and 16.

This dam has had its original crest restored both in width and height back to its original size. Due to this, the dam is in the most impressive condition of all the Conservancy dams visited, including at the East Demerara Water Conservancy. There was some overtopping of the dam in January and prompt work was carried out by a local contractor making a wave wall out of clay fill. Possible overtopping of this dam has always been a concern to local people due to extensive habitations directly downstream.

There has been some restoration of the crest level by the use of sand bags at some short lengths along this dam. These were cheap quality sand-bags filled with sand, which have often been torn. A typical example of this is shown in Photo No. 17.

There has been damage to the dam on the upstream side due to the wave action from boats and jet-skis and moorings, as referred to above. Nevertheless, due to the quality of the clay fill and the restored width of crest, these shallow slips do not provide a risk.

10.2.4 South-East Dam

This is in a similar condition to the East Dam. A typical view is shown in Photo No 18.

10.3 Emergency Works

10.3.1 Monitoring and Repair

The Superintendent has a good understanding of the monitoring requirements for the dams and takes a very close interest in their integrity. Similarly he has a team of rangers, who have been briefed by him on the required monitoring along the full length of the dams. This monitoring should be on a daily basis during the wet season.

The Superintendent does however at certain times have difficulties finding the required workforce for maintaining the dams due to better wages on the nearby agricultural estates. It is understood that this is not a critical issue, but the authorities need to be aware of this in case a problem arises at a critical time.

10.3.2 Dam Embankments

The main concern in terms of the integrity over the next three months is the risk of overtopping. A longitudinal survey of the crest over the full length of the dams should be carried out and the crest brought to a consistent level. The following flood levels have been indicated:

January/February 2005	El 63.17 ft
1 in 500 year return period	El 62.03 ft
1 in 1000 year return period	El 62.07 ft

The adopted crest level may depend on the SEEC studies, but should not be less than GD 62.0 ft. In parallel there should be measures to improve the capacity of the various spillway or outlet structures, incorporated into the dams to control storage levels.

Where sand-bags are to be used to restore the crest level, use should be made of proper quality geotextile bags, as discussed in Section 7.3.2. This should ensure a longer term integrity than solely the emergency period. Of particular relevance in this context is the North Dam, which has a narrower crest, which is also too low. Use of geotextile bags may be the most appropriate solution to also cover for the Medium Term works at this location.

10.3.3 Need for Increased Spillway Capacity

The 2004 Hydrology and Water Resources Report (MM, 2004) has indicated the limitations on the overall spillway capacity at the Conservancy. This can immediately be achieved by excavating the channel downstream of the Potosi Relief Sluice. A particular benefit of a good operational sluice here is that it helps to dissipate flows on east side of the Conservancy. At present there is no significant spillway capacity on this side.

In addition to the above structure, there is a second relief sluice structure at Potosi, which is not operational and has a downstream overgrown with vegetation. This channel should be cleared and the sluices made operational, so that this can be available for use in an emergency.

A typical view of the Potosi Relief Sluice and the downstream channel are shown in Photo Nos. 19 and 20.

Appendix A MEDIUM TERM WORKS –SUGGESTED SITE INVESTIGATION PROGRAMME

A.1 Non-Pareil to Flagstaff

A.1.1 Fieldwork

Preliminary investigations by hand auger (or post hole) on 500 ft by 500 ft, to 12 ft depth, with small 2 kg disturbed samples to identify the optimum location of the dam

Investigations for new alignment, 400-500 ft downstream:

- 15 Nr boreholes to 25 ft depth, with undisturbed U4 sampling at 6 ft intervals or every change of strata, SPTs at 4 ft intervals and small disturbed samples (2kg) at 4 ft intervals. Bulk samples (20 kg) to be obtained down to 6 ft depth and 6-12 ft depth;

- 15 Nr in-situ vane penetration holes, adjacent to boreholes, to 20 ft depth with vane tests at 3 ft intervals.

A.1.2 Laboratory Testing

20 Nr Atterberg limits tests

10 Nr specific gravity

10 Nr particle size (sieve analysis and hydrometer)

Undisturbed samples:

- 10 Nr consolidation (oedometer) tests
- 6 Nr natural density and moisture content
- 6 Nr quick undrained triaxial tests
- 6 Nr consolidated undrained triaxial tests (plus 1 Nr extra corroborative test at selected laboratory overseas)

Recompacted samples from bulk samples from boreholes (Proctor compaction at natural moisture content):

- 6 Nr dry density and moisture content
- 6 Nr consolidation (oedometer) tests
- 4 Nr consolidated undrained triaxial tests (plus 1 Nr extra corroborative test at selected laboratory overseas)
- 2 Nr pinhole tests

A.2 Flagstaff to Maduni Relief Sluice

A.2.1 Fieldwork

- 6 Nr boreholes at existing dam axis, extending through embankment and 6 ft into foundation, with undisturbed U4 samples at 4 ft intervals and small disturbed samples at 3 ft intervals and any change of strata;

- 6 Nr boreholes at ground level directly downstream of the centreline boreholes, to depth of 18 ft, with undisturbed U4 samples at 4 ft intervals, SPTs at 4 ft intervals and small disturbed samples (2kg) at 4 ft intervals. In addition large bulk samples (20 kg) taken down to 6 ft and 6 to 12 ft, for use for recompaction tests;

- 6 Nr in-situ vane penetration holes to 12 ft depth, with vane tests at 3 ft intervals.

