HYDROLOGIC AND HYDRAULIC STUDY

FOR

CONSTRUCTION OF A 3MW SOLAR FARM AT LA TOURNEY HILL, VIEUX FORT

SAINT LUCIA

Hydrologic & Hydraulic Study – Final Document

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Summary

A hydrologic and hydraulic study was conducted to determine the expected 100-year flood levels for installing solar panels to generate 3MW of power from a solar farm at La Tourney Hill, St. Lucia. The study also was used for developing stormwater management options within the farm. The major influence on flood levels was from the Vieux Fort River which flowed along the southern boundary of the solar farm site. Runoff from the surrounding hill catchments also had some influence on water levels and had to be taken into consideration for the stormwater management system.

Annual daily maximum rainfall records from the nearby Hewanorra International Airport station were acquired for the period 1985 to 2016 and they were used to perform a statistical analysis for obtaining the 100-year, 24-hour rainfall depth. The observed distribution of rainfall from several rainfall stations in the southern region of the island during the passage of Hurricane Tomas in 2010 was used to develop the 100-year design storm. For the design of the stormwater management system, the intensity-duration-frequency (IDF) curve from Dennery was used to obtain rainfall intensities for estimating peak flows at critical locations.

A suitable hydrologic procedure was used with the 100-year design storm to produce the 100-year flow hydrographs from Vieux Fort River Catchment and the other catchments from the surrounding hills. Land cover descriptors were obtained from recent satellite imagery and soil properties were obtained from a detailed soil survey done several decades ago. Topographical description of the catchments was obtained from the DOS 1:25000 series map of St. Lucia.

The hydraulic analysis for estimating flood levels over the site was done with suitable hydraulic models capable of simulating flood depths over floodplains. Detailed geometric features of the river were collected over a two-kilometre length of the river that bordered the southern boundary of the farm site. Topographical features of within the farm site were obtained from the DOS map supplemented with additional field data.

The study accounted for the effects of climate change in two ways: Regarding the prediction of more intense rainfall, it was assumed that the 24-hour rainfall depth would be increased by 10% and that intensities for short duration storms would be increased by 20%; regarding the anticipated increase in sea level, the predictions of the most recent IPCC (2013) estimate to year 2100 of sea level rise for the Caribbean was used. The conservative upper limit of 0.6 m from that report was added to the estimated highest astronomical tide at Vieux Fort Bay for setting the downstream hydraulic boundary for the hydraulic analysis.

The model outputs suggested that both the river and the runoff from the surrounding hill may inundate some sections of the proposed farm sites especially the southwestern segment where Solar Farm 7C is located. The estimated flood levels were provided in drawings that can be used for the vertical setting of the solar panels.

Stormwater management options were formulated with the aim of ensuring that the natural drainage at the site is maintained and that the adverse hydrologic impacts of solar farms are minimized. The hydraulic analysis on flows at the small streams crossing the proposed roadway provided expected peak flows which were subsequently used to determine culvert dimensions and drainage channel sections along the roads. A proposal for a diversion of a tributary in the vicinity of a solar farm was formulated to reduce flood risks.

Contents

1. Introduction

This report is on the hydrologic and hydraulic study for guiding the installation of a 3MW Solar farm facility at La Tourney Hill, St. Lucia. The proposed system consists of three farms of solar panels located as shown in Figure 1. The study was performed: (i) to determine expected 100-year flood levels and so guide the vertical clearance for each solar panel; and (ii) to design stream crossings of the roadways and to protect the solar panels against erosion from any increased flow velocities through the farms, during both the construction and operational phases.

Item (ii) also is of importance for environmental control as it can result in increased sediment loadings in the nearby Vieux Fort River.

1.1. Design Principles and Standards

The adopted governing philosophy was to provide minimal disruption to the natural drainage system on the site while protecting the solar farms from erosion and accommodating the proposed master plan development. The objectives of the design were to:

1. Protect the solar farm panels from submergence by flood waters;
2. Protect the solar farm panels from erosion;
3. Provide basic storm water management infrastructure for the master plan;
4. Reduce flows from the development to pre-development conditions;
5. Minimize intervention to the natural drainage of the site.

Item 1 is to minimize the risk of flooding at an acceptable level. Item 2 is to ensure that the supports of the panels are not threatened by erosion caused from high velocity storm water, either within, or running onto the farms. Item 3 is to cater for the future development of the site by LUCELEC for power generation and distribution. Item 4 and Item 5 are to minimize adverse environmental impacts resulting from the power plant at the site.

The following design standards were established to meet these design objectives:

1. The water levels for the 24-hour, 100-year design storm are to be used for the vertical setting of the solar panels. Solar panels are to be set 600 mm clear above the 100-year flood level;
2. The recurrence interval for the stormwater management system is to be done at the 10-year level.
3. No flooding of the road deck is to be permitted for passage of runoff from the design storm.
4. Any ponded water from the one hour 1 in 25-year event is to be drained within one day.
5. Drain segments are not to be lower than a minimum slope to ensure velocities are at least equal to 0.6 m/s for self-cleansing.

6. Drain segments are not to exceed a maximum slope to ensure velocities do not erode soils at their location.
7. Runoff coefficients are to be estimated based on the assumption that the master plan as proposed would be implemented and that unoccupied lands would be preserved in their natural condition.
8. Climate change uncertainties are to be provided by applying 10% to peak flows estimated from long duration storms; and 20% to the ones from shorter duration storms.

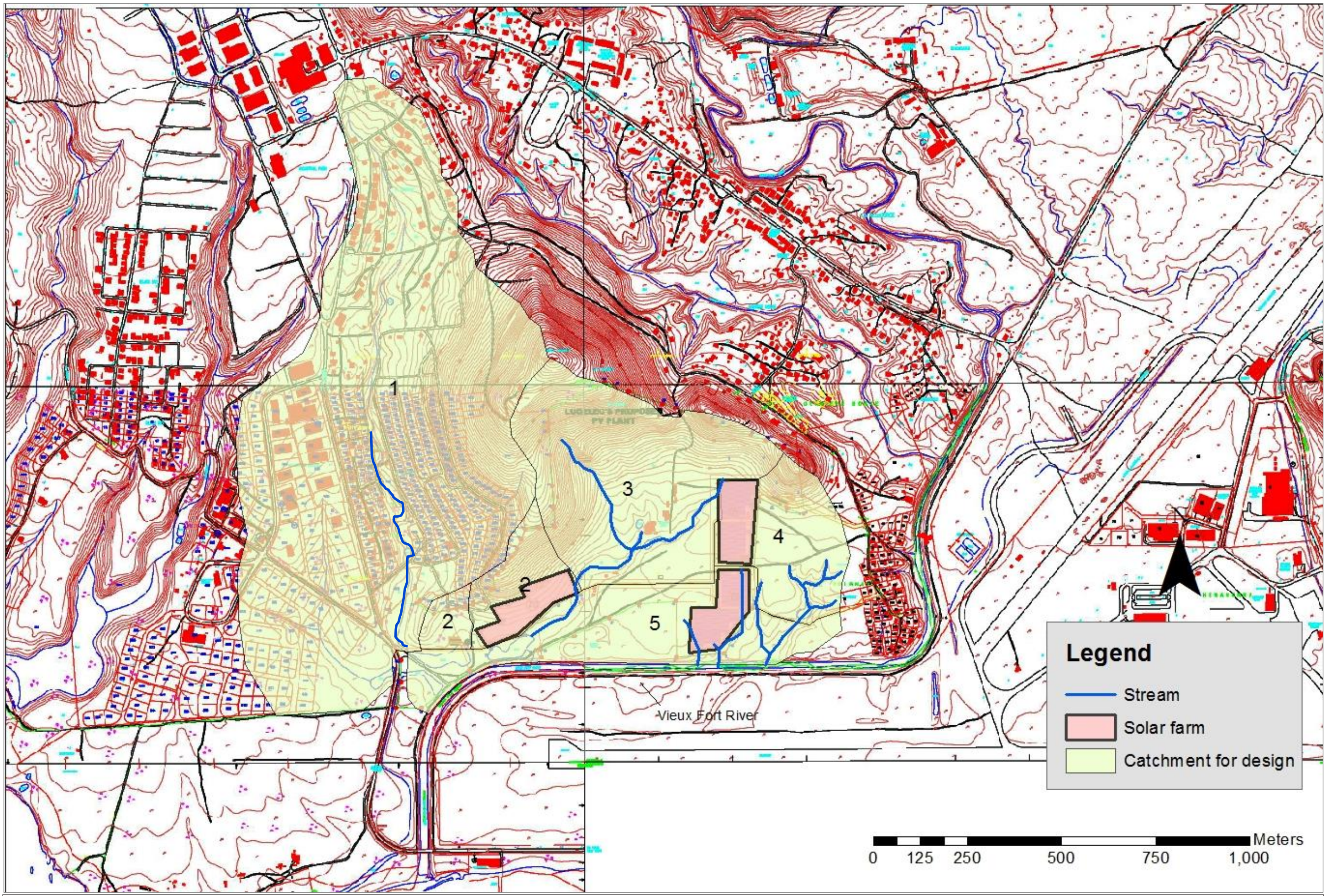

Figure 1. La Tourney Hill site for the solar farm

2. Hydrologic Setting

The farm site is a 13-hectare pasture land situated in the southern region of the island. The northern area of the site is a hill that rises steadily to an elevation of 140 m MSL. The lowest point of the land is on the southwestern side where the grade is flat and the area is marshy. It rises eastward by about 12 metres to a small mound, levels off over a short distance, and falls off to a valley to the east. The land also slopes toward the south to Vieux Fort River. An earthen embankment has been constructed along its southern boundary to protect the site from flooding by Vieux Fort River. An earthen embankment also exists on the other side of the river that serves to limit flooding of the runway of Hewanorra Airport.

There are several small ephemeral stream systems draining the farm site to Vieux Fort River. Their catchment sizes (as defined for the stormwater management measures) range from 9 hectares to 84 hectares. The site is well covered with thick shrubbery. Soils are mainly of a clayey loam texture. Within the site are several earthen lined ponds that have been used for aquaculture.

Vieux Fort River drains a 2670-hectare catchment. This catchment has a mean slope of about 3% and its land cover is mixed, consisting of forest in various hydrologic conditions, from poor to good, and housing of density varying from low to medium. With respect to their infiltration capacities, soils within Vieux River Catchment belong mainly to HSG B[1] (see Table 1), which are soils with high to moderate rate of infiltration. About one fifth of the soils in the catchment belongs to HSG D, which is for soils with low infiltration capacities.

From all indications, the course of the lower segment of the river has been diverted from its original southward alignment to one which flows westward and parallel to the runaway of Hewanorra Airport and then bends south at the end of the runway on its discharge into Vieux Fort Bay. There has been frequent flooding of the airport from breaches of the south bank in the vicinity of the upstream end of the diversion, the most recent event being caused by flows from a tropical depression on Christmas Eve, 2013. The river may be tidal in its lower reaches. The section of the river is essentially trapezoidal, with well vegetated banks.

3. Approach to the Study

The following was the approach for obtaining: (i) the water depths and velocities relevant to the installation of the solar panels; and (ii) for sizing the drain segments and hydraulic structure for the stormwater management system within the site:

a. The setting of the vertical clearances for the solar panels is to be informed by the estimated water levels from the 24-hour, 100-year design rain storm:

[1] HSG B—Soils with infiltration capacity at saturation within the range of 4 to 8 mm/h, according to the Natural Resources Conservation Service (NRCS) hydrologic procedure. See Natural Resources Conservation Service (1997).

i. Flooding of the farm site is from runoff from the catchments within the site and from runoff from Vieux Fort River Catchment when the carrying capacity of the lower segment of the river along the southern side of the site is exceeded. The chosen design storm was one which possessed features most closely resembling the storms known to cause flooding on this part of the island. The storm was used as inputs into suitable hydrologic procedures to obtain flow hydrographs for conducting the hydraulic analysis.

