APPENDIX G5. INTERIOR RESIDUAL FLOODING ANALYSIS

Huntington, VA (Cameron Run)
Documentation of Interior Residual Flooding Analysis

Residual Flooding

An interior residual flooding analysis was necessary both to size the drainage structures through the proposed levee and to document the residual flooding in the interior after the proposed levee has been constructed. A pumping station analysis was necessary to determine the pumping capacity necessary to reduce the residual flooding elevation to at or below the zero damage elevation. All of the hydrology (HEC-1, Flood Hydrograph Package), interior drainage (INTDRA), pumping station sizing (INTDRA), and residual flooding (from the results of the INTDRA analyses) were performed at a concept level design effort. The county and the COE decided to evaluate the 10-, 25- and 100-year rainfall runoff events for this phase of the study.

Hydrologic Analyses

As a starting point for this concept level analysis for the rainfall/runoff (hydrology) for Huntington, VA, the HEC-HMS computer model which was developed for the Cameron Run Watershed Study was reviewed. The drainage area behind the proposed levee in Huntington, VA is part of the Cameron Run watershed. The two subareas in the Cameron Run HEC-HMS model that drain the interior drainage area behind the proposed levee in Huntington, VA are CR1.30A and CR.84. A review of this HEC-HMS model revealed that, since this was part of a basin-wide model, the analysis was not done in a detailed enough fashion to directly use any of the parameters from this model. Therefore, it was necessary to determine the Natural Resource Conservation Service (NRCS) curve numbers (CNs), the drainage areas (DAs), and the NRCS lag times (LTs) for both of these subareas. For the purposes of this study Subarea CR.84 was renamed Subarea A, and Subarea CR1.30A was renamed Subarea B. The DAs were redrawn for Subareas A and B based on an examination of the 1-foot and 5-foot contour interval topography in this area. The boundary of Subarea B changed very little from the boundary of CR1.30A in the HEC-HMS study, but the boundary for Subarea A changed significantly from the boundary of CR.84 from the HEC-HMS study. The reasons that the boundary of Subarea A changed significantly was both due to the inclusion of the proposed levee and because the field investigation revealed that construction related to new development will add a significant amount of drainage area behind the proposed levee. The boundaries of Subareas A and B are shown on Plate 1. The DAs for Subareas A and B were then determined for use in this study. The DAs for Subareas A and B are shown in Table 1.

The aerial photography was examined to determine the appropriate land uses in Subareas A and B for use in the determination of the CNs. The land uses determined for Subarea A were open space (lawns, parks, etc.) in fair condition, commercial and business, 1/5 acre size lot (residential), and woods. The land uses determined for Subarea B were impervious areas (paved parking lots), commercial and business, 1/5 acre size lot size (residential), and woods. The area for each type of land use was approximated for

each subarea, and a weighted CN for each subarea was determined. The NRCS hydrologic soil groups are A, B, C, and D, with Group A resulting in the lowest amount of rainfall runoff, and Group D resulting in the highest amounts of rainfall runoff. All of the CNs selected from the tables in "Urban Hydrology for Small Watersheds" by NRCS assumed Group C. This assumption was made because there was no soil data readily available for this part of the Cameron Run basin. The Cameron Run basin wide study originally assumed Group B, but during the calibration of the HEC-HMS model this was changed to the assumption of Group C. Therefore the Huntington, VA study also assumed Group C, which resulted in the determination of higher CNs for the subareas, and therefore higher quantities of runoff (less rainfall is absorbed by the soil), than if Group B was assumed. After the analysis was completed, a figure was discovered that approximated the soil groups throughout the Cameron Run basin. When the boundary of the drainage area for Huntington, VA was compared to this figure, it was discovered that a small portion of the Huntington, VA drainage area was Group C, with the remainder of the drainage area approximately evenly divided between Groups B and D. Since a comparison of the CNs for Groups B, C, and D for the land uses discussed previously shows that the CN for Group C is always greater than or equal to the average of the CNs for Groups B and D, using Group C was somewhat conservative. Therefore, this figure supported the conclusion that the assumption of Group C for the Huntington, VA study was reasonable, though probably conservative. The weighted CNs for Subareas A and B are shown in Table 1.

