Amina Reservoir Project Report

Prepared for Instituto Nacional de Recursos Hidráulicos, Dominican Republic

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Executive Summary

The following report summarizes our investigation of the proposed Amina Reservoir in the Dominican Republic. Our study included a hydrologic feasibility analysis as well as an assessment of benefits related to water availability, drought storage, flood control, and hydropower.

Our analysis revealed several concerns that will be discussed in detail in the report. First, the proposed demands are unknown. Demands for irrigation, drinking water, hydropower, and evaporation must be known for a conclusive water balance. Second, the proposed lower outlet pipe has been specified with a 2.0-m diameter. Based on our calculations, this size is insufficient to pass the required flow of 100 m3/s and should be resized to 2.5 m. Third, there was not enough available information about hydropower facilities, including intake elevation, head difference, and location. Researching or redefining these details is necessary if sustainable hydropower is to be developed at this site.

Despite the lack of some information, from a hydrologic standpoint, Amina Reservoir is feasible. Historically, there is enough water to provide for the proposed regulated flow, with reduced flows during the filling phase and dry periods. The construction of a dam would further allow for storage of water during droughts and would improve flow reliability to downstream regions. The reservoir would also provide substantial flood protection and offer potential for hydropower development.

Geological and structural factors must also be considered, along with non-technical issues, but in our opinion the project is both feasible and beneficial from a hydrologic perspective.

Introduction

The Dominican Republic's national water agency, Instituto Nacional de Recursos Hidráulicos (INDRHI), has proposed to construct Amina Reservoir near the city of San José de las Matas, with the dam being located about 1.5 km downstream of the confluence of the Amina and Inoa rivers. The proposed project is intended to provide benefits for drought storage, regulated flow, flood control, and hydropower for the region.

Previous technical reports related to Amina Reservoir were done by Hanson & Rodriguez in 1978 as part of a Río Yaque del Norte Master Plan and by SOGREAH-SERCITEC in 2003 as part of another flood-control initiative. Most of the details about the dam were outlined by Hanson & Rodriguez.

However, new data and better technology are available to update some aspects of the project. Our purpose was to evaluate the proposed reservoir from a hydrologic perspective, analyzing water availability based on historical data and evaluating the potential for the aforementioned benefits. In the following pages we will discuss in more detail:

- Watershed characteristics
- Reservoir and dam characteristics
- Water availability
- Drought storage
- Model calibration
- Flood control
- Hydropower

Our conclusions on the project will be discussed at the end of the report.

Watershed Characteristics

Table 1 summarizes the watershed characteristics used in this study. These will be discussed further on the following pages.

Table 1: Watershed Characteristics

Watershed area	339 km^2
Computed composite CN	73
Adjusted composite CN	86
Probable maximum precipitation (24-hour)	600 mm
Average annual precipitation	1200 mm
Predominant hydrologic soil group	C

Area

The tributary area of the proposed reservoir includes both the Amina and Inoa river basins. Using the Watershed Modeling System (WMS) and a 30-m digital elevation model (DEM), we computed flow directions with TOPAZ and delineated the watershed (Figure 1). WMS computed an area of 349 km^2; INDRHI indicates an area of 339 km^2. The difference may be attributed to exact placement of the outlet location or low resolution in the DEM.

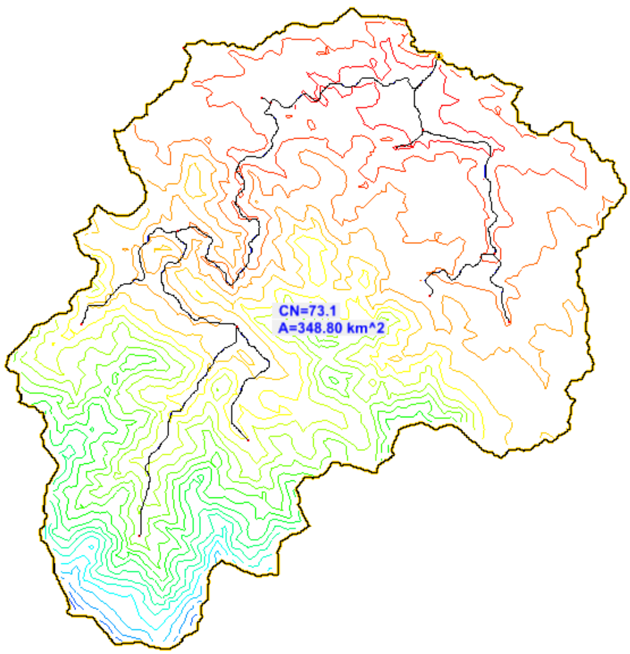

Figure 1: Delineated watershed

Curve Number and Terrain

Based on shapefiles of soil type and land use, we used WMS to compute a composite curve number (CN) of 73. However, as will be discussed later, our HEC-HMS models did not produce enough runoff volume with this CN. Since most precipitation occurs during the rainy season in the Dominican Republic, we suspected that a "wet condition" (Condition 3) might typically apply for the CN. Based on tables of CN adjustments for antecedent conditions, we ultimately chose a CN of 86.

Typical terrain and vegetation in the basin is illustrated by Figure 2.

Figure 2: Typical basin terrain and vegetation

Precipitation Data

Next, we gathered precipitation data for the watershed. At first, three nearby gages were used to determine average rainfall using Thiessen polygons. However, we were unable to calibrate the precipitation time series to the available flow time series. This could be due to the drastic size of the watershed and the geographic scarcity of precipitation data.

Rather than using precipitation gage data and calibrating that to flow data, we decided to use precipitation-frequency maps provided by INDRHI. The contours in the precipitation frequency maps for 2.33-, 5-, 10-, 25-, 50-, and 100-year 24-hour storms were digitized into WMS. Then, using WMS, the weighted area precipitation was calculated over the watershed for each storm. A digitized precipitation-frequency map in WMS can be seen in Figure 3.