Table 1. Soil Classification by Hydrologic Soil Group
(According to the Natural Resources Conservation Service (NRCS) method)

Soil Group	Description	Infiltration Rate (mm/h)	Soil
A	Soils in this group have low runoff potential when thoroughly wet. Water is transmitted freely through the soil. Group A soils typically have less than 10 percent clay and more than 90 percent sand or gravel and have gravel or sand textures. Some soils having loamy sand, sandy loam, loam or silt loam textures may be placed in this group if they are well aggregated, of low bulk density, or contain greater than 35 percent rock fragments	8-12	Sand, loamy sand, sandy loam.
B	Soils in this group have moderately low runoff potential when thoroughly wet. Water transmission through the soil is unimpeded. Group B soils typically have between 10 percent and 20 percent clay and 50 percent to 90 percent sand and have loamy sand or sandy loam textures. Some soils having loam, silt loam, silt, or sandy clay loam textures may be placed in this group if they are well aggregated, of low bulk density, or contain greater than 35 percent rock fragments.	4-8	Silt loam, loam.
C	Soils in this group have moderately high runoff potential when thoroughly wet. Water transmission through the soil is somewhat restricted. Group C soils typically have between 20 percent and 40 percent clay and less than 50 percent sand and have loam, silt loam, sandy clay loam, clay loam, and silty clay loam textures. Some soils having clay, silty clay, or sandy clay textures may be placed in this group if they are well aggregated, of low bulk density, or contain greater than 35 percent rock fragments.	1-4	Sandy clay loam.
D	Soils in this group have high runoff potential when thoroughly wet. Water movement through the soil is restricted or very restricted. Group D soils typically have greater than 40 percent clay, less than 50 percent sand, and have clayey textures. In some areas, they also have high shrink-swell potential.	0-1	Clay loam, silty clay loam, clay, sandy clay, silty clay.

ii. Using suitable hydraulic procedures, the determined water levels from the passage of these hydrographs through the small watercourses on site and through Vieux Fort River are to be used for setting the solar panels.

b. As the roadway is expected to have low-traffic throughout the year, the sizing of the stream crossings was based on the estimation of peak flows for the 10-year return period for a storm equal in length to the catchment's time of concentration, or 10 minutes, whichever was longer. The sizing of the crossings would allow longer duration storms of higher return periods to pass without flooding. Very short storms of higher return periods will cause flooding of the roadway, but for a short time, provided that the geometry of the road design permits runoff to the surrounding lower lands.

c. Drainage within the farms is to follow the natural pattern existing before the development. It is going on the premise that the farms will have well maintained vegetated surfaces throughout; that is, not only of the strips between the columns of panels, but under the panels, themselves. The grade of the farm is to be limited to an acceptable level. The flows through the farm onto maintenance pathways within and then off to the surrounding slopes are to be controlled by the choice of grade within the farms, pathways and roads and the maintenance of well vegetated slopes for protection against erosion.

d. Roadside drainage is to be provided principally to protect the toe of the road embankment from runoff from surrounding lands that would have flown in a direction across the road before any road construction. Wherever possible, use is to be made of drains lined with vegetation. Slopes to protect against erosion and to prevent sedimentation will be determined.

e. Culverts are to be sized along the major tributaries that would be crossed by the roads. The number of such crossings are to be kept to a minimum; wherever possible, consideration will be given to consolidation of several small crossings into fewer ones. Any storage opportunity behind the culvert opening from the road embankment is to be maximized to reduce the size of opening required and to check flows to pre-development levels.

f. The consequence of concentration of flows from existing condition by any drainage intervention is to be mitigated by the installation of energy dissipators, such as catchpits, stilling basin or rip-rap.

4. Data sources

4.1. Design Rainfall

It is being assumed that the rainfall return period corresponds exactly with the flood return period. With this assumption, the rainfall for the required return period can be determined and subsequently be used for flow estimation using an appropriate rainfall-runoff procedure. For the definition of the design rain storm, it is required to specify the depth, its duration and its temporal distribution.

4.1.1 Duration

Two levels of output were required in the study, the first one for establishing the flood level for setting the solar panels and the other was for the design of the drains and crossing within the site. For the former, and as specified, the water level for the 100-year flood depth produced by the 24-hour rain storm is required and so a design storm of that length was required. For the determination of the design peak flow for the infrastructure design a shorter duration (equal to the time of concentration of the catchment, or at least ten minutes) was required.

4.1.2 Depth

The depth for the 24-hour rain storm was based on a statistical analysis of rainfall records from Hewanorra International Airport and information from several recent hydrologic studies. The Hewanorra station, which is nearby to Vieux Fort Catchment, has one of the longest rainfall databases on the island. Rainfall records from 1985 to 2016 were obtained and used for the analysis. The study also considered the report by Jetten et al. (2016) and Egis Eau Sociale (2014).

For the shorter duration events required for the design of the drainage infrastructure, use was made of the Intensity Duration Frequency Curve at Dennery, as reported in Egis Eau Sociale (2014).

4.1.3 Temporal Distribution

The rainfall database on the island consists mainly of manual recording of rainfall depth, which is expressed as a daily value. Few gauges record the depth accumulation with time, and as far as is known, no study has been done to develop a representative distribution over the island. When such a temporal distribution is needed, recourse is frequently made to the distributions developed by the Natural Resources Conservation Service (NRCS) for rainfall measured within North America. They have been classified into four types, Type 1, Type 1A, Type II and Type III according to region. It has been suggested by a few researchers that in the absence of any other locally available record, then the Type III distribution should be used for the Caribbean islands. Several rainfall stations recorded the depth variation with time for Hurricane Tomas and they are plotted in Figure 2, alongside the Type III distribution. A few comments can be made. The first is that the distribution across the island for Hurricane Tomas was also spatially uniform, at least in the south, as there is little to distinguish the distributions from the five gauges from Dennery to Ruby Soufriere. The rainfall pattern was near constant throughout. The Type III distribution, on the other hand, starkly departs from this pattern. It showed very light rainfall for the most part at the start and at the end, with a very intense period around mid-day. These widely differing distributions may have an influence on the observed runoff, but there is insufficient information to choose over the other. As a result, both distributions were considered for estimating runoff and a decision made as to which of the two types of distribution should be used for the estimation of flood depths within the site.

For the shorter duration storms for designing the drainage infrastructure, it was assumed that the rainfall was uniformly distributed over the time period.

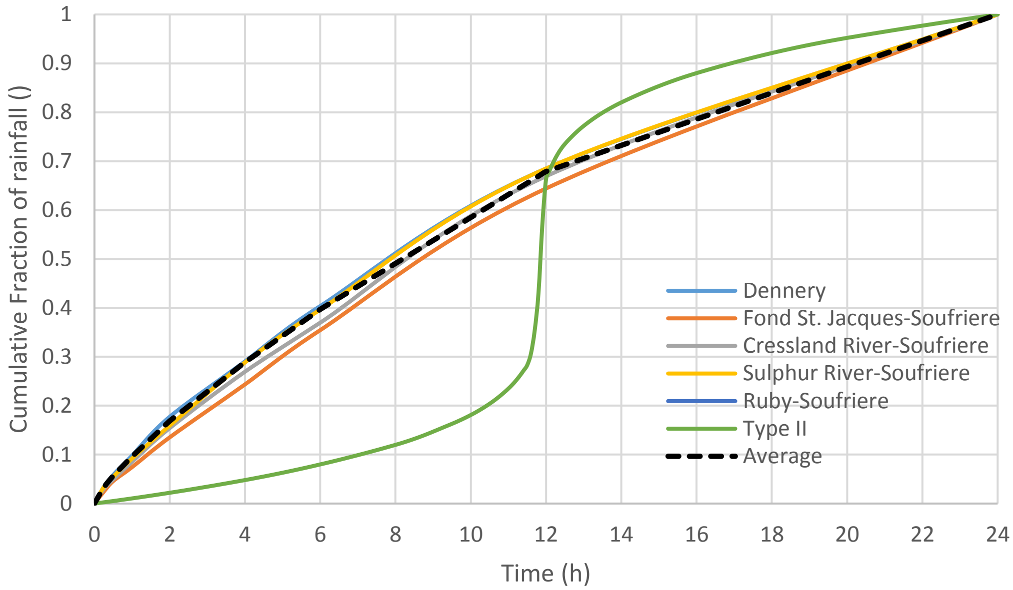

Figure 2. Rainfall distribution over a 24-hour period

4.2. Physical data

4.2.1 Soils

The Stark et al. (1966) report was the source of information for the types of soils within the site and in Vieux Fort River Catchment. The soils were subsequently classified according to the Natural Resources Conservation Service (NRCS) system, which is based on the infiltration capacity of the soils. Soils are placed into four hydrologic soils groups, A, B, C, D as defined by Table 1. The resulting classification is shown in Figure 3.

4.2.2 Land cover

Current land cover was obtained from the most recent Google Earth satellite imagery. The various land covers within the catchment are shown in Figure 4. The land cover types were eventually classified according to the NRCS procedure, which uses an index of runoff potential called the Curve Number (CN). This number varies from 1 to 100, the values increasing as runoff potential increases. Table 2 shows the area of each type of land cover present within Vieux Fort River Catchment and the CN values for each type.