The lag times were then computed for both subareas using the CNs determined above, an average of the longest distance the rainfall runoff would travel in the subarea, and the average slope of this longest distance the runoff would travel (utilizing both the 1-foot and 5-foot contour interval topography). For Subarea A the slope and length were determined for four different longest distances the runoff would travel, the four lag times were then computed, and the lag time used for Subarea A was the result of determining the average of these four lag times. For Subarea B the slope and length were determined for two different longest distances the runoff would travel, the two lag times were then computed, and the lag time used for Subarea B was the result of determining the average of these two lag times.

Table 1

Subareas	Drainage Areas (square miles)	Curve Numbers	Lag Times (hours)
A	0.107	84	0.267
B	0.153	82	0.431
AB	0.260	83	0.322

Due to this being a concept design level analysis, it was decided to use HEC-1 for this analysis instead of HEC-HMS, Hydrologic Modeling System. The rainfall distribution to use in the HEC-1 (hydrologic – rainfall/runoff) models for the 100-year, 25-year, and 10-year hypothetical rainfall events was obtained from the Point

Precipitation Frequency Estimates from National Oceanographic and Atmospheric Administration (NOAA) Atlas 14 for the rain gage at Washington Reagan National Airport (WRA), VA. Corps criteria require that the rainfall distribution used be for the 5, 15, 60, and 120-minute and 3, 6, 12, and 24-hour rainfalls from Atlas 14. The 24-hour rainfalls used for each event for both Subareas A and B are shown in Table 2 below.

Table 2

Rainfall Events	24-hour Rainfalls (inches)
100-year	8.24
25-year	5.98
10-year	4.78

Hydrologic computer models (HEC-1) were developed using the DAs, CNs, lag times, and rainfall distributions discussed above for both Subareas A and B for the 100-, 25-, and 10-year hypothetical rainfall events. Table 3 below displays the peak discharges of the rainfall/runoff determined for Subareas A and B for the 100-, 25-, and 10-year hypothetical rainfall events.

Table 3

Hypothetical Rainfall Events	Subarea A Peak Discharges (cfs)	Subarea B Peak Discharges (cfs)	Subarea AB Peak Discharges (cfs)
100-year	265	302	578
25-year	202	223	434
10-year	161	173	341

Interior Residual Flooding Analyses

For this concept level design effort, it was decided that the interior flooding analysis would be performed using the simplifying assumption that the gravity outlets are blocked by high flows in Cameron Run for the entire duration of the rainfall event. If the project proceeds, and it is desired that this levee project be designed in a manner that will permit the Corps to certify it for FEMA flood insurance purposes, it would be necessary to analyze a number of historic events, and include the effect of the historic river hydrographs (the river discharges observed during each event, which will affect the discharge from the storm drain pipes and culverts through the proposed levee to Cameron Run), and the hypothetical events, which are for the 2-, 5-, 10-, 25-, 50-, and 100-year rainfall distributions (and which assumes that the river flows are low enough to not affect the discharge from the storm drain pipes and culverts through the proposed levee to Cameron Run) . Historic and hypothetical peak pond elevation frequency curves are then drawn and combined using joint probability (the probabilities of both curves at a range of elevations are added together) to determine the elevation of the 1% chance exceedance frequency event. Though the assumption of blocked flow for the gravity outlets is usually conservative, it is considered a reasonable assumption for Huntington, VA due to

the fact that the storm drains are at such a low elevation that they are frequently affected by the discharge in Cameron Run.