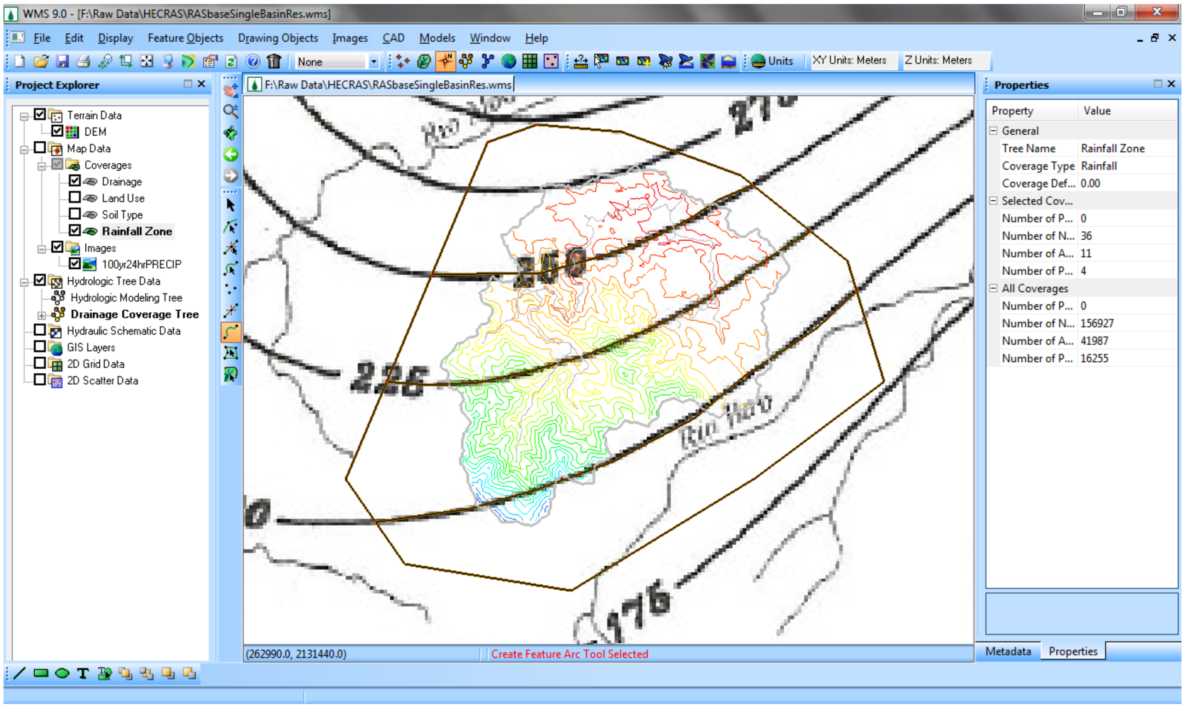

Figure 3. Digitized precipitation frequency map for the Amina watershed

According to INDRHI, the average precipitation in the watershed is 1200 mm/yr.

Area Reduction Factor

In order to compensate for the watershed size, a depth-area-reduction factor was use to adjust the precipitation values according to the size of the watershed. Using the method used by the U.S. Bureau of Reclamation, it was determined that a watershed area of 350 square kilometers should use a depth-area-reduction factor of 0.91. This factor was multiplied by the area weighted 24-hour precipitation values that were determined in WMS to get the average rainfall over the entire watershed.

Furthermore, to determine the probable maximum precipitation (PMP) for the watershed, we followed INDRHI protocol and used the 24-hour precipitation values from Hurricane Jeanne in 2004 as recorded in Puerto Rico. This produced a 24-hour PMP of 600 mm.

Once the precipitation data were ready, we developed a HEC-HMS model. The SCS Type II 24-hour storm was used because of INDRHI protocol. For the losses in the watershed, the SCS curve number method was used. For hydrograph attenuation, the Clark Transform method was used.

Reservoir and Dam Characteristics

According to Hanson & Rodriguez, Amina Reservoir will have a useful storage capacity of 337,000,000 m^3. This is consistent with our calculations based on the DEM and watershed characteristics. The maximum surface area of the reservoir (at full capacity) will be 14.0 km^2.

The proposed dam is to be situated in a canyon, about 1.5 km downstream from the confluence of the Amina and Inoa rivers, on the north side of the watershed. According to Hanson & Rodriguez it is to be a concrete gravity dam with a height of 86.0 m, a crest length of 230 m, and a spillway at an elevation of 400.0 m. The spillway capacity is to be 850 m^3/s.

The lower outlet was specified with a diameter of 2.0 m. However, as will be discussed later, our analysis revealed that this is insufficient to release a flow of 100 m^3/s as intended for flood control. With the spillway and lower outlet combined, the reservoir can regulate up to 950 m^3/s.

Water Availability

In their flood-control plans, Hanson & Rodriguez recommended a regulated outflow of 5.5 m^3/s from the proposed reservoir, while SOGREAH-SERCITEC more recently recommended 7.0 m^3/s. To evaluate the potential to meet this demand we examined local streamflow records.

Fortunately, there is a stream gage located very near the proposed dam site which is maintained by INDRHI. With 29 years of daily flow data (1967–1995) we developed a flow-duration curve (FDC) as shown in Figure 4. Table 2 summarizes pertinent flow statistics.

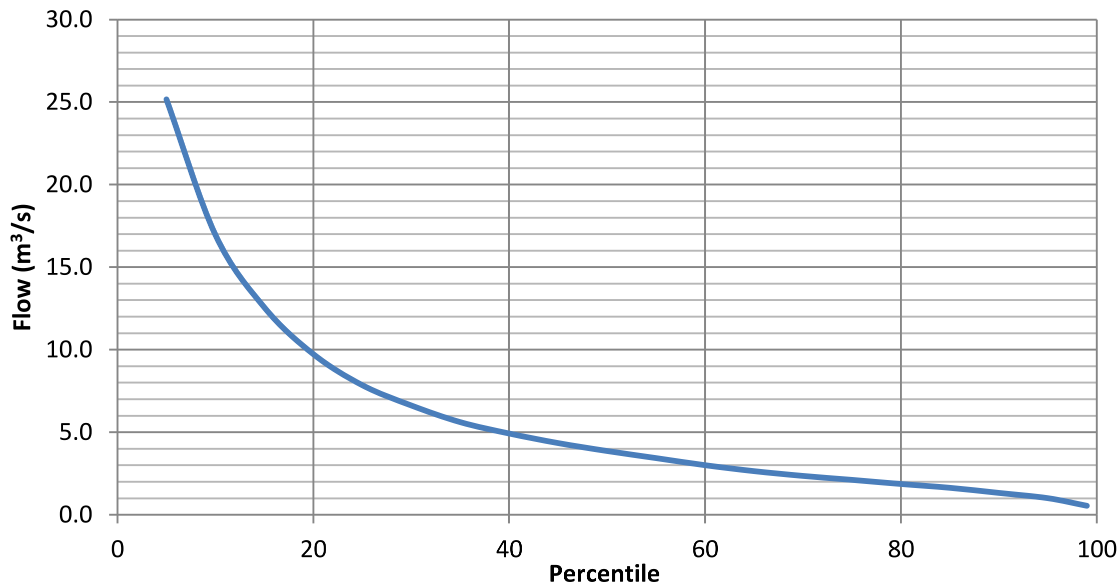

Figure 4: Flow-duration curve

Table 2: Daily Flow Statistics

Mean	7.7 m^3/s
95th percentile	1.0 m^3/s
50th percentile	3.9 m^3/s
Minimum	0.0 m^3/s
Maximum	315.1 m^3/s

From Table 2 we note that even though the mean flow is 7.7 m^3/s, the 50th percentile flow is only 3.9 m^3/s. This is because extreme precipitation events (hurricanes) inflate the average.

According to Hanson & Rodriguez, the reservoir was intended to provide irrigation supply for 2100 ha. According to INDRHI, a maximum flow of 2.4 m^3/s would be needed for this use. This corresponds to the 69th percentile. This demand is likely to have changed since 1978, but INDRHI has no updated irrigation demand for this project.

Figure 5 shows a graph of annual average daily streamflow for the years of record.