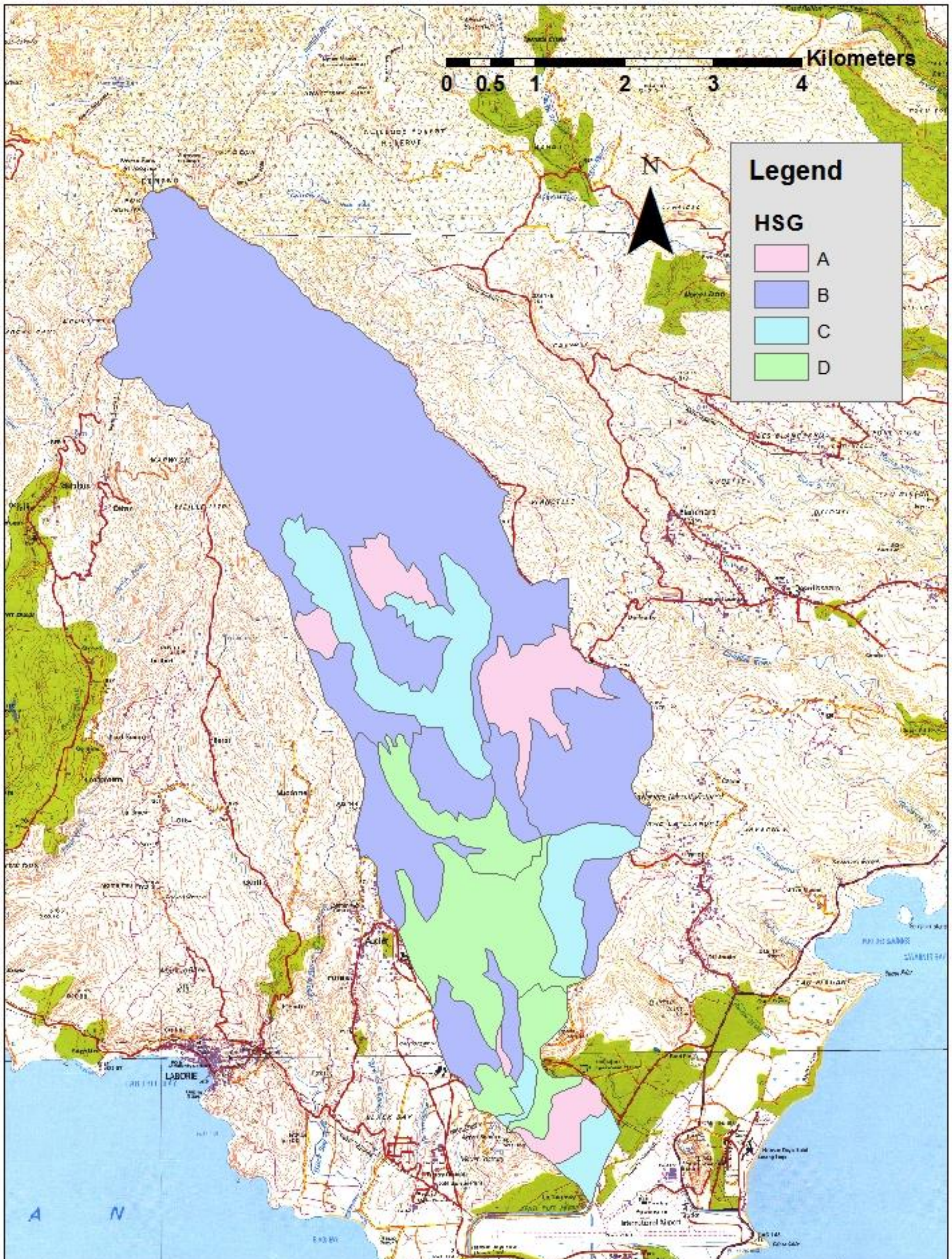

Figure 3. Soils map for Vieux Fort River Catchment

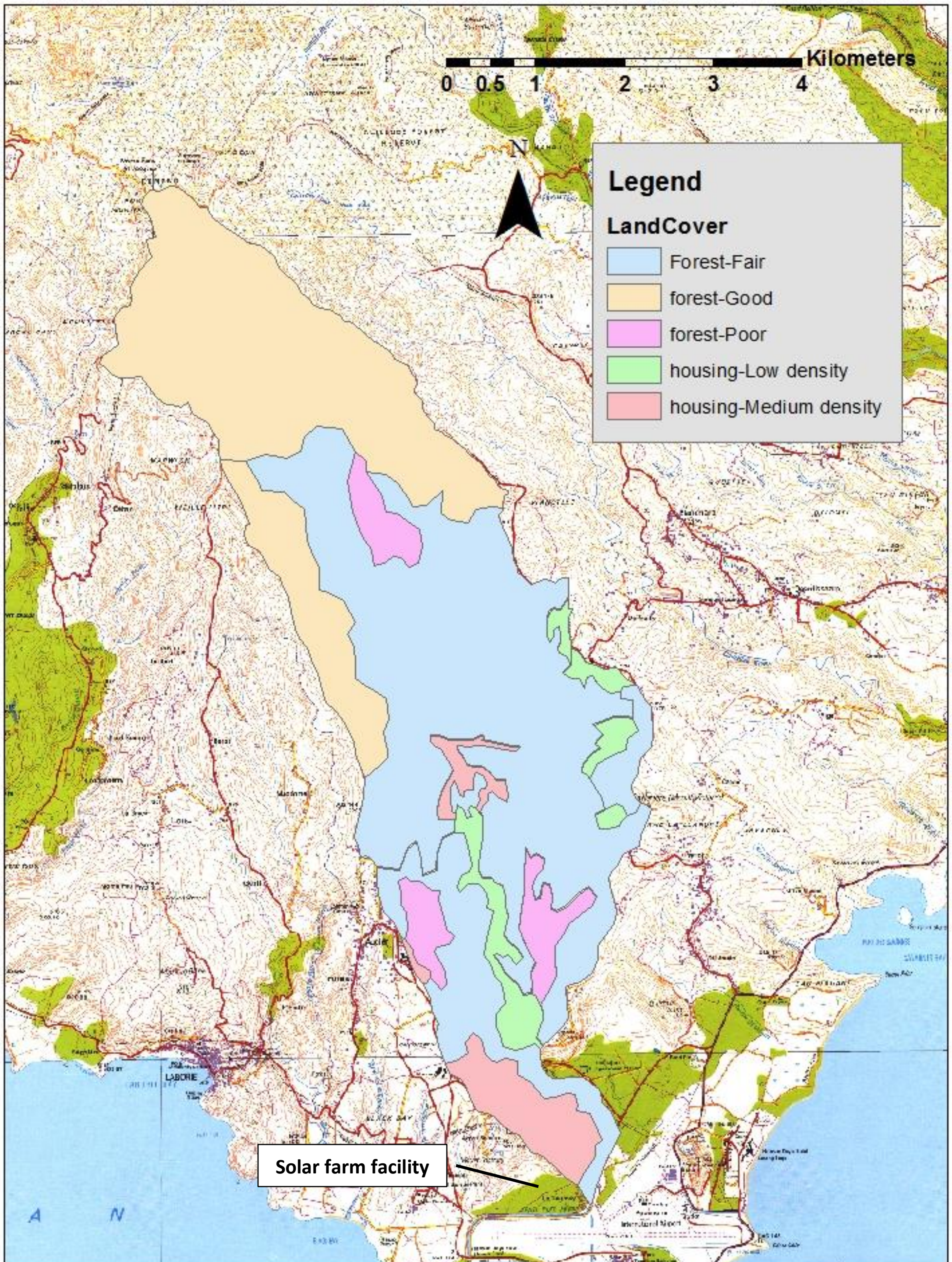

Figure 4. Land cover within Vieux Fort River Catchment

Figure 5. Vieux Fort River catchment and tributaries

Table 2. Land cover within Vieux Fort River Catchment

Land cover	Area present (ha)	CN for hydrologic soil group:			
		A	B	C	D
Forest-Good	877	30	55	70	77
Forest-Fair	1373	36	60	73	79
Forest-Poor	145	45	66	77	83
Housing-medium density	150	54	70	80	85
Housing-Low density	120	61	75	83	87

4.2.3 Topography

The relief within Vieux Fort River Catchment was obtained from the 1:25000 series map for St. Lucia, which is presented at 25-foot contour intervals. The highest elevation was 2150 feet (656 m) above mean sea level (MSL) and the outlet was 25 feet (7.6 m) above MSL. The catchment area was 26.7 km^2, the catchment length was 17.9 km and the longest stream path was 17.4 km. The time of concentration, t_c, given by the Kirpich formula (below) was 132 minutes. Figure 5 shows the catchment area of Vieux Fort River.

$$t_c = 0.0078 L^{0.77} S^{-0.385}$$

where, L was the channel length (ft) and S is the stream slope (ft/ft).

River cross-sections were taken at regular intervals along the lower Vieux Fort River to define its geometry for performing a hydraulic analysis. Additional topographical details of the site were obtained from engineering surveys.

5. Method

5.1. Determination of Water Levels for Installation of Solar Panels

At the heart of the output for design was the use of two hydrologic procedures and one hydraulic model:

- The HEC-HMS hydrologic model was used for the estimation of the flow hydrograph from Vieux Fort River Catchment and the Rational Method was used to estimate peak flows for the design of the culvert crossings within the site.
- The Flo2D model was used for the estimation of flood levels produced by the chosen design storm on the site and within Vieux Fort River Catchment.

5.1.1 Design storms

For the 24-hour design storm:

The thirty-one years of rainfall records were first examined with a view to determining the distribution to which they might have belonged. Three most likely distributions, namely normal and log normal distributions, and Gumbel distribution, were tested by plotting the records on probability paper for each distribution. The one which plotted as a straight line would then have been chosen for estimating the 100-year rainfall depth. The Gumbel plot on Gumbel probability paper, shown in Figure 6 was typical for all the others. None of the plots was satisfactory as five values (four shown here) consistently departed from the linear fit; three of these were rain depths from storms Debbie, Matthew and Tomas.

Jetten et al. (2016) suggested that it perhaps was better to fit the rainfall records to the generalized extreme value distribution instead of assuming that its special case, extreme value Type I applied, which is the Gumbel distribution. The authors produced a comprehensive analysis of the rainfall and eventually estimated depths for the various return periods, up to the 50-year frequency. It is to be noted that that depth was 285 mm.

Their approach was followed and extended to the 100-year level, taking note of the parameter values they had gotten for the application of the GEV method. The resulting depth was 317 mm.

Taking into the various suggestions by researchers for accounting for the uncertainty in future rainfall due to climate change, a factor of 1.1 was applied to this depth, resulting in a magnitude of 352 mm for the daily rainfall.

With this depth, two sets of design storms were produced, one using the Hurricane Tomas temporal distribution, the other using the Type III distribution from the NRCS procedure.

For the shorter duration design storms:

- The time of concentration was estimated for catchment 1, using Kirpich formula.
- The Dennery IDF curve was then used to determine the rainfall intensity at the 10-year return period.
- The nominal time of concentration of 10 minutes was set as the value for all the other catchments within the site.
- The Dennery IDF curve was then used to determine the rainfall intensity at the 10-year return period.

5.1.2 Estimation of flows

Flow estimation for the 24-hour design storm for all the catchments was done using the NRCS procedure in HEC-HMS. A composite CN value was estimated for each catchment and the time lag required for developing the unit hydrograph was determined from the physical features of the catchment and used as input into the model. In all, flows were generated for the two design storms being considered, one using the Type III distribution, the other using the Hurricane Tomas distribution.

For comparison, flow estimates were also done for Hurricane Tomas as this event generated the largest rainfall depth on record at the rainfall station at Hewanorra Airport.

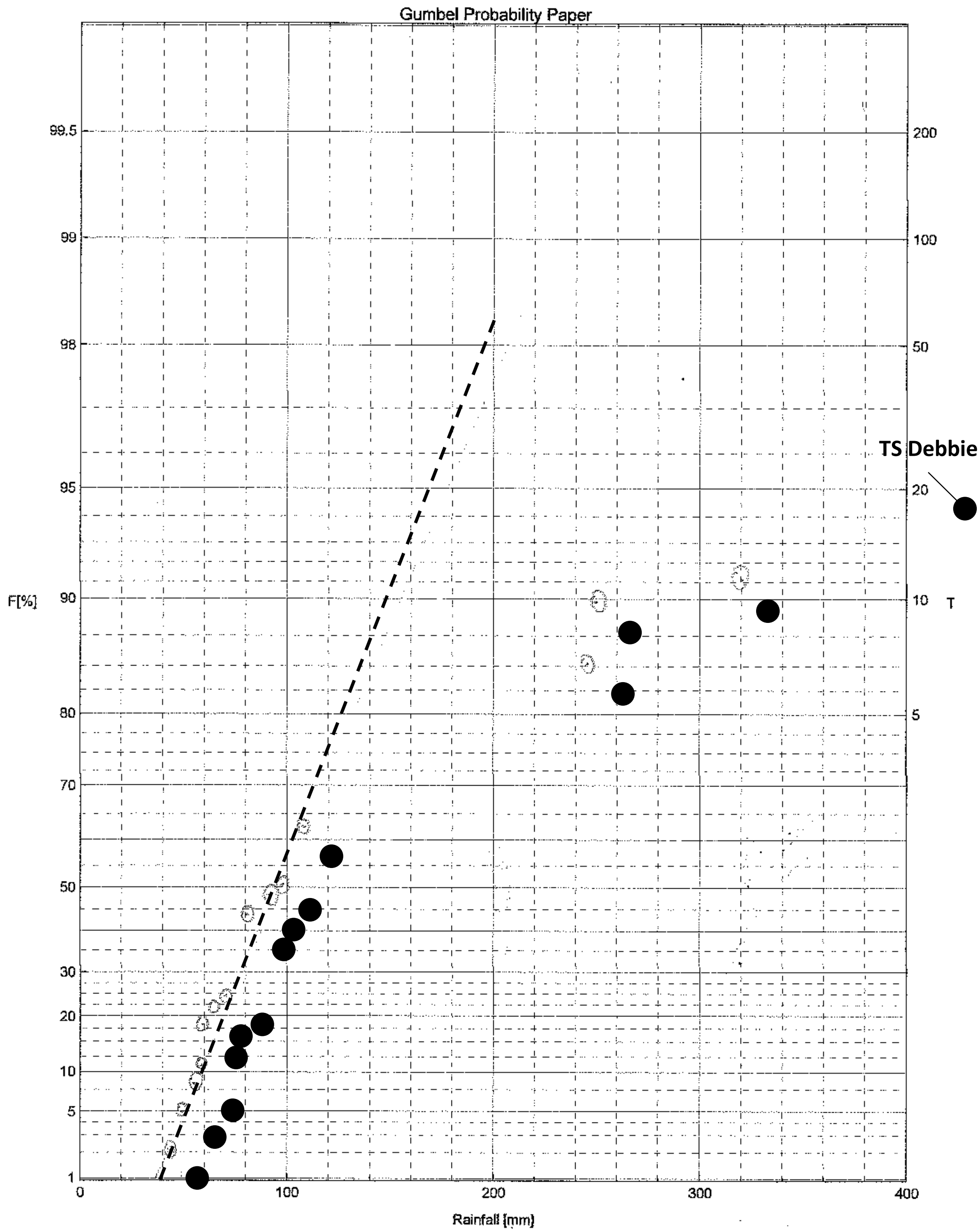

Figure 6. Data points plotted on Gumbel probability paper

5.1.3 Estimation of water levels

The hydrologic conceptualization of the site for estimating water levels was as follows:

The site was assumed to have been receiving flow inputs from:

- Vieux Fort River, which runs along its southern boundary and may inundate the site wherever flow levels within the river exceed the right embankment. Inundation of the site depended then on its topography.
- The small surrounding catchments flowing into the site via small streams. There were four such catchments, one to the northwest from the Mount Tourney area, and three small ones from the surrounding La Tourney Hill. These streams discharged onto the site at specific locations.
- Rain falling directly over the entire site which may cause inundation, depending on the topography.