The interior residual flooding analysis was performed using the runoff hydrograph routing program INTDRA. The runoff hydrograph from the HEC-1 output files for each hypothetical rainfall event for both Subareas A and B were included in the INTDRA models. Due to the assumption of Cameron Run always blocking the discharge from the storm drain pipes and culverts through the proposed levee to Cameron Run, this rainfall runoff hydrograph was not routed through these drainage pipes or culverts, but was instead ponded using the pond capacities determined from the topographic mapping. The peak pond elevation for each event for Subareas A and B, as determined by the INTDRA models, are shown in Table 4.

Table 4

Rainfall Events	Subarea A Peak Pond Elevations (feet - NGVD29)	Subarea B Peak Pond Elevations (feet - NGVD29)	Subarea AB Peak Pond Elevations (feet - NGVD29)
100-year	9.0	18.7	10.7
25-year	8.3	16.8	9.7
10-year	7.9	15.6	9.0

The INTDRA models for Subareas A and B also were developed assuming no overflow of the ponded rainfall runoff was permitted from one subarea to the other. Since the minimum overflow elevation from Subarea B to Subarea A is 6.0 feet, the peak pond elevations shown in Table 4 for the three events for Subareas A and B are affected by this assumption. It was decided, based on the results of the INTDRA analyses of Subareas A and B which does not allow overflow of the ponded rainfall runoff from one subarea to another, that a more appropriate way to model the interior residual flooding for Huntington, VA would be to combine Subareas A and B. This combination of Subareas A and B was called Subarea AB. The DA for Subarea AB was determined by adding together the DAs of Subareas A and B. The CN for Subarea AB was determined by adding together the sum of the CN and DA of both Subareas A and B, and then dividing this sum by the DA for Subarea AB. The LT for Subarea AB was determined by summing all of the LTs determined for each flow path (the longest distance the rainfall runoff would travel) for both Subareas A and B, and then dividing this sum by the total number of LTs that were computed for these flow paths. The DA, CN, and LT for Subarea AB are displayed in Table 1. The HEC-1 models for the three rainfall events were then developed for Subarea AB using the method described previously. The peak discharges for Subarea AB for the 100-, 25-, and 10-year events are shown in Table 3. INTDRA models for the three events for Subarea AB were developed using the rainfall/runoff from these HEC-1 models for Subarea AB. The peak pond elevations for these three events for Subarea AB are shown in Table 4 and plotted on Figure 1. The extent of the residual flooding caused by the ponding of the interior drainage to the elevations shown in Table 4 are displayed on Plate 2 for the 100-year rainfall event, Plate 3 for the 25-year rainfall event, and Plate 4 for the 10-year rainfall event. It was decided

that since the depth of overflow for these three events varies from 10.7 – 6.0 = 4.7 feet (depth of overflow for the 100-year event), 9.7 – 6.0 = 3.7 feet (depth of overflow for the 25-year event), and 9.0 – 6.0 = 3.0 feet (depth of overflow for the 10-year event), that it was appropriate to combine Subareas A and B into Subarea AB for this analysis.

Pumping Station Analyses

It should be noted that the pumping station analyses were performed at a concept level. The pumping capacities determined could change significantly as more detailed analyses are conducted.

During this phase of the study, it was necessary to determine the pumping capacities needed to lower the peak pond elevations for these three events below the zero damage elevation of 9.1 feet (the elevation of the lowest opening into a structure). It was decided that pumping capacities should be determined to lower the peak pond elevations to 9.0 feet, which is just below this zero damage elevation. The determination of the appropriate pump sizes is an iterative process, and, to follow Corps guidance, it is recommended that the pump capacities determined for each rainfall runoff event should be supplied by a minimum of three pumps. The INTDRA models for Subarea AB were therefore modified to include a number of different pumping capacities for these three pumps at various pump start elevations. After evaluating a number of different pumping capacities, it was determined that for the 100-year rainfall event the pumping capacity needed to lower the peak pond elevation from 10.7 feet to 9.0 feet would be 60,000 gpm (a 10,000 gallons per minute (gpm) pump with a pump on elevation = 7.0; a 25,000 gpm pump with a pump on elevation = 7.5; and another 25,000 gpm pump with a pump on elevation = 8.0 (producing a total pumping capacity at elevation 8.0 of 10,000 gpm + 25,000 gpm + 25,000 gpm = 60,000 gpm)). For the 25-year rainfall event the pumping capacity needed to lower the peak pond elevation from 9.7 feet to 9.0 feet would be 8,500 gpm (a 1,500 gpm pump with a pump on elevation = 7.0; a 3,500 gpm pump with a pump on elevation = 7.5 feet ; and another 3500 gpm pump with a pump on elevation = 8.0 (producing a total pumping capacity at elevation 8.0 of 1,500 gpm + 3,500 gpm + 3,500 gpm = 8,500 gpm)). The total pumping capacities needed for the 100-, 25-, and 10-year events are shown in Table 5 and plotted on Figure 2.