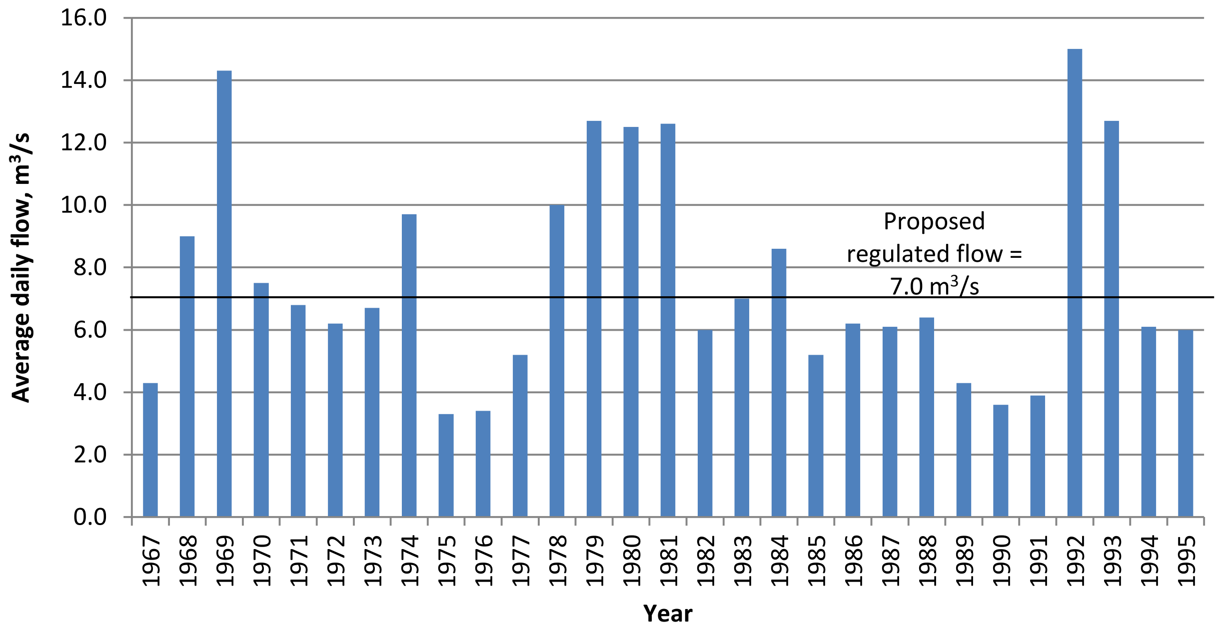

Figure 5: Annual average daily flows

From Figure 5 we see that in the available record, 18 of the 29 years have average daily flows at or below 7.0 m^3/s. As will be discussed in the next section, a reservoir could provide some storage during low-flow periods, but the regulated flow may need to be reconsidered.

Drought storage

With the streamflow data we developed a cumulative mass curve (Figure 6) to analyze long-term storage fluctuations.

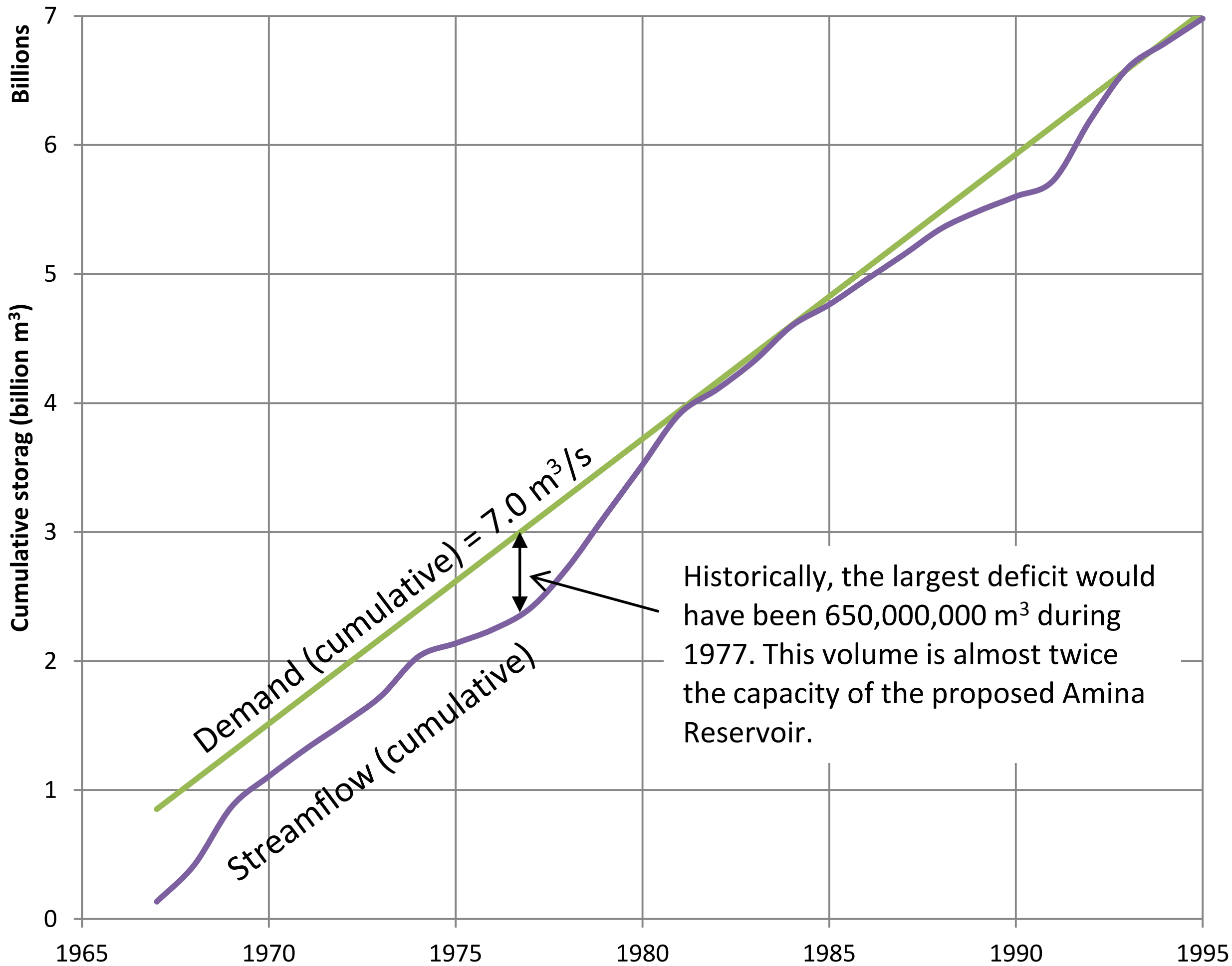

Figure 6: Cumulative mass curve

Considering that the average daily flow is only 7.7 m^3/s for the 29 years of record and that a constant flow of 7.0 m^3/s should be regulated, this leaves only 0.7 m^3/s on average as excess flow for storage. At this rate it would take 15 years to fill the reservoir from empty, and longer (or not at all) if the reservoir happens to be constructed during a particularly dry period. Table 3 outlines several filling scenarios. We suggest that the regulated outflow be reduced during the filling phase of the reservoir in order to more quickly provide useful storage.