The site was assumed to have been drained mainly through the south-western section and so specific locations were identified. The Flo2D model was set up by defining the boundary within which water received from river and stream inputs and from rainfall were distributed and eventually discharged. The grid size for the 2D computation was set at 20 metre x 20 metre cells, the time step for the analysis was set between 0.1 sec and 30 sec. For the farms and surrounding lands, the Manning roughness was set at 0.05 consistent with the dense growth cover on the site. The value for the river channel was set at 0.04, consistent with the advice from Barnes (1967). The feasible flow values from the HEC-HMS runs were then in turn input into Flo2D to obtain the corresponding output of water levels over the site.

The water level output from the model for each run was prepared for plotting in ArcGIS against the topographical features of the site for observing the variation of flood level over the site for the extreme rain storm. The flood level map was then used to advise on the level at which the solar panels should be set.

5.2. Stormwater management infrastructure

The Rational Method was used for determining the peak flows to size the drain segments along the roads and the road crossings of streams within the site. The geometry of the drain segments was set so as to accommodate the design flow and to limit erosion and deposition of materials in the channels. The road crossings were sized assuming that for passage of the design flow, the maximum water elevation on the upstream side would be below the road deck.

5.2.1 Conceptual layout

The proposed stormwater management system with the various channel segments and road crossings is shown in Figure 7. This system can be schematically represented as consisting of three independent sub-drainage systems seen in Figure 8. The lines represent channel segments defined between two nodes.

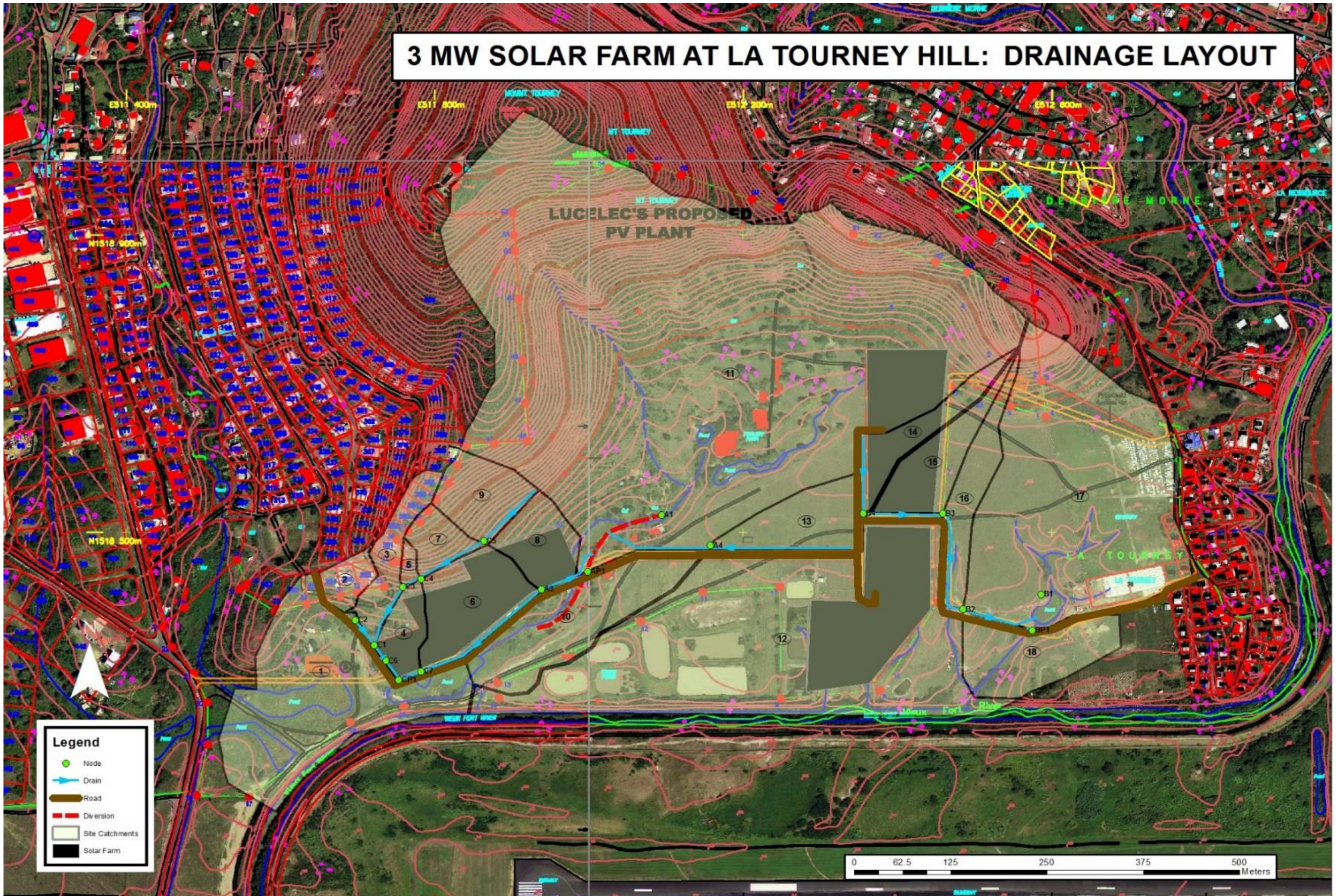

Figure 7. Catchments for stormwater design

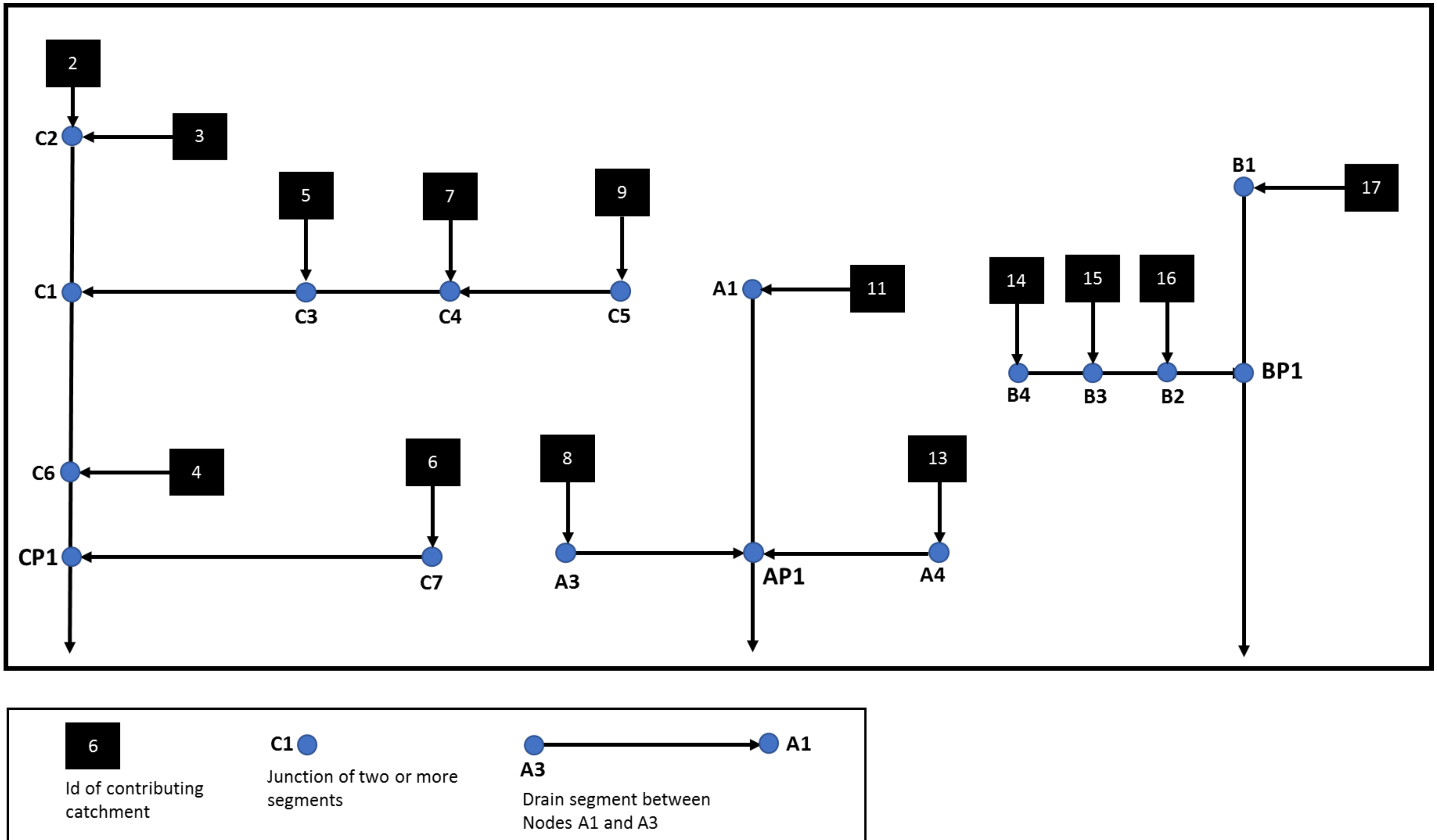

Figure 8. Schematic of the drainage network on the LUCELEC solar farms at La Tourney

The nodes were locations where either particular drainage structures, such as culverts and catchpits were sited, or increased inflows from a catchment occurred. The outlet of each sub-system is at one of three culverts that drain into natural watercourses before entering Vieux Fort River. The locations of these culverts, which can be seen in Figure 7, are: (i) a culvert to the southeastern side of Solar Farm 7C (SF7C) (node CP1); a culvert to the southwestern side of SF7C (node AP1); and a culvert to the northeastern side of SF7B (node BP1). The square symbols represent catchments that flow into the drain segment, with the numbers within being the catchment labels in Figure 7.

5.2.1.1 Sub-system C

Sub-system C drains through Culvert CP1 into a low lying swampy area. The access road through the site forms a dyke and disrupts the natural drainage to the southwest, so this culvert is required to permit flow across the roadway. Flows are from the southwestern catchments whose land covers would undergo very little modification under the planned development. There is, however, some increase in flows as a result of a diversion of flows from Catchment 9 into this sub-system via channel segment C5 – C4.

5.2.1.2 Sub-system A

Sub-system A drains through Culvert AP1 into a natural watercourse. This is the major tributary within the site and it drains a substantial fraction of the site. A substantial amount of changes in land cover is expected to occur within this catchment and would result in an increase in flows. However, flows from Catchment 9 are to be diverted to Sub-system C, via channel segment C5 – C4. The access road does disrupt the natural flows and so Culvert AP1 is required for discharge to Vieux Fort River.

5.2.1.3 Sub-system B

Sub-system B drains through Culvert BP1 into a tributary of Vieux Fort River. Part of the flows from Sub-system A are diverted by roadside drains. Some increase in flow is expected as a result of the modification of the land cover by the solar farm. The proposed road for access from the east disrupts the natural drainage system and so Culvert BP1 is required for discharge to Vieux Fort River.

5.2.2 Catchments

Figure 7 shows the catchments contributing to the various drainage segments within each sub-system. Their boundaries were determined from a 2-metre contour map of the site and its surroundings. A satellite image provided very detailed current land cover information that was used to determine the existing runoff coefficient of each site. A substantial fraction of the land is covered with vegetation, the diversity in terms of density, clearly being observed from the satellite image.

The master plan for the site was used to determine the expected runoff coefficient after buildings, roads and solar panels have been installed. All but two of these catchments have slopes steeper than 10%, with some being as steep as 35%. As a result, time of concentrations are very short, all being less than ten minutes (as estimated with Kirpich formula).

5.2.3 Peak flow estimation

Estimation of the peak flows for each catchment with the Rational Formula was done by first estimating the runoff coefficient, assuming that the proposed plans for site development would be realised. Use was made of the Dennery Intensity-Duration-Frequency curve as it was the closest available one to the site. It has been assumed that the duration of the design storm is a minimum of ten minutes. A factor of 1.2 for these short duration storms was applied to account for the uncertainties of climate change.