Table 5

Rainfall Events	Total Pumping Capacities (gpm) Needed to Reach Elevation 9.0 (NGVD29)	Total Pumping Capacities (gpm) Needed to Reach Elevation 9.3 (NGVD29)
100-year	60,000	35,000
25-year	8,500	4,000
10-year	0	0

Since there is only one structure at the minimum zero damage elevation of 9.1 feet, and since the next zero damage elevation is 9.4 feet, it was decided to determine the pumping capacity needed to pump the peak pond elevations for the 100- and 25-year

rainfall events to elevation 9.3 feet to give the local sponsor the option of reducing the pump station capacity needed and then to floodproof (or buy out) this one structure. After running a number of different pumping capacities, it was determined that for the 100-year rainfall event the pumping capacity needed to lower the peak pond elevation from 10.7 to elevation 9.3 would be 35,000 gpm (a 5,000 gpm pump with a pump on elevation = 7.0; a 15,000 gpm pump with a pump on elevation = 7.5; and another 15,000 gpm pump with a pump on elevation = 8.0 (producing a total pumping capacity at elevation 8.0 of 5,000 gpm + 15,000 gpm + 15,000 gpm = 35,000 gpm)). For the 25-year rainfall event the pumping capacity needed to lower the peak pond elevation from 9.7 feet to 9.3 feet would be 4,000 gpm (a 500 gpm pump with a pump on elevation = 7.0; a 1,500 gpm with a pump on elevation = 7.5 feet; and a 2,000 gpm pump with a pump on elevation = 8.0 (producing a total pumping capacity at elevation 8.0 of 500 gpm + 1,500 gpm + 2,000 gpm = 4,000 gpm)).

Since this a concept level design, engineering judgment, not design analyses, was used to select a 4-foot high by 6-foot wide box culvert to convey the drainage from the large channels and the stream near the upstream end, and the large channel near the downstream end, of the proposed levee. This culvert would only convey this drainage when Cameron Run was conveying a large enough discharge to block the storm drain pipes and culverts which convey the interior drainage to Cameron Run. During this blocked condition the pump station would be in operation to prevent the ponding of the interior drainage from causing damage to the residences in the vicinity of the proposed levee.

For the 100-year rainfall runoff event, the concept level size of the outlet pipes for the pump station discharge was determined by selecting a pipe size which can discharge the total pumping capacity of the pump station for the full flow condition (when the headwater (H) divided by depth (D) equals one. Using this methodology, a 60-inch diameter RCP was selected for a total pumping capacity of 35000 gpm (pumping to elevation 9.3) and a 72-inch diameter RCP was selected for a total pumping capacity of 60000 gpm (pumping to elevation 9.0) for the outlet pipe conveying the pump discharge through the proposed levee to Cameron Run. A concrete structure would be needed to protect the location where the 60-inch and 72-inch RCPs discharge from being eroded while the pumps are in operation. The final sizes of the discharge pipes and the box culvert, as well as the dimensions of the concrete structure, will need to be determined during the detailed design.

Some time after the above described analyses were performed it was decided that fill would need to be added to the pond storage area to cover the 4-foot high by 6-foot wide box culvert. This additional fill, which would reduce the available pond storage area, would need to be taken into account during the Feasibility level design.