Table 3: Reservoir fill time

Inflow (m^3/s)	Percentile flow	Fill time (years)
0.7	98	15.3
1.0	95	10.7
3.9	50	2.7
7.0	28	1.5

Assuming that the reservoir is full (337,000,000 m^3) at the beginning of a drought and that no additional inflows contribute to the reservoir, a regulated outflow of 7.0 m^3/s could be maintained for 557 days (1.5 years). As seen earlier in Figure 5, droughts typically last more than one year. For example, the years 1985–1991 were all below 7.0 m^3/s. With this under consideration, we suggest that the regulated outflow be reduced during dry periods in order to provide longer supply.

If we assume a constant demand (regulated flow) of 7.0 m^3/s, we can compute from the mass curve the storage capacity needed to outlast a drought. For the years of record, the largest deficit would have been in 1977, with a deficit volume of 650,000,000 m^3 . This is almost twice the storage capacity of the proposed reservoir. Based on our analysis, to provide enough storage for this historical drought the dam would have needed to be 22.5 m higher and the water surface would cover an area of 19.5 km^2. However, a reservoir this size at this location is infeasible, if for no other reason than the length of time it would take to fill.

Note that from Figure 4, a 7.0-m^3/s flow corresponds to the 28th percentile. If this were to be the regulated flow, the reservoir would have to provide the remaining flow 72 percent of the time.

HEC-HMS Model Calibration

Near the proposed dam site on the Amina River there is a flow gage that has recorded data for 29 years. For calibration purposes the measured daily flow was converted to a daily discharge volume. Using this flow data we were able to determine the 2.33-, 5-, 10-, and 25-year streamflow return periods. We then ran HEC-HMS (Hydrologic Modeling System) with the 2.33-, 5-, 10-, and 25-year 24-hour precipitation values for the basin and compared the HEC-HMS 24-hour discharge volumes to the 24-hour measured flow volumes from the recorded data for the corresponding return periods. The error between the measured 24-hour discharge volumes and the uncalibrated HEC-HMS 24-hour discharge volumes can be seen in Table 4.

Table 4: Measured 24-hour discharge vs. uncalibrated HMS 24-hour discharge

Return Period (years)	Measured 24-hour Discharge Volume (m^3)	Uncalibrated HEC-HMS 24-hour Discharge Volume (m^3)	Error
2.33	347,000,000	159,000,000	51%
5	541,000,000	253,000,000	51%
10	717,000,000	371,000,000	47%
25	934,000,000	480,000,000	47%

The uncalibrated HEC-HMS 24-hour discharge volumes results were consistently about 50% of the measured volumes, indicating that not enough runoff was produced in our model.

We then calibrated the HMS model to the 24-hour measured discharge volumes for the 2.33-, 5-, 10-, and 25-year return periods. The two parameters that were calibrated were the curve number and the initial abstraction. As discussed earlier, the CN was calibrated to be 86 based on wet antecedent conditions. However, INDRHI pointed out that inflating the CN to calibrate the data is unusual for the Dominican Republic; usually they need to lower the CN to match measured values. Ultimately we decided to use the high CN because of the importance of matching measured data. The initial abstraction was calibrated to be 0.1S rather than 0.2S since a saturated watershed will not have as much initial abstraction.

A comparison of the measured and calibrated 24-hour discharge volumes is presented in Table 5.

Table 5. Measured 24-hour discharge vs. calibrated HMS 24-hour discharge

Return Period (years)	Measured 24-hour Discharge Volume (m^3)	Calibrated HEC-HMS 24-hour Discharge Volume (m^3)	Error
2.33	347,000,000	353,000,000	–8%
5	541,000,000	471,000,000	9%
10	717,000,000	618,000,000	11%
25	934,000,000	736,000,000	19%
50	N/A	1,382,000,000	N/A
100	N/A	1,822,000,000	N/A
PMP	N/A	1,179,000,000	N/A

From Table 5 it can be observed that that the error is less than 20% but it seems to get larger as the return period increases. This suggests that the HMS model results may not have a linear relationship with the measured flow. However, by looking at the calibrated HMS hydrographs for all of the return periods in Figure 7 it can be seen that although the 2.33-, 5-, 10-, and 25-year HMS results seem to increase linearly, there is a big gap between the 25- and 50-year return periods. The difference would likely decrease the error if measured data were available for the 50- and 100-year events.

With a calibrated model we were ready to proceed with further analysis.

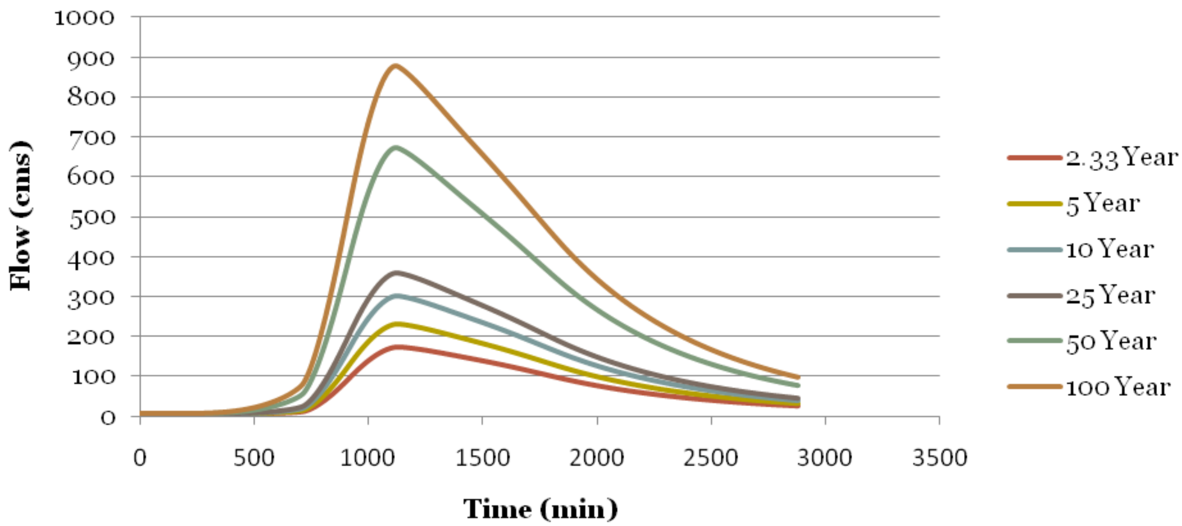

Figure 7: Calibrated HMS results for a 24-hour storm at the proposed dam site

Flood Control

In our analysis we developed models to compare pre- and post-reservoir scenarios, examining the efficacy of the reservoir for flood attenuation and flow routing. Reservoir routing was analyzed with HEC-HMS, while downstream flood scenarios were evaluated with HEC-RAS and GSSHA models.

Reservoir Routing

In order to determine the flood control capabilities of the dam, an elevation-storage-discharge curve was developed in WMS for the proposed reservoir. As mentioned earlier, there is to be a 2.0-m diameter outlet pipe at the base of the dam. One of the problems we encountered with the proposed dam's documentation is that the 2.0-m outlet needs to be able to handle 100 m^3/s of flow in order to reduce the water in the reservoir in preparation of large storms. With the outlet being only 2.0 m in diameter, the maximum capacity of the pipe when the head is at the base weir elevation is only about 65 m^3/s. In order to handle the 100 m^3/s capacity when the head is at the base weir elevation, the pipe would need to have a diameter of 2.5 m. When developing the elevation-storage-discharge curve in WMS an outlet pipe size of 2.5 m was used instead of 2.0 m so that an emergency release of 100 m^3/s could be regulated. Furthermore, the curve was adjusted so that the dead storage and outlet and weir elevations matched the provided documentation. The elevation–storage–discharge curve can be seen in Figure 8.