The runoff coefficient for each the solar farm was found from consideration of the proposed configuration, which is to consist of rows of inclined panels with swaths of unaltered land (with its native vegetation intact). Such a proposed arrangement results in about two-thirds of the land covered by the impervious solar panels. Using this information, the estimated runoff coefficient of the farms was 0.8 based on the impervious areas and unaltered lands having runoff coefficients of 0.9 and 0.5, respectively.

The runoff coefficients for each catchment and their estimated peak flows are shown in Table 3.

Table 3 – Peak Flows from the Site Catchments

Name	Area (ha)	Intensity (mm/h)	C	Clim. Chng factor	Peak Flow (m^3/s)
1	4.817	130.00	0.50	1.20	1.04
2	0.385	130.00	0.80	1.20	0.13
3	0.875	130.00	0.66	1.20	0.25
4	0.600	130.00	0.67	1.20	0.17
5	0.113	130.00	0.61	1.20	0.03
6	1.553	130.00	0.59	1.20	0.39
7	0.520	130.00	0.80	1.20	0.18
8	0.705	130.00	0.67	1.20	0.28
9	1.258	130.00	0.80	1.20	0.35
10	2.707	130.00	0.60	1.20	0.64
11	27.745	130.00	0.63	1.20	7.34
12	10.705	130.00	0.59	1.20	2.72
13	1.399	130.00	0.60	1.20	0.36
14	1.193	130.00	0.58	1.20	0.22
15	0.966	130.00	0.60	1.20	0.28
16	1.685	130.00	0.60	1.20	0.29
17	8.254	130.00	0.64	1.20	2.26
18	0.790	130.00	0.71	1.20	0.48

5.2.4 Sizing culverts

Use was made of the HY-8 procedure of the Federal Highways Administration (FHWA). It has been assumed that storage may occur upstream of these culverts because of the road dyke. Minimal alteration of the lands near crossings at CP1 and BP1 is to be done. For the crossing at AP1, some earthworks are required to develop the required stream diversion. This provides an opportunity of creating storage upstream of the crossing, which may ultimately reduce the size of the culvert opening and the diversion section downstream.

All culverts have been taken to be two metres longer than the road deck width. It was also assumed that they would all have wing walls at 45° and that their inlet configuration would be of a square edge with a headwall. The assumed Manning number was 0.014.

The performance curves considered for the following culvert sizes at the three locations are shown in Table 4.

Table 4 – Performance Curves for Culverts

Location	Performance curve for concrete pipes of diameter (mm):
CP1	900
AP1	900
	1200
	1500 (Twin barrel)
BP1	900
	1500

5.2.5 Channel segments

The decision for the type of lining for the channel segments is to be based on the following considerations:

- The protection of the solar farms and the road infrastructure from water erosion;
- The available area for the drain as this would determine the allowable velocity; and
- The total cost both for their installation and the subsequent maintenance to ensure the drain continues to function.

The options available for lining are the use of precast concrete segments, grass-lined channels, or composite cross-sections consisting of precast concrete lining at low flow depth with grass lining at higher depths. All options were considered in determining the preferred lining and the following was decided:

1. For the catch drain or interceptor drain along the northern side of SF7C, the lining should be a composite of precast lined at low depths with grass lining to the top of the drain section. The main reason was to ensure that the form of the drain can be preserved without much maintenance. This will ensure that it continues to protect the solar farm from possible high flow velocities, were the drain to become overgrown.

2. For all other roadside drains within the development, solely grass-lined drains can be used as they are more accessible and more likely to be easily and frequently maintained.

Where the flows of the roadside drains enter Culvert CP1, catchpits are to be installed to reduce scour.

At the other culvert locations, Culvert AP1 and Culvert BP1, catchpits are not being proposed. Rather, the roadside drains are to be mitred away from the openings and flow into the natural watercourses that are draining toward the culverts.

A catchpit is to be installed at the eastern end of the road between 7A and 7B to accommodate the change in direction at the intersecting channels.

5.2.6 Energy dissipators

Energy dissipators are required at the downstream end of the catch drain where it flows into the roadside drain. Riprap or gabion baskets can provide such a function and is to be designed.

Energy dissipators are required on the downstream ends of the three culverts and are to be designed. Some riprap may be needed where the roadside drains enter the natural watercourses, near Culvert AP1 and Culvert BP1, and is to be designed.

6. Outputs for the solar farms

6.1. Floodwater distribution over the site

The water levels expected for the 100-year design storm are shown in Drawing 1 at the end of this report. It was based on the design storm that used the Hurricane Tomas temporal distribution as water levels from this event were regarded as being more representative of actual occurrence of flooding for this magnitude storm.

The drawing shows that flood levels from Vieux Fort River may encroach Solar Farm 7C on its southeastern end to a depth of up to 0.9 metres, for an area less than 150 m^2. The flooding is largely from runoff from a nearby tributary. Even though there is backwater from Vieux Fort River, this does not extend to within the proposed site for Solar Farm 7C. Some encroachment may occur in small sections of Solar Farms 7A and 7B but this is due solely to ponding of water from rainfall in localized depressions, and the depth is not greater than 0.15 metres. Because of their relatively high elevations, there is no flooding of these sites from Vieux Fort River.

Detailed flood levels on the solar farms are shown in Drawing 2 and Drawing 3.

The estimated flooding for Hurricane Tomas is also shown in Drawing 4. Significantly greater areas were inundated by this hurricane, including parts of Solar Farm 7A and Solar Farm 7B. This storm, however, is regarded as being much greater than a 100-year event and so these flood levels were not used.

6.2. Stormwater management System

6.2.1 Culvert crossings

Table 5 provides details for the three culvert crossings. Based on the assumed invert elevations at the crossings, at the natural slope, and a cover of 300 mm above the crest of the culvert to the road surface, single barrel concrete pipes at CP1 and BP1 are possible. At CP1, no flooding of the land upstream of the crossing is expected, provided that the approach channel immediately upstream of the crossing is trapezoidal with a bottom width of 0.9 m.

At BP1, the 900 mm diameter culvert may be able to convey the design flows without overtopping the roadway, if storage is allowed upstream of the opening. This is a feasible option as this location currently has no hindrance to such storage. Additionally, there is evidence that ponding does occur at this location. A 1200 mm barrel also is not sufficient to convey the design flow of 3.05 m^3/s without some storage upstream. However, if storage is indeed allowed, the requirement is reduced from that required for the 900 mm barrel. A 1500 mm diameter barrel can safety convey the design flow without the need for any storage upstream.

At AP1, diversion works are to be performed on the natural watercourse to minimize the threat of flooding to SF7C from high flows. The road embankment may cause storage to occur on its upstream side. It is hydraulically possible to use a 900 mm diameter culvert and prevent flooding of the roadway, provided the road elevation is at least at 5.2 m MSL. A slightly lower elevation is required if a 1200 mm diameter barrel is used. If no storage is to be allowed, then a double barrel 1500 mm culvert crossing is required. If the latter culvert configuration is chosen then channel works also are to be done to the approach and the tailwater segments, with a trapezoidal section, having a bottom width being at least 3.3 metres wide.

Energy dissipation is to be provided by using riprap or gabion baskets extending to at least 3 metres for diameters of at least 1200 mm for the 900 mm barrels, the protection should extend at least 2 metres.

In all cases, the road deck at the crossings should be covered with a hard surface, up to 5 metres on either side of the centreline of the culvert.

Table 5 – Possible culvert sizes at the road crossings

Position	Description	Invert Elevation (m MSL)	Road deck Elevation (m MSL)	Design Requirements	Comments
CP1	Culvert at south-western side of 7C	2.00	3.50	Concrete circular 900 mm dia. With square edge.	Adequate for design flow
AP1	900 mm Culvert at southeastern side of 7C	3.54	4.740	Concrete circular 900 mm dia. With square edge.	Conveys 1.38 m^3/s of the 8.00 m^3/s design flow
AP1	1200 mm Culvert at southeastern side of 7C	3.54	5.040	Concrete circular 1200 mm dia. With square edge.	Conveys 2.6 m^3/s of the 8.00 m^3/s design flow
AP1	Two 1500 mm barrels Culvert at southeastern side of 7C	3.54	5.340	Concrete circular double barrel 1500 mm dia. With square edge.	Adequate for design flow
BP1	900 mm Culvert on the eastern end of the site	5.00	6.200	Concrete circular 900 mm dia. With square edge.	Conveys 1.37 m^3/s of the 3.05 m^3/s design flow
BP1	1200 mm Culvert on the eastern end of the site	5.00	6.500	Concrete circular 1200 mm dia. With square edge.	Conveys 2.64 m^3/s of the 3.05 m^3/s design flow
BP1	1500 mm Culvert on the eastern end of the site	5.00	6.800	Concrete circular 1200 mm dia. With square edge.	Adequate for design flow

6.2.2 Channel Segments

Table 6 provides details on the drainage sections that can be used for conveyance of the 1 in 10-year peak flows within the site. It shows that nominally sized trapezoidal earthen sections of bottom width 0.5 metres and at grade no steeper than the natural grade are sufficient for all segments of the stormwater management system. The recommended grade for the catch drain is 1.5%. For cases where the design grade is less than the natural grade, scour checks are required to encourage siltation and so achieve the grade. There are other cases (the ones shown in italics) where velocities are greater than the allowable 1.8 m/s. With these high velocities channel erosion may ensue. One such case, the catch drain, should have a concrete lining and so can safely convey such high velocities. In the other cases, relatively moderate exceedance has occurred. Scour checks should also be installed, along with provision for conveyance on the channel banks.

Table 6 – Channel segments for storm water management on the site

Location	Shape	Wdth (m)	Dpth (m)	SideSlp (m)	Slpe (m/m)	Manning n	Q cap m3/s	Velocity m/s	Q des m3/s	Contributing Catchment	Natural slope (m/m)
C2 - C1	Trapezoid	0.500	0.330	1.000	0.015	0.025	0.445	1.6	0.38	2,3	0.015
C5 - C4	Trapezoid	0.500	0.300	1.000	0.015	0.025	0.372	1.5	0.35	9	0.083
C4 - C1	Half circle	0.450	0.300	1.000	0.030	0.015	0.807	3.6	0.56	5,7,9	0.050
C1 - C6	Trapezoid	0.500	0.450	1.000	0.020	0.025	0.936	***2.2***	0.94	2,3,5,7,9	0.027
C6 - CP1	Trapezoid	0.500	0.500	1.000	0.020	0.025	1.120	***2.3***	1.12	2,3,5,7,9,4	0.027
C7 - CP1	Trapezoid	0.500	0.300	1.000	0.020	0.025	0.429	1.8	0.39	6	0.027
A3 - AP1	Trapezoid	0.500	0.240	1.000	0.020	0.025	0.684	1.6	0.28	8	0.041
A1 - AP1	Trapezoid	3.300	0.7	1.000	0.01	0.025	7.34	***2.6***	7.34	11	0.015
A4 - AP1	Trapezoid	0.500	0.300	1.000	0.016	0.025	0.386	1.6	0.36	13	0.016
B4 - B3	Trapezoid	0.500	0.200	1.000	0.027	0.025	0.238	1.7	0.22	14	0.027
B3 - B2	Trapezoid	0.500	0.350	1.000	0.020	0.025	0.575	1.9	0.50	15,14	0.029
B2 - BP1	Trapezoid	0.500	0.420	1.000	0.020	0.025	0.818	***2.1***	0.79	15,14,16	0.029
B1 - BP1	Trapezoid	1.500	0.500	1.000	0.015	0.025	2.400	***2.4***	2.26	17	0.029

The diversion requires a widened section (3.30 m or 10 feet) at a gentler grade (1%) than currently exists to convey the peak flow. This section is to continue downstream of the road crossing into the existing natural drain. A transition section is required at the location where the diversion ends into the natural watercourse.

7. Conclusion

The site chosen for the solar farms at La Tourney Hill may be subject to flooding from Vieux Fort River and from several small steep catchments surrounding it. Furthermore, to the southwestern end of the site, there are some marshy areas from frequent flooding by Vieux Fort River. Therefore, safeguarding the solar panels is required and the TOR has recommended that they be set to above 600 mm of the 100-year flood from the 24-hour rainfall event.