Once the County selected a final levee plan (Plan 2C), additional interior residual flooding analyses were conducted. The county decided that they wanted an additional pump station design performed that resulted in a total pumping capacity that would reduce the ponding elevation for the 100-year rainfall runoff event to 8.0. Again, to

follow Corps guidance, it is recommended that the pump capacities determined for each rainfall runoff event be supplied by a minimum of three pumps. The INTDRA models for Subarea AB were therefore modified to include different pumping capacities for these three pumps at various pump start elevations. It was decided to run a total pumping capacity of 140,000 gpm, hoping that the resulting peak pond elevation would be less than or equal to 8.0. It was determined that for the 100-year rainfall event a 140,000 gpm total pumping capacity lowered the peak pond elevation from 10.7 feet to 7.8 feet. This pump station would have a 20,000 gpm pump with a pump on elevation = 6.0; a 60,000 gpm pump with a pump on elevation = 6.5; and another 60,000 gpm pump with a pump on elevation = 7.0 (producing a total pumping capacity at elevation 7.0 of 20,000 gpm + 60,000 gpm + 60,000 gpm = 140,000 gpm)). A plot of the pumping station capacities versus peak pond elevations for the 100-year rainfall runoff event is shown on Figure 3.

For the 100-year rainfall runoff event, the size of the outlet pipe for the pump station discharge was again determined by selecting a pipe size which can discharge the total pumping capacity of the pump station for the full flow condition (when the headwater (H) divided by depth (D) equals one. Using this methodology, a 96-inch diameter RCP was selected for the outlet pipe conveying the pump discharge through the proposed levee to Cameron Run for a total pumping capacity of 140,000 gpm. A concrete structure would again be needed to protect the location where this 96-inch RCP discharges from being eroded while the pumps are in operation. The size of the discharge pipe and as the dimensions of the concrete structure, will need to be determined during the detailed design.

Another alternative investigated was to determine the pump station design which would result in a peak pond elevation of 8.0 with the excavation of the pond storage area to elevation 6.0. The determination of the appropriate pump sizes was again an iterative process, and, to follow Corps guidance, it is recommended that the pump capacities determined for each rainfall runoff event be supplied by a minimum of three pumps. The INTDRA models for Subarea AB were modified to include a number of different pumping capacities for these three pumps at various pump start elevations. After running a number of different pumping capacities, it was determined that for the 100-year rainfall event the pumping capacity needed to lower the peak pond elevation from 10.7 feet to 8.0 feet would be 100,000 gpm (a 20,000 gpm pump with a pump on elevation = 6.0; a 40,000 gpm pump with a pump on elevation = 6.5; and another 40,000 gpm pump with a pump on elevation = 7.0 (producing a total pumping capacity at elevation 8.0 of 20,000 gpm + 40,000 gpm + 40,000 gpm = 100,000 gpm)). The final selected plan assumes that the open space area will be excavated down to elevation 6.0 and that a 100,000 gpm capacity pump station will be constructed as part of the project. Plate 5 shows the extent of the residual flooding for elevation 8.0.

For the 100-year rainfall runoff event, the size of the outlet pipe for the pump station discharge was again determined by selecting a pipe size which can discharge the total pumping capacity of the pump station for the full flow condition (when the headwater (H) divided by depth (D) equals one. Using this methodology, an 84-inch diameter RCP was selected for the outlet pipe conveying the pump discharge through the

proposed levee to Cameron Run for a total pumping capacity of 100,000 gpm. A concrete structure would again be needed to protect the location where this 84-inch RCP discharges from being eroded while the pumps are in operation. The size of the discharge pipe and the dimensions of the concrete structure will need to be determined during the detailed design.

Minimum Facilities Design

The minimum facilities design, which is the sizing of the gravity outlets through the levee, is also affected by the concept design in a similar manner as the pumping capacities. These sizes could change significantly after the detailed design.