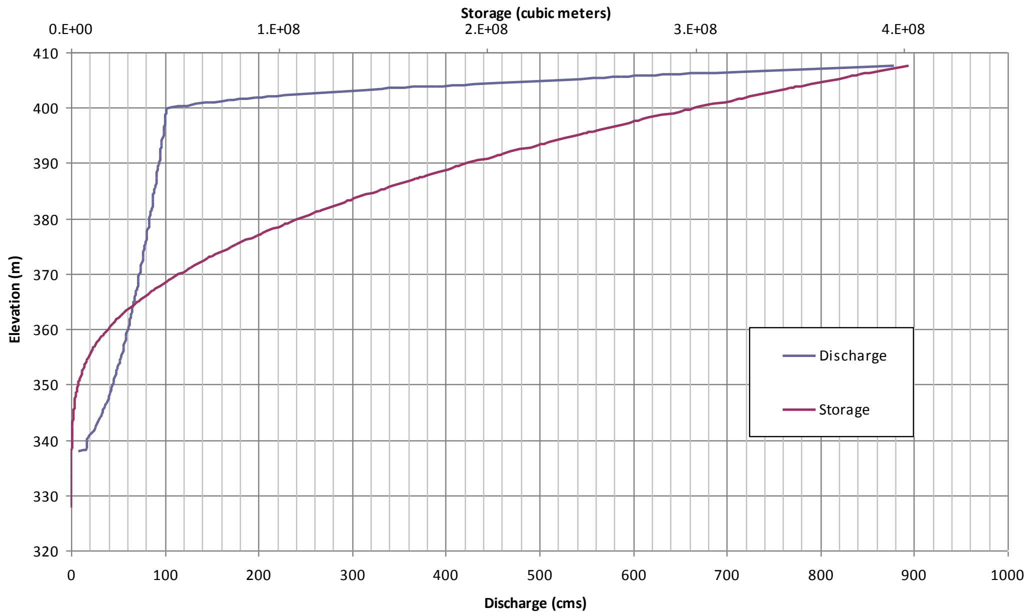

Figure 8. Elevation-Storage-Discharge curve for proposed reservoir

By examining the elevation-storage-discharge curve it can be seen that just before the water level reaches the weir at 400 m the discharge is 100 m^3/s (through the lower outlet). When the water reaches the top of the weir the discharge jumps to 950 m^3/s. Furthermore, the storage at the base of the weir is about 310,000,000 m^3 which is near the 337,000,000 m^3 of usable storage that was mentioned in the proposed dam's documentation.

The calibrated hydrographs from Figure 7 were then routed through the dam in HEC-HMS using the elevation-storage-capacity curve from Figure 8 with the dam at a 75% capacity. The routed outflow hydrographs can be seen in Figure 9.

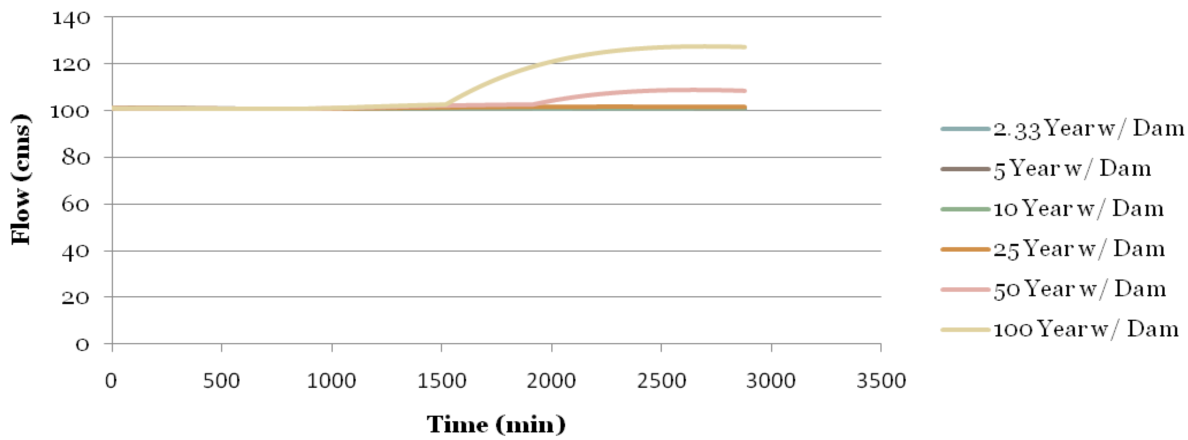

Figure 9: Calibrated HMS results for a 24-hour storm routed through the reservoir at 75% capacity

First, it must be pointed out that the flow starts at 100 m^3/s because it is assumed that before a large storm hits, the dam operators will start releasing 100 m^3/s in preparation of the storm. Doing this when the dam is already at a capacity 75% seems to be overkill because there is already enough available storage to dramatically attenuate or completely absorb large flooding. It may only be worthwhile to release the 100 m^3/s before a large storm if the dam is already near capacity or if the storm is expected to have a 50-year or greater return period. Figure 9 also shows that at 75% capacity there is enough storage for the reservoir to completely absorb 2.33-, 5-, 10-, and 25-year storms. Table 6 shows the recommend amount of time needed to drain 100 m^3/s before a storm if the reservoir is at full capacity in order to eliminate downstream flooding. It also shows the recommend pre-storm water-surface elevation needed to attenuate the flood enough to eliminate downstream flooding. It should be noted that this table assumes that during the large storms, 100 m^3/s is being released from the reservoir.

Table 6: Large-Storm Preparation Table

Return period (years)	Time needed to drain at 100 m^3/s prior to storm to prevent flooding (hours)	Pre-storm reservoir volume needed to prevent flooding (m^3)	Pre-storm recommended elevation (m)
2.33	0.0	285,000,000	398.6
5	1.0	282,000,000	398.2
10	13	277,000,000	397.9
25	22	274,000,000	397.7
50	72	256,000,000	395.8
100	107	243,000,000	395.2
PMP	357	153,000,000	385.0

The table shows that if the reservoir is full, about 107 hours (4.5 days) are needed to drain at 100 m^3/s to increase the amount of available storage needed to eliminate downstream flooding during a 100-year storm. Since sufficient warning may not be possible, we recommend that the water level in the reservoir be kept 2 to 4 m below the base weir level in the wet season so that less time is needed to prepare the reservoir for large storms.

Figure 10 shows the attenuation of the PMF if the reservoir is at 75% capacity.