Thirty-one years of rainfall records from the nearby Hewanorra International Airport were available for analysis to determine the design storm for estimating water levels for the installation of the solar panels. The records were used to perform a statistical analysis using the GEV distribution for estimation the 100-year daily rainfall depth and a factor of 1.1 was applied to include the impacts of climate change on future rainfall. The temporal distribution observed from the passage of Hurricane Tomas in 2010 was chosen as it gave reasonable flow hydrographs using the NRCS hydrologic procedure. The FLO2D hydraulic model was subsequently used to estimate water levels on the farm sites, taking into account the possibility of sea level rise from climate change. It has been assumed that there will be no change in the embankment of Vieux Fort River, on its floodplain with the airport.

The resulting flood maps showed that Solar Farm 7C may experience flooding at its southeastern section but only about 20 m^2 will be above a depth of 0.6 m. Flood depths over very small segments of Solar Farm 7A and Solar Farm 7B do not exceed 0.15 m.

Solar farms have the potential for causing serious adverse hydrologic impacts, including increased runoff, increased soil erosion and subsequent increase in sediment loads to nearby streams. Stormwater management measures have been recommended for the site with a view to minimizing these adverse impacts. It has been founded on maintaining, as far as possible, the natural drainage patterns of the site. It is to be noted that achieving this requires frequent maintenance to ensure that the site remains well vegetated.

The proposed stormwater management options are to provide protection of the base of the solar panels against erosion from runon from surrounding lands, to minimize increased sediment loadings into Vieux Fort River and to protect the road embankments. Grass-lined channels are being recommended throughout the site. A catch drain to the north of Solar Farm 7C is to be lined with concrete. The recommended culvert sizes at the stream crossings are to ensure that overtopping does not occur over the roads and to also check flows existing the site into Vieux Fort River. Hard surfaces should be laid near the crossing to minimize damage of the roadway by high flows.

The diversion recommended near Solar Farm 7C is to protect it from flooding.

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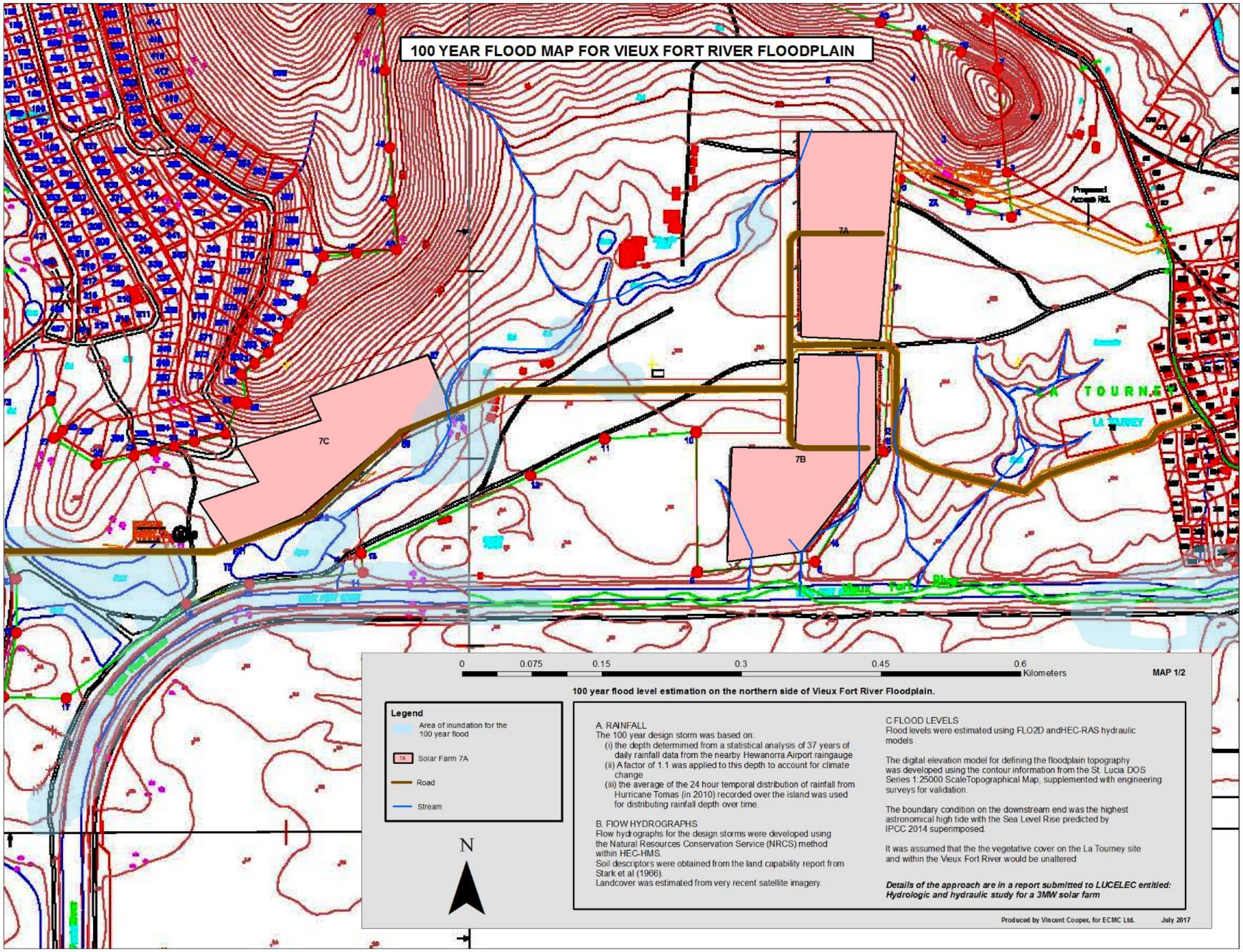
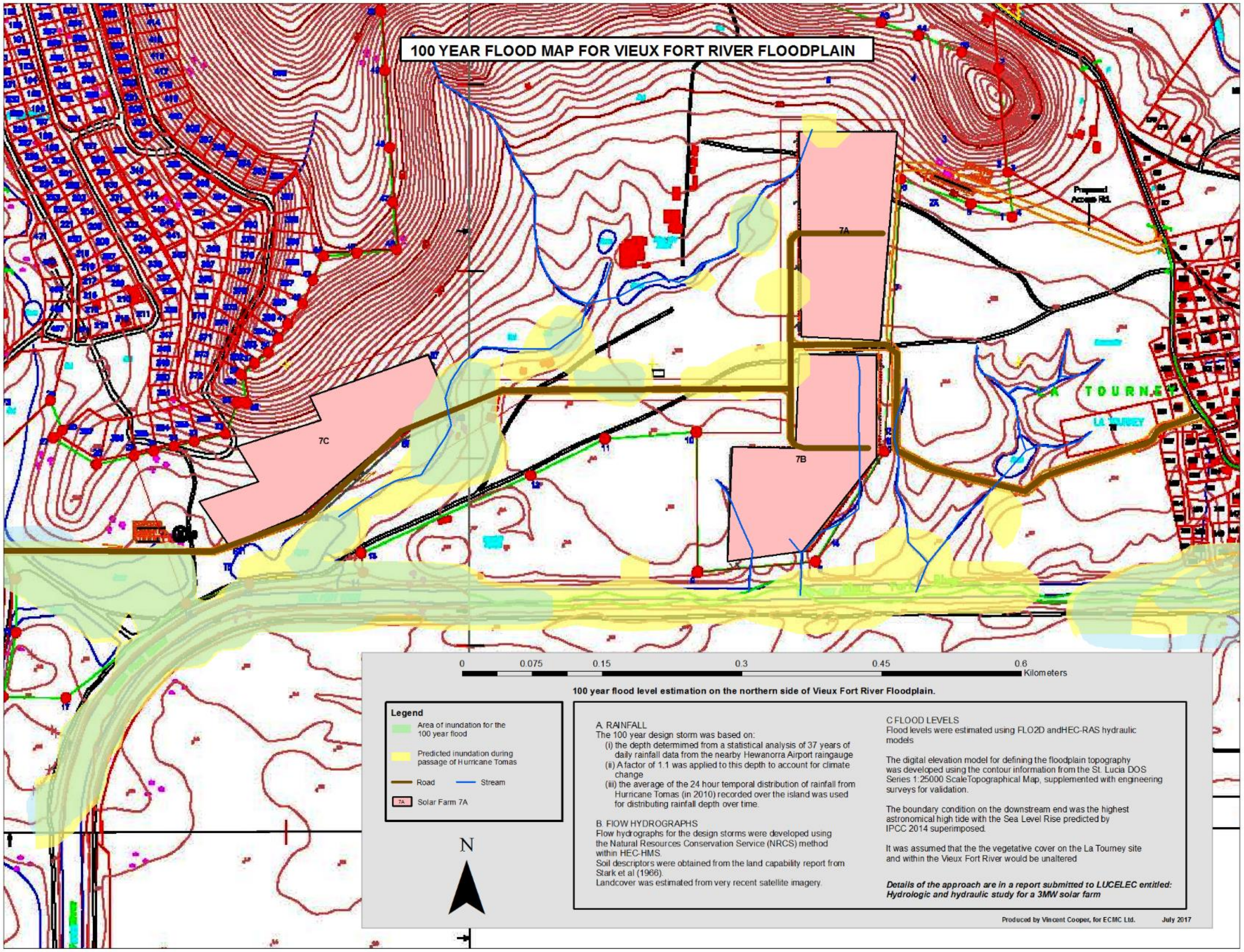
Appendix 1