Since this is only a concept level design, it was decided to use a simplified methodology to determine the minimum facilities. The minimum facilities are the storm drain pipes and culverts which are sized to be approximately the same size that a local jurisdiction would require if a developer were constructing a storm drain system in their community. This is usually considered to be a storm drain which can convey the rainfall runoff from a 10-year or a 25-year frequency event. The methodology used to design the minimum facilities for this project was to first determine the capacity of all of the existing pipes that would contribute storm drainage to any pipe that conveys storm drainage through the proposed levee by determining headwater (HW) / diameter of pipe or culvert (D) = 1 for each pipe. A HW/D = 1 assumes a full flow is conveyed by these pipes. The pipe capacities were then added together to determine the minimum capacity needed by the pipe or culvert that would drain through the proposed levee. The storm drainage survey provided by the local sponsor showed six ditches conveyed storm drainage to the proposed levee. The design was complicated by the necessity to provide a 4' H x 6' W box culvert to convey interior drainage to the proposed pump station during high discharges on Cameron Run. This meant that to avoid any conflicts with the 4' H x 6' W box culvert it was necessary to determine the sizes of the pipes that would carry the storm drainage from the ditch locations to two culverts which would convey storm drainage through the proposed levee. This was done by summing the HW/D = 1 for all of the pipes and then selecting a pipe which had a HW/D = 1 greater than this resulting number. The two culverts were then selected based on both having a HW/D = 1 greater than the sum of all the storm drain pipes that conveyed storm drainage to each of the two culverts. The levee stations (LSs) where these ditches cross the centerline of the proposed levee are as follows: Ditch 1 = LS 23+76; Ditch 2 = LS 17+86; Ditch 3 = LS 14+32; Ditch 4 = 3+75; Ditch 5 = LS 2+28; and Ditch 6 = LS 1+02. The results of this analysis are summarized in Table 6.

It was later determined that pressure rated joints will be required for these culverts, and that these are not available for precast box culverts. Therefore, when the sizes of these culverts are finalized in the detailed analysis, the size of the culverts selected will be much closer to the actual capacity needed.

Long after the design summarized in Table 6 was completed, it was decided that a storm drain pipe size should be determined that would convey the storm drainage from

Ditch 3 through the pump station to Cameron Run. The sum of the HW/ D =1 for the 24-inch RCP (12.7 cfs) and the 18-inch RCP (6.2 cfs), which is 18.9 cfs, was used to determine an appropriate pipe size at this location. In this case, though a 30-inch RCP would provide a HW/D = 1 of 22.0 cfs, it was decided to select a 36-inch RCP which has a HW/D = 1 of 35.0 cfs for this pipe. This was done to account for any additional local storm drainage that may reach this location. Since this is a concept level design, and since 18.9 cfs was only a small portion of the storm drainage that reaches the 5' H x 6' W box culvert, it was decided not to reduce the size of this culvert. It was later decided that this 36-inch storm drain pipe through the pumping station may cause impacts to the wetlands at the location of the pump station, and it was eliminated from the design.

Table 6

Ditch Number from Survey	Sizes of Pipes Draining to Ditch	HW/D =1 for Pipes Draining to Ditch (cfs)	Size of Pipes Draining to Culvert	HW/D = 1 of Pipes Draining to Culvert (cfs)	Size of Proposed Culvert	HW/D = 1 of Proposed Culvert (cfs)
3	24-inch RCP	12.7				
3	18- inch RCP	6.2	30-inch RCP from Ditch 3 to Ditch 2	22.0		
2	36-inch RCP	35.0				
2	36-inch RCP	35.0	54-inch RCP from Ditch 2 to Ditch 1	95.6		
1	42-inch RCP	50.0				
1	24-inch RCP	12.7				
Proposed Culvert is at Ditch 1	Total for Ditches 1, 2, & 3	151.6			5’ H x 6’ W Box Culvert at Ditch 1	192.6
4	42-inch RCP	50.0	42-inch RCP from Ditch 4 to Ditch 5	50.0		
5	24-inch RCP	12.7	48-inch RCP from Ditch 5 to Ditch 6	71.0		
6	48-inch x 72-inch RCP	110.0				
6	42-inch RCP	50.0				
Proposed Culvert is at Ditch 6	Total for Ditches 4, 5, & 6	222.7			6’ H x 8’ W Box Culvert	338.4