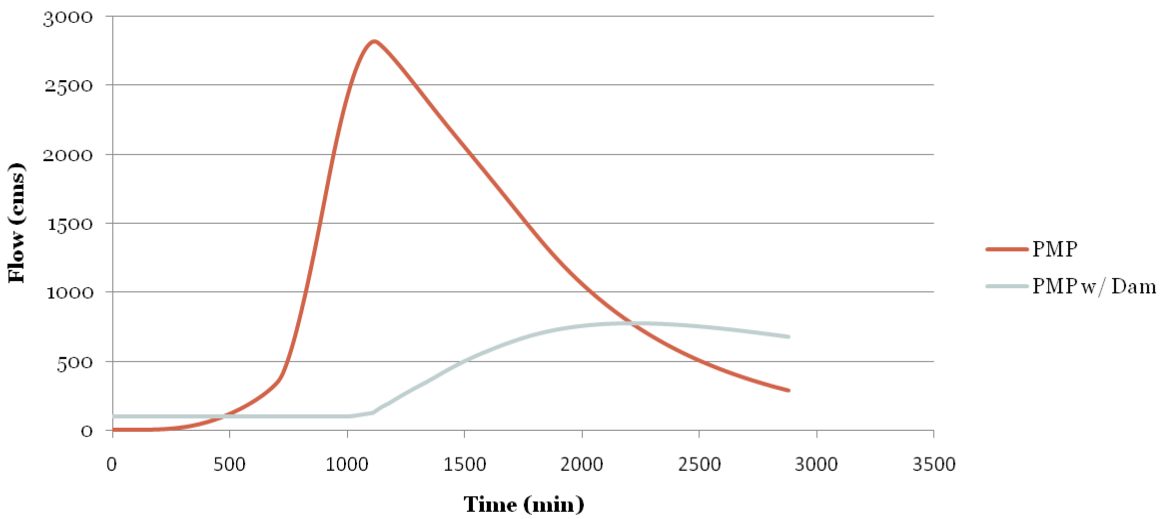

Figure 10. PMF hydrograph with reservoir at 75% capacity

If the reservoir is less than 75% full, there is enough capacity to absorb much of the PMF and attenuate the hydrograph. In this manner, reservoir routing would reduce the peak PMF flow from 2800 m^3/s to 750 m^3/s. However, if the reservoir is above a capacity of 75% then there would likely be dam overtopping in the case of a PMF. If dam operators would prefer that the dam have enough capacity to handle a PMF, we recommend that the reservoir should never be above 75% capacity before a storm.

Flood Modeling With HEC-RAS

To model the flooding protection that the dam would offer to the Amina and other downstream towns, HEC-RAS (River Analysis System) was used along with WMS. It should be noted that cross-section survey data was not available so elevation data was extrapolated from a 30-m DEM and then estimated channel geometry was manually inserted into the cross sections. This means that the results are a very rough estimate of the potential flooding. Figure 11 shows the cross sections that were used in the 1D model.

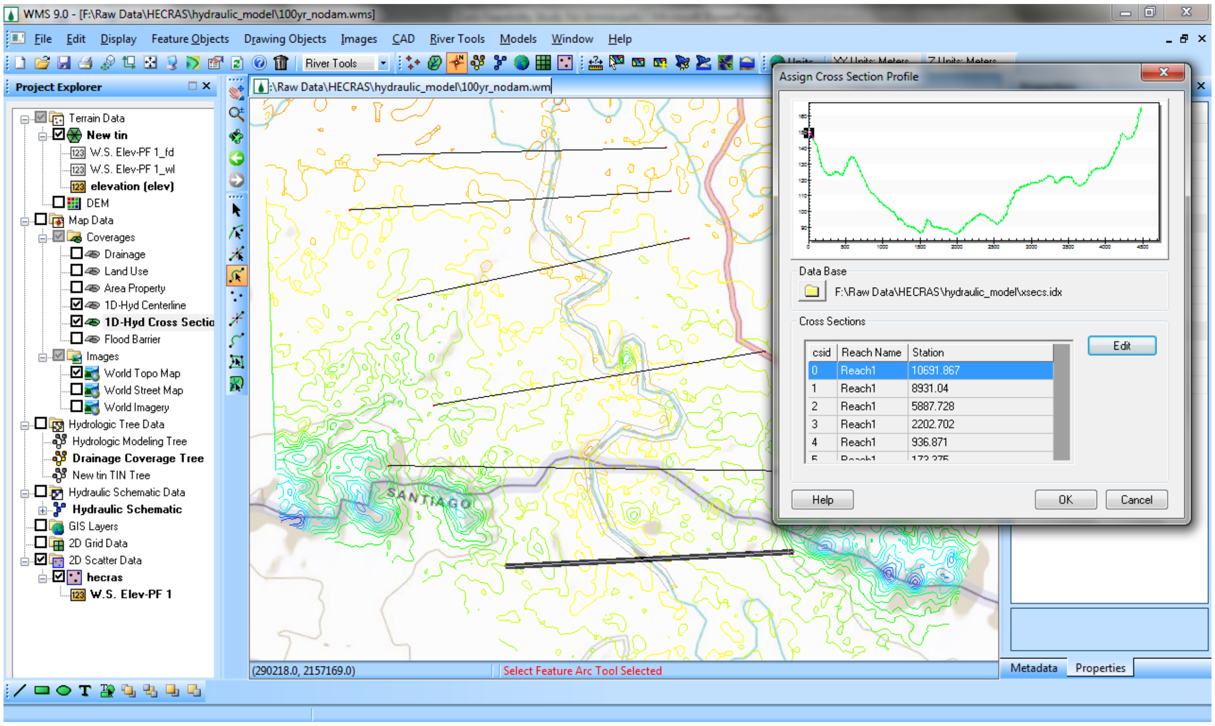

Figure 11. Amina cross sections that were extrapolated from DEM in WMS and then input into HEC-RAS

The HEC-RAS model was set up to be steady-state for two different scenarios. First, a 100-year 24-hour peak flow without the dam was used to show the flooding that would occur in the town of Amina if no dam was built. Next, a 100-year 24-hour peak flow with the dam and reservoir at a 75% capacity was used to show the flooding that would occur in the town of Amina if the dam was built. The results can be seen in Figures 12 and 13.