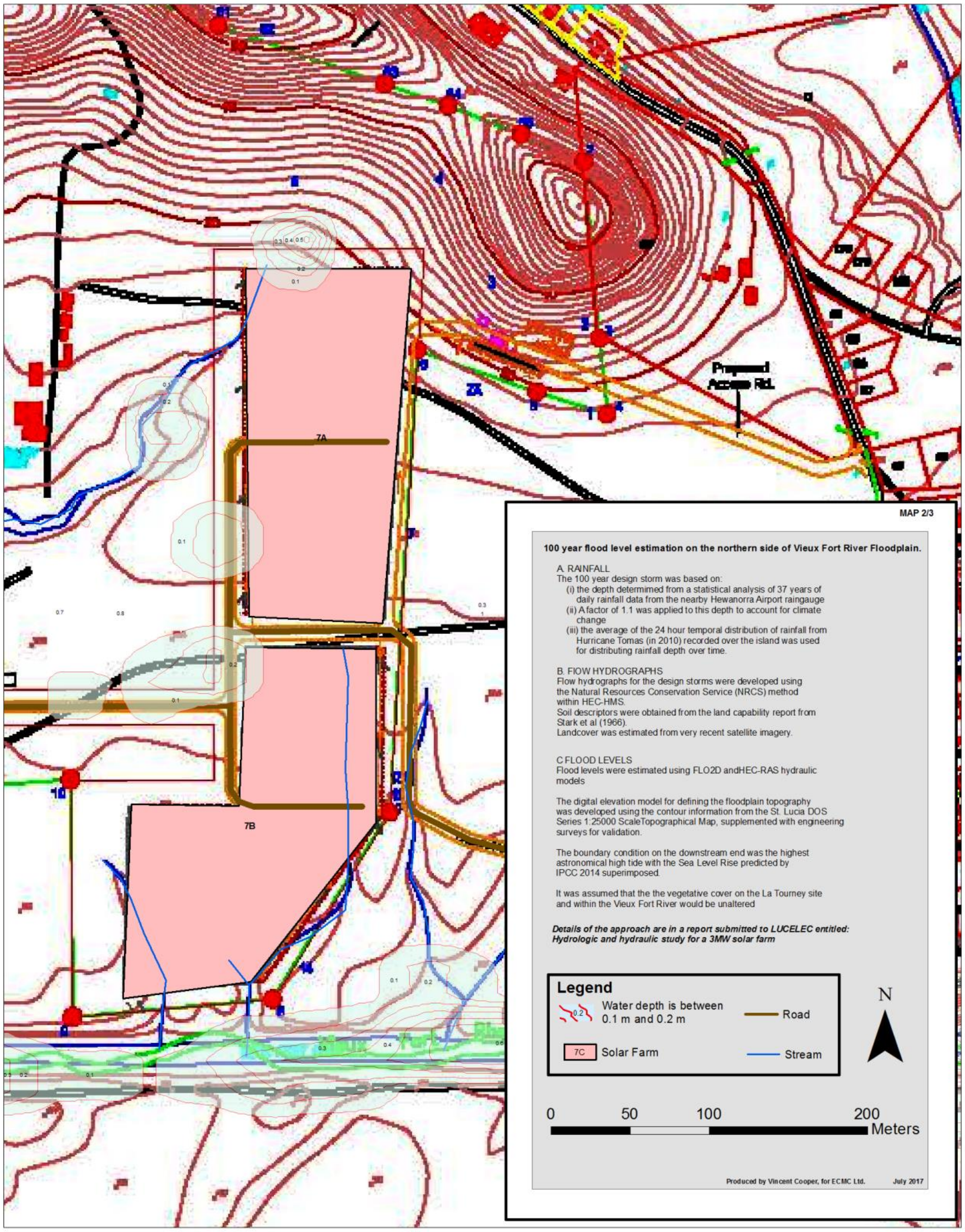


Drawing 1. Flood levels for the 100-year design storm

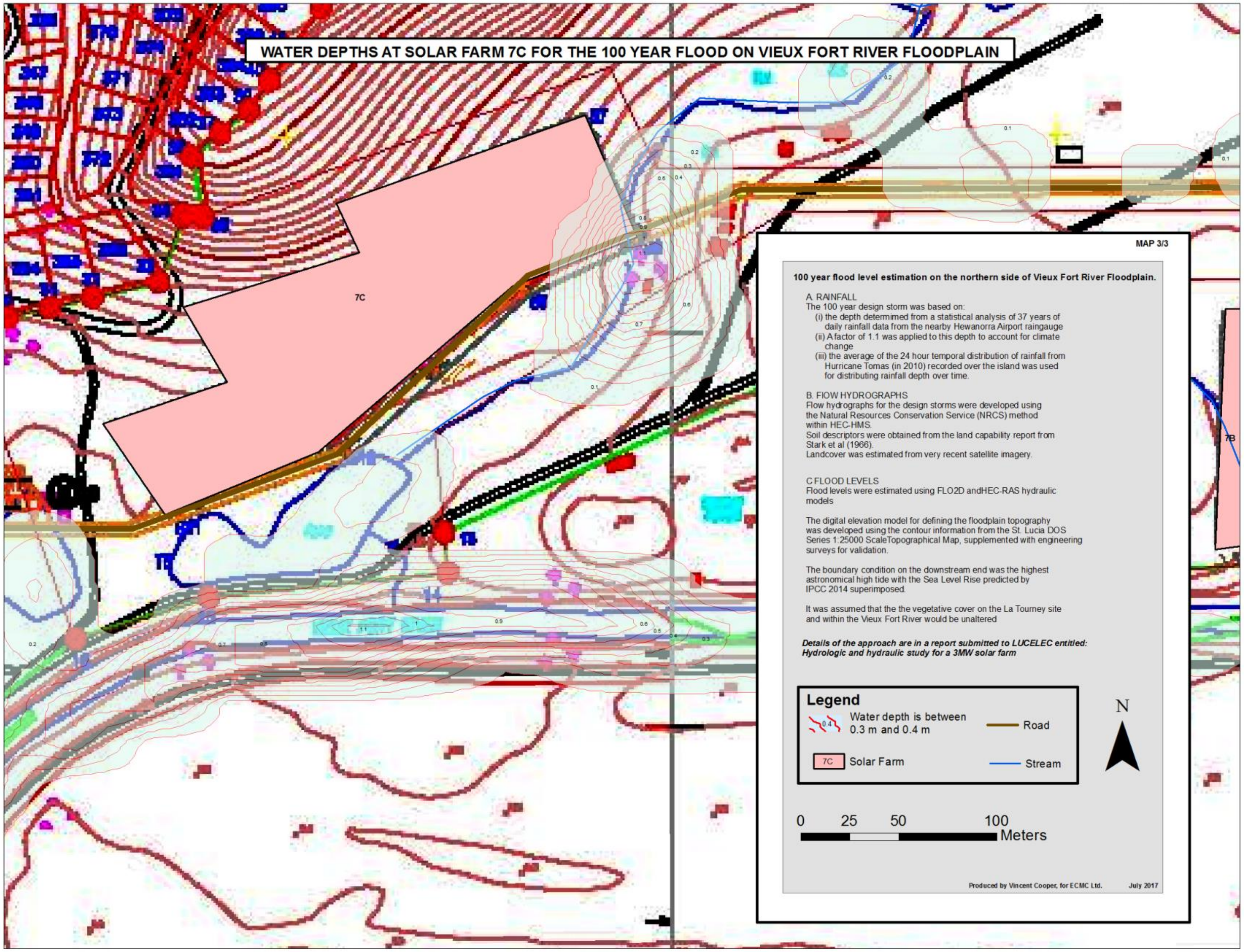


Drawing 2 Relation between flooding from Hurricane Tomas and the 100-year flood level

WATER DEPTHS AT SOLAR FARMS 7A AND 7B FOR THE 100 YEAR FLOOD ON VIEUX FORT RIVER FLOODPLAIN



Drawing 3. Flood levels within and in the vicinity of Solar Farms 7A and 7B.