Additional Analyses Recommended

The remaining tasks to complete a detailed residual flooding analysis include:

1) Develop hypothetical rainfall runoff models (HEC-1) for interior runoff using parameters determined in previous work efforts along with newly developed precipitation data. The 2-, 5-, and 50-year events for the subareas remain to be done.

2) Input data into historical rainfall runoff models (HEC-1) for interior runoff using parameters determined in previous work efforts, along with selecting the historical events and developing the precipitation data, assuming 6 events for each subarea. The work efforts in 1) and 2) also assume that changes will need to be made to the HEC-1 models to divert a portion of the flow from Subarea A to Subarea B due to conflicts with the culvert which will carry runoff to the proposed pumping station.

3) Determine information needed for the interior drainage (INTDRA) models for the subareas along the levee (determining gravity and pressure flows for the outlet pipes/culverts, river hydrographs for the historic events, and river flow data, etc.)

4) Develop INTDRA models for hypothetical events using information from Item 3) for the subareas along the levee. Assumes improved conditions (with project) models for 6 events for the subareas.

5) Develop INTDRA models for historical events using information from Item 3) for the subareas along the levee. Assumes improved conditions models for 6 events for the subareas.

6) Size minimum interior drainage facilities.

7) Pump station modeling for one location.

8) Develop two joint probability stage-frequency curves for interior flooding.

9) INTDRA analyses for the 6 hypothetical events for one pond excavation alternative.

10) INTDRA analyses for the 6 historical events for one pond excavation alternative.

11) Pump station modeling for one pond excavation alternative.

12) Develop two joint probability stage-frequency curves for interior flooding with one pond excavation alternative.

13) Delineate limits of residual flooding for interior and from river for the 100-year (1% chance exceedance frequency) and one historical event using GIS.

14) Report writing and preparation of tables, figures, and plates.

US Army Corps of Engineers
Baltimore District

Plate 1

Drainage Areas for Interior Residual Flooding Analysis

0 500 1,000 2,000 Feet

Aerial Photograph dated 2004 courtesy of AerialExpress

Legend

Subarea Boundary

N

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Plate 2

Huntington
Interior Residual Flooding
Without Pump
100-year Rainfall Ponding

0 300 600 1,200 Feet

Aerial Photograph dated 2004 courtesy of AerialExpress

Legend

N

Residual Flooding Elevation 10.7 (NGVD29)

US Army Corps of Engineers
Baltimore District

Plate 3

Huntington
Interior Residual Flooding
Without Pump
25-year Rainfall Ponding

0 300 600 1,200 Feet

Aerial Photograph dated 2004 courtesy of AerialExpress

Legend

N

Residual Flooding Elevation 9.7 (NGVD29)