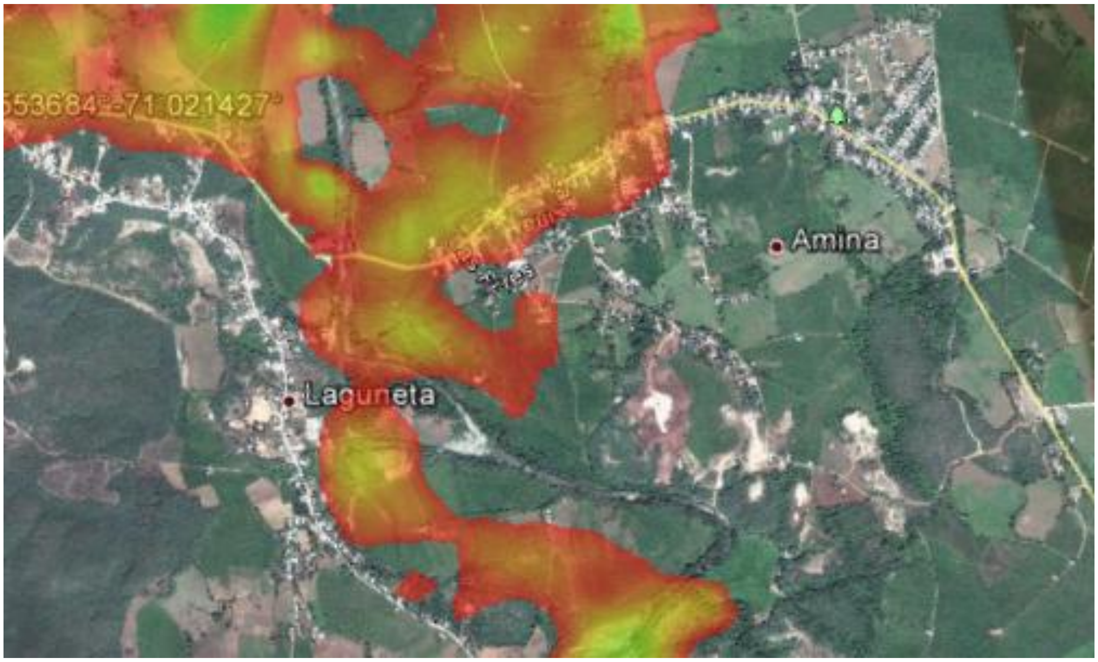

Figure 12. Flooding in the town of Amina during a 100-year 24-hour storm without the proposed dam

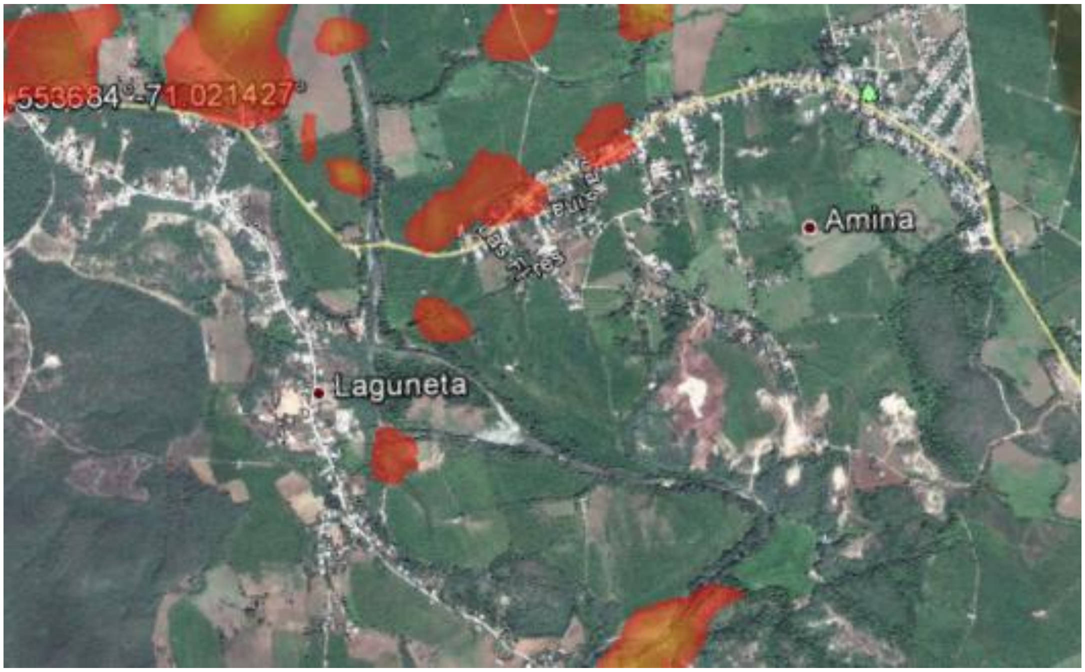

Figure 13. Flooding in the town of Amina during a 100-year 24-hour storm with the proposed dam and reservoir at 75% capacity

From Figures 12 and 13 we see that the proposed reservoir and dam would significantly reduce flooding in the case of a 100-year event. While Amina and Laguneta are the towns that would be most affected by the flood control capabilities of the dam, many other small farm communities would also benefit. It should be noted that the flood-control capabilities depend both on the willingness of dam operators to release water before a large storm and on the amount of available storage in the reservoir. Depending on these factors, the flooding in Amina could be much greater or completely nonexistent with the same 100-year 24-hour storm.

GSSHA Dam Break Flood Wave Model

In addition to the modeling of large storm events, we also modeled a dam break scenario using the Gridded Surface-Subsurface Hydrologic Analysis (GSSHA) model. The GSSHA model uses physics to perform 2D hydrologic and hydraulic modeling. GSSHA operates on a grid, where runoff and other computations are performed on each cell. An area downstream of the dam was modeled using a 200-m cell size. The gridded model can be seen in Figure 14.

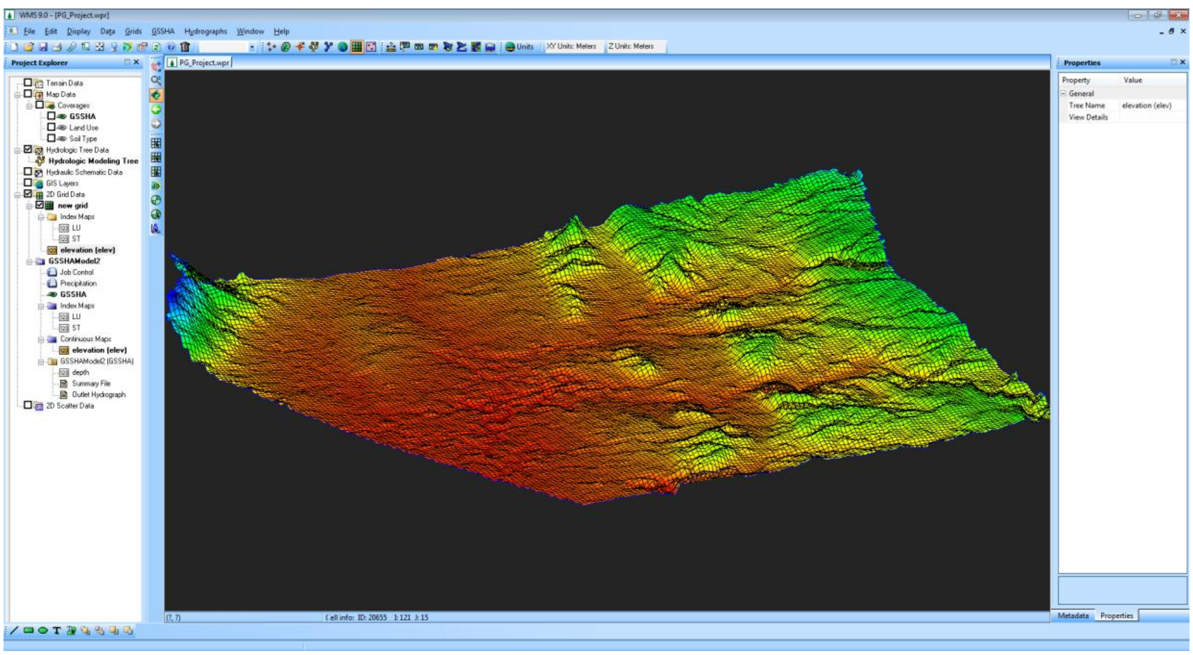

Figure 14. Gridded GSSHA model downstream of the proposed Amina dam

Roughness and soil types were mapped to the model using provided shapefiles. The dam break was modeled by creating an arc boundary condition at the location of the existing dam. A plot of time versus water-surface elevation was input in the boundary condition using a generic convex decreasing function type. The plot was calibrated to the reservoir storage capacity by increasing the length of the dam break. The length of the dam failure ended up being about 4 hours long.