2 Nr of the above sets of boreholes to be located close to existing slip locations.

In addition 20 Nr hand auger holes, to 12 ft, to identify potential borrow areas, with small disturbed samples taken at 3 ft intervals.

A.2.2 Laboratory Testing

20 Nr Atterberg limits tests

10 Nr specific gravity

12 Nr particle size (sieve analysis and hydrometer)

Undisturbed samples from boreholes on dam axis:

- 6 Nr consolidation (oedometer) tests

- 6 Nr natural density and moisture content

- 4 Nr quick undrained triaxial tests

- 6 Nr consolidated undrained triaxial tests (plus 1 Nr extra corroborative test at selected laboratory overseas)

Undisturbed samples from boreholes in downstream foundation:

- 10 Nr consolidation (oedometer) tests

- 10 Nr natural density and moisture content

- 6 Nr quick undrained triaxial tests

- 6 Nr consolidated undrained triaxial tests (plus 1 Nr extra corroborative test at selected laboratory overseas)

Recompacted samples from bulk samples from boreholes (Proctor compaction at natural moisture content):

- 6 Nr dry density and moisture content
- 6 Nr consolidation (oedometer) tests
- 4 Nr consolidated undrained triaxial tests (plus 1 Nr extra corroborative test at selected laboratory overseas)
- 1 Nr pinhole test

A.3 Nancy Stop-Off to Non-Pareil

A.3.1 Fieldwork

- 4 Nr boreholes at existing dam axis, extending through embankment and 6 ft into foundation, with undisturbed U4 samples at 4 ft intervals and small disturbed samples at 3 ft intervals and any change of strata;
- 4 Nr boreholes at ground level directly downstream of the centreline boreholes, to depth of 18 ft, with undisturbed U4 samples at 4 ft intervals, SPTs at 4 ft intervals and small disturbed samples (2kg) at 4 ft intervals. In addition large bulk samples (20 kg) taken down to 6 ft and 6 to 12 ft, for use for recompaction tests;
- 4 Nr in-situ vane penetration holes to 12 ft depth, with vane tests at 3 ft intervals.
- 20 Nr hand auger holes to 12 ft depth.

A.3.2 Laboratory Testing

20 Nr Atterberg limits tests

10 Nr specific gravity

12 Nr particle size (sieve analysis and hydrometer)

Undisturbed samples from boreholes on dam axis:

- 6 Nr consolidation (oedometer) tests
- 6 Nr natural density and moisture content
- 4 Nr quick undrained triaxial tests
- 6 Nr consolidated undrained triaxial tests (plus 1 Nr extra corroborative test at selected laboratory overseas)

Undisturbed samples from boreholes in downstream foundation:

- 10 Nr consolidation (oedometer) tests
- 10 Nr natural density and moisture content
- 6 Nr quick undrained triaxial tests
- 6 Nr consolidated undrained triaxial tests (plus 1 Nr extra corroborative test at selected laboratory overseas)

Recompacted samples from bulk samples from boreholes (Proctor compaction at natural moisture content):

- 6 Nr dry density and moisture content
- 6 Nr consolidation (oedometer) tests
- 4 Nr consolidated undrained triaxial tests (plus 1 Nr extra corroborative test at selected laboratory overseas)
- 1 Nr pinhole test

Appendix B: Photographs

Photo 1: Vegetation on Eastern dam of EDWC

Photo 2: EDWC North Dam

Photo 3: EDWC North Dam

Photo 4: EDWC North East Dam location needing urgent repairs

Photo 5: EDWC poor condition of EDWC North East Dam: Upstream slips

Photo 6: EDWC poor condition of EDWC North East Dam: Deteriorated Crest

Photo 7: EDWC poor condition of EDWC North East Dam: Damaged Crest

Photo 8: EDWC North East Dam crest reinstatement: Repairs to crest

Photo 9: EDWC North East Dam proximity of borrow trench (flooded)

Photo 10: EDWC East Dam downstream slope failure

Photo 11: EDWC East Dam downstream slope failure

Photo 12: EDWC East Dam temporary crest repairs using vegetation

Photo 13: EDWC East Dam temporary crest repairs using vegetation

Photo 14: EDWC North East Dam leak at Enmore Koker

Photo 15: Boersarie East Dam typical view

Photo 16: Boersarie East Dam typical view

Photo 17: Boersarie East Dam broken sand bags

Photo 18: Boersarie South East Dam typical section

Photo 19: Boersarie Patosi Relief Sluice

Photo 20: Boersarie Patosi Relief channel