Drawing 4. Flood levels within and in the vicinity of Solar Farm 7C

APPENDIX 2

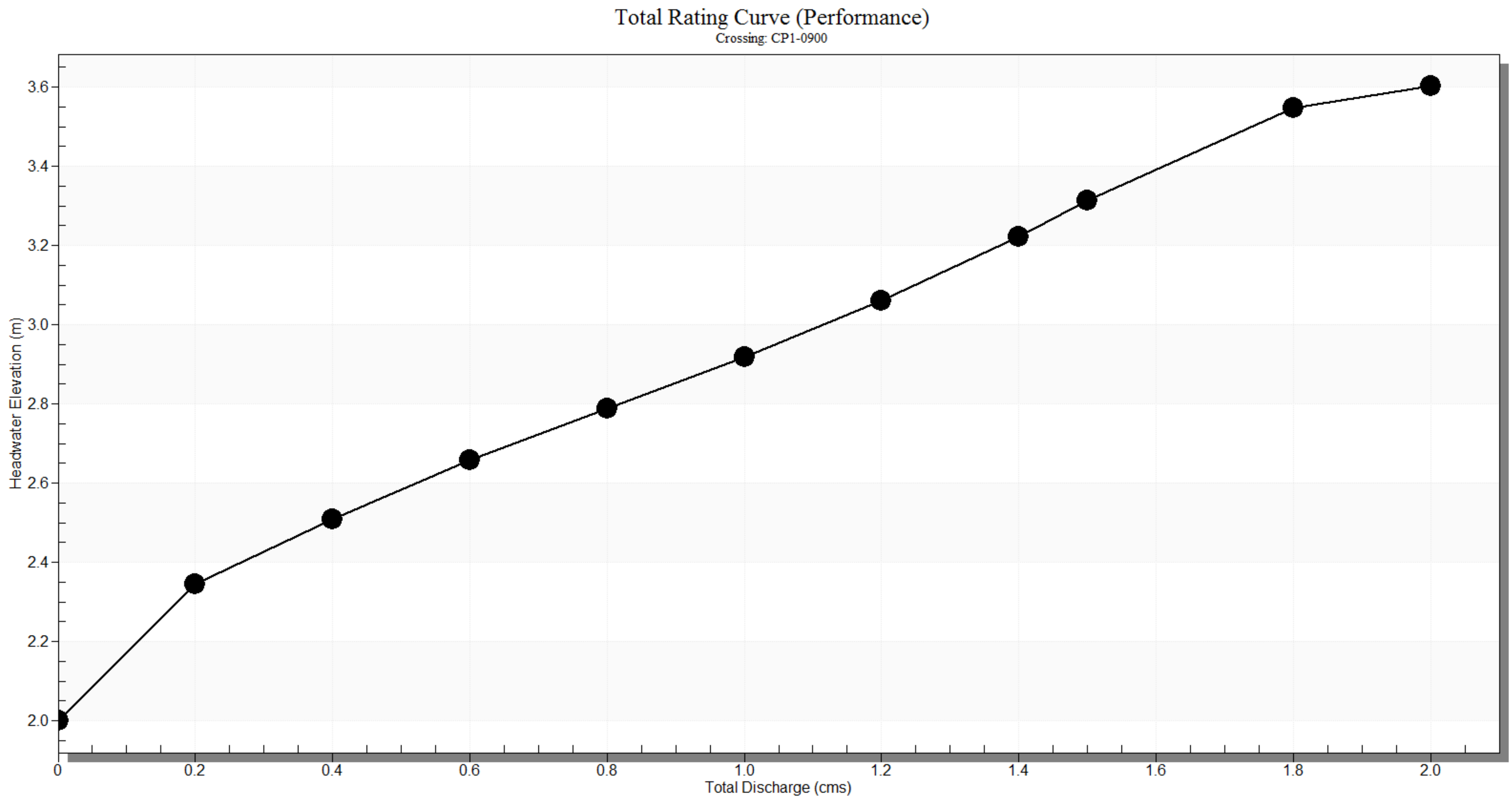


Figure 1. Rating curve for CP1 — Single barrel 900 mm culvert crossing

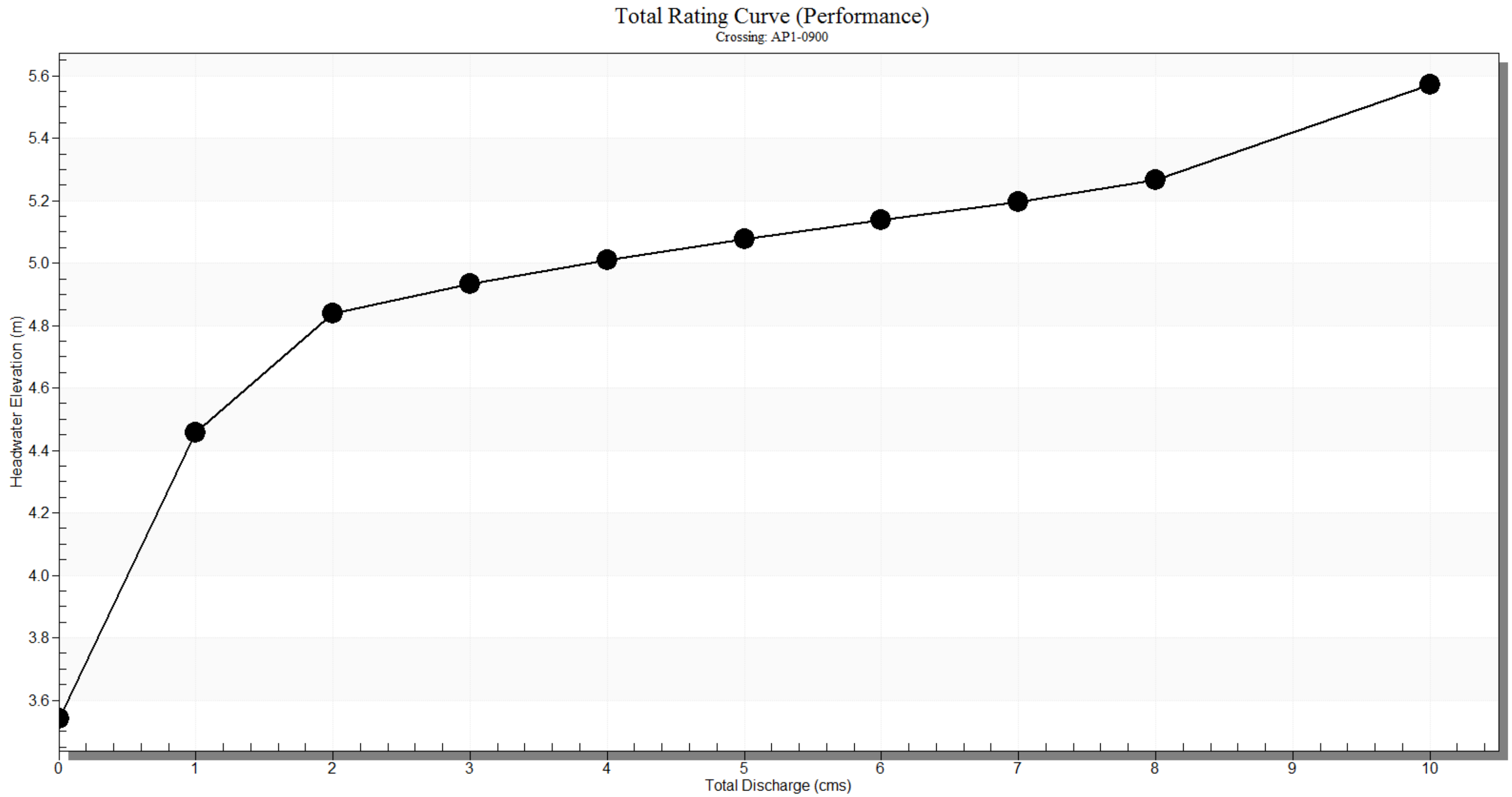


Figure 2. Rating curve for AP1 — Single barrel 900 mm culvert crossing

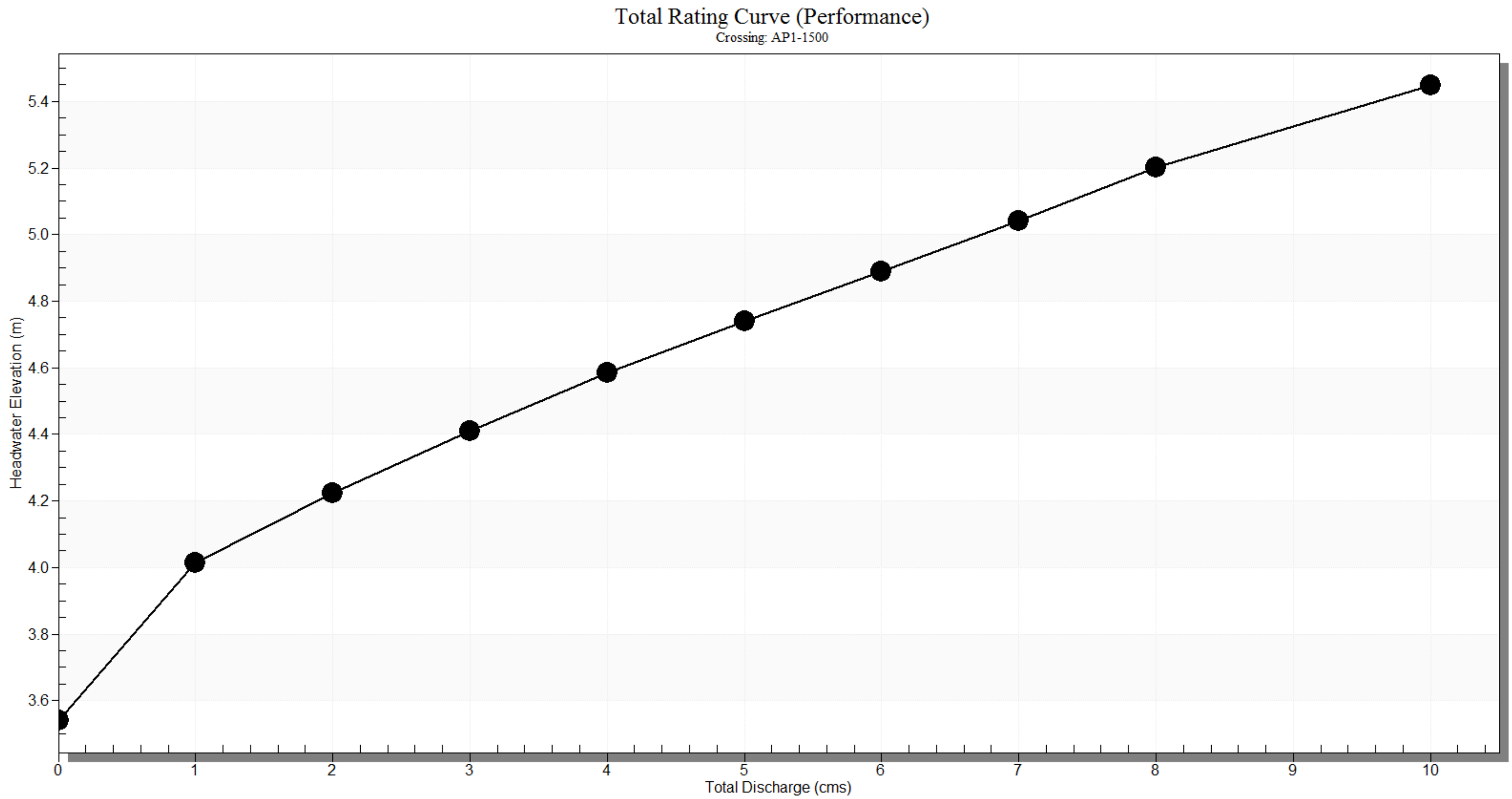


Figure 3. Rating curve for AP1 — Two-barrel 1500 mm culvert crossing

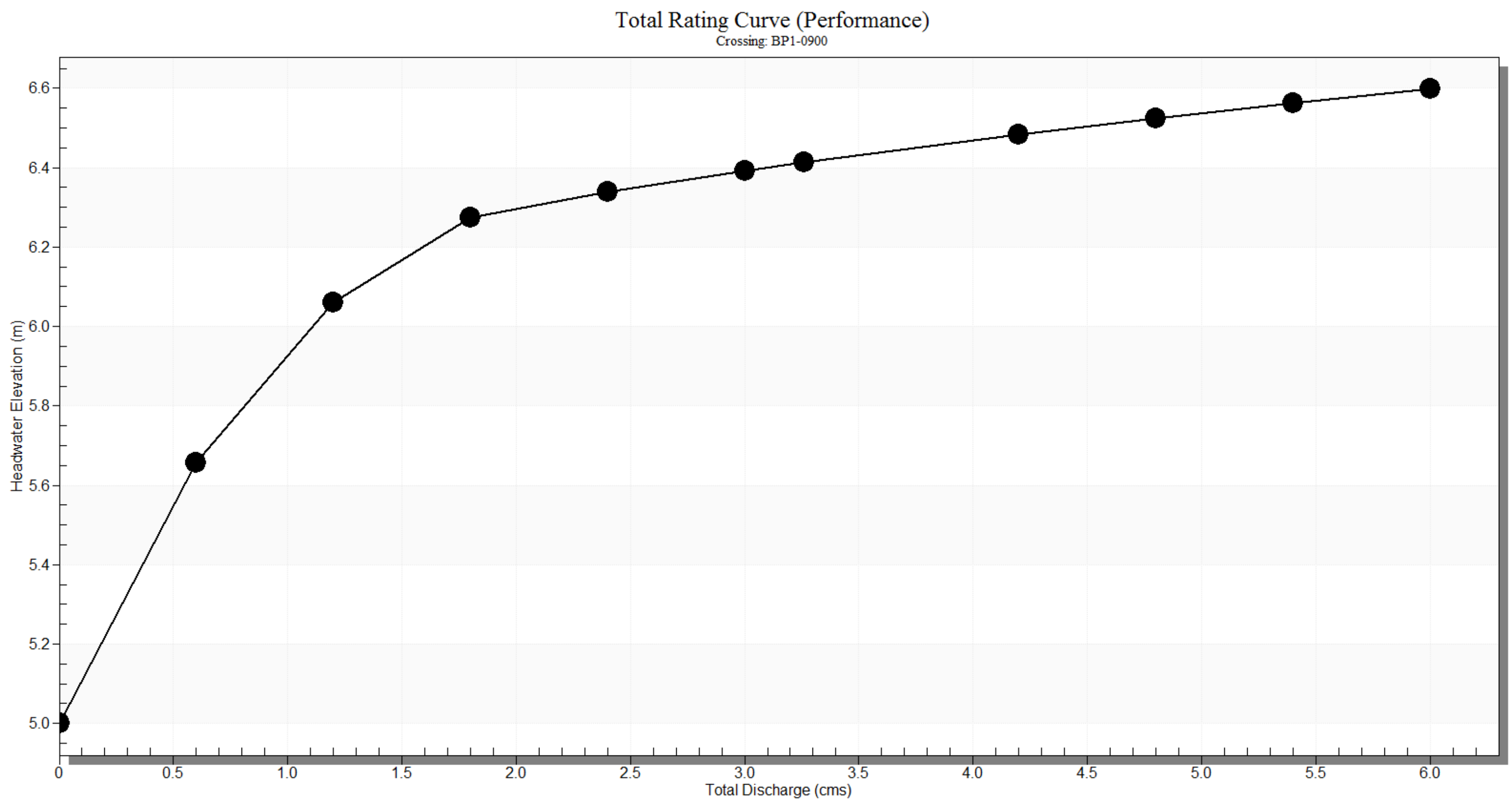


Figure 4. Rating curve for BP1 — Single barrel 900 mm culvert crossing

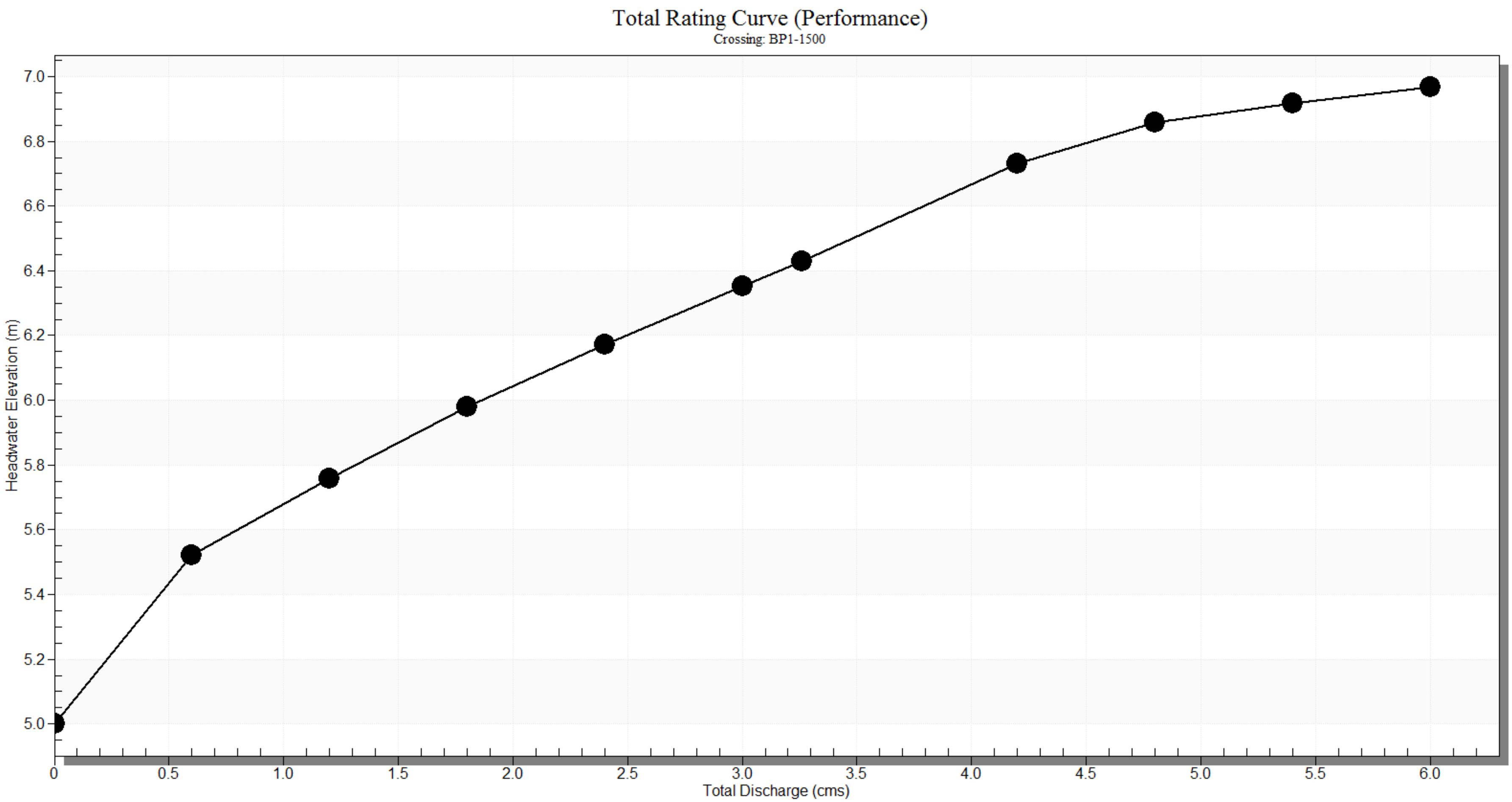


Figure 5. Rating curve for BP1 — Single barrel 1500 mm culvert crossing