The results were then exported as an animation in Google Earth. Screenshots of the maximum flooding in the downstream communities can be seen in Figures 15 and 16.

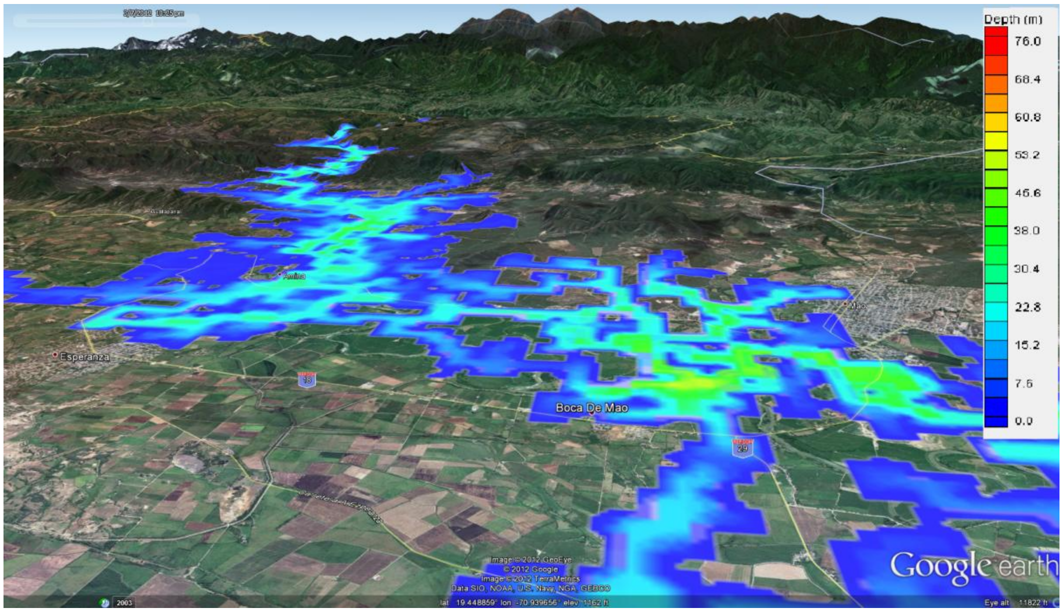

Figure 15. GSSHA dam break results for communities downstream of dam

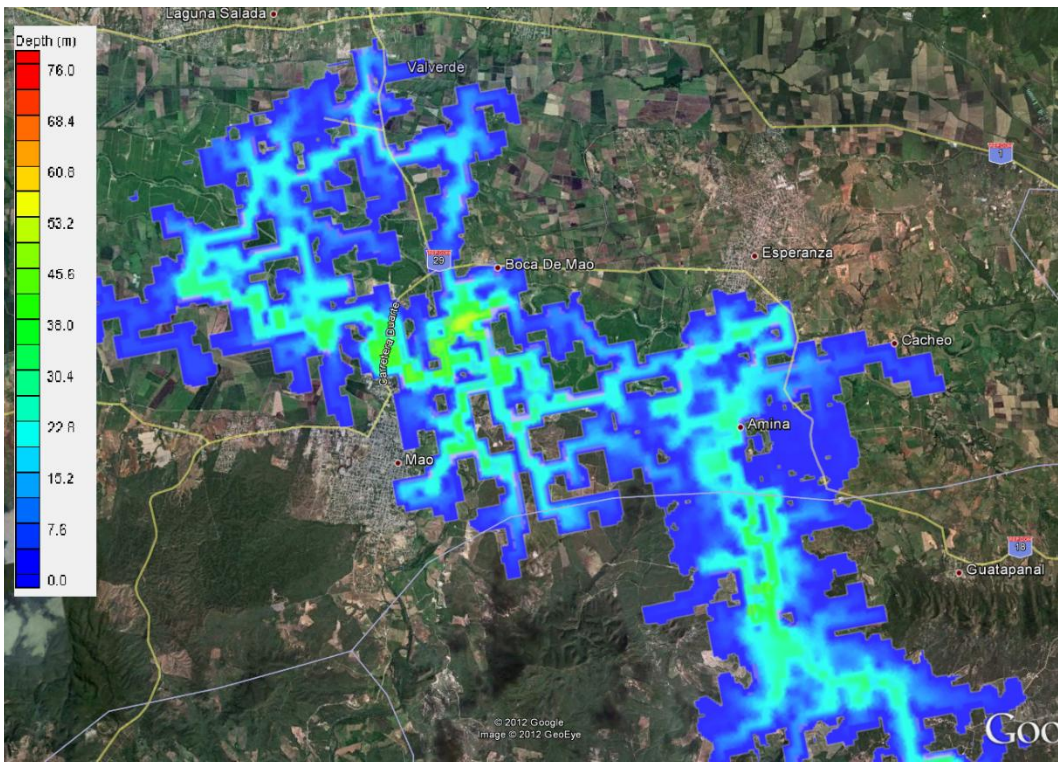

Figure 16. GSSHA dam break results for communities downstream of dam

According to our model, the towns that would be completely submerged by the dam break flooding are Amina and Laguneta. The towns and cities that would have major flooding but were not submerged were Boca de Mao, Cacheo, and Mao. Minor flooding would occur in Esperanza.

Hydropower

According INDRHI, the main purpose of Amina Reservoir would be to provide hydropower. However, due to a lack of information on the intake elevation, plant location, and head difference, we were unable to provide any meaningful results on this topic.

Evaporation

Our feasibility study would be incomplete without some discussion of evaporation. According to INDRHI, pan evaporation in the area totals 1700 mm/year. When applied to a reservoir of the size discussed (maximum surface area of 14 km^2), this equates to a loss of 24,000,000 m^3 annually assuming a pan coefficient of 1.0, or about 7 percent of the usable capacity. Of course, reservoir evaporation is always less than pan evaporation, but nonetheless represents a significant loss in the water balance.

Sedimentation

We understand that past experience in the Dominican Republic, especially the case of Aguacate, suggests that reservoir sedimentation is a major concern. A full-scale proposal for Amina Reservoir must include considerations for sedimentation prevention, control, and maintenance, but is beyond the scope of our work.

Conclusions

From a hydrologic standpoint, Amina Reservoir is feasible. Historically, there is enough water to provide for the proposed regulated flow, with reduced flows during the filling phase and dry periods. The construction of a dam would further allow for storage of water during droughts and would improve flow reliability to downstream regions. The reservoir would also provide substantial flood protection and offer potential for hydropower development. Geological and structural factors must also be considered, along with non-technical issues such as the displacement of local residents, but in our opinion the project is both feasible and beneficial from a hydrologic perspective.