US Army Corps of Engineers
Baltimore District

Plate 4

Huntington
Interior Residual Flooding
Without Pump
10-year Rainfall Ponding

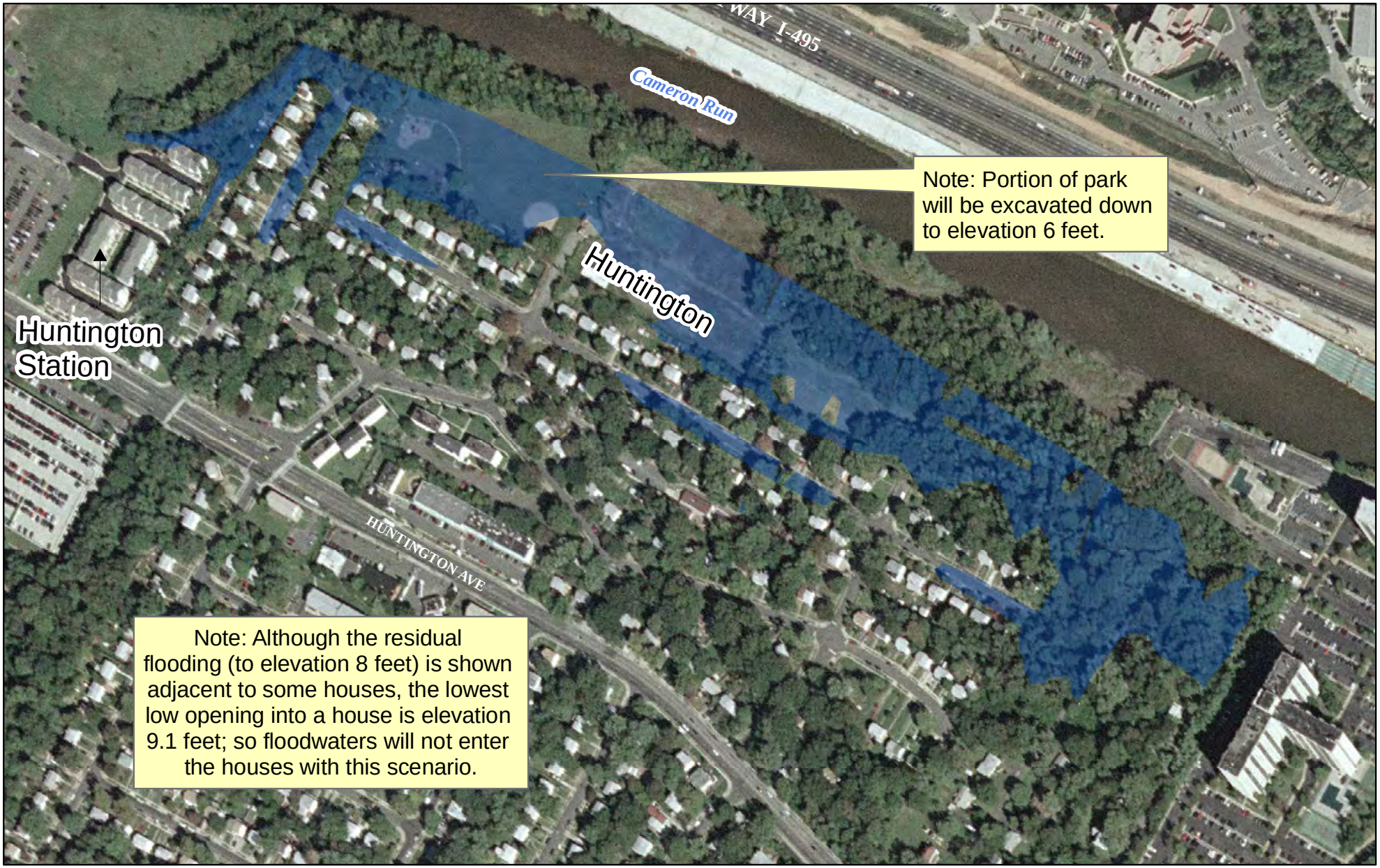
0 300 600 1,200 Feet

Aerial Photograph dated 2004 courtesy of AerialExpress

Legend

Residual Flooding Elevation 9.0 (NGVD29)

N



US Army Corps of Engineers
Baltimore District

Plate 5

Huntington
Interior Residual Flooding
for 100-year Rainfall Ponding
with a 100,000 gpm Pump Station
and with excavated ponding area.

0 300 600 1,200 Feet

Aerial Photograph dated 2004 courtesy of AerialExpress

Legend

N

Residual Flooding Elevation 8.0 (NGVD29)

Figure 1

Huntington, VA (Cameron Run) Residual Flooding Elevations (NGVD29) for Subarea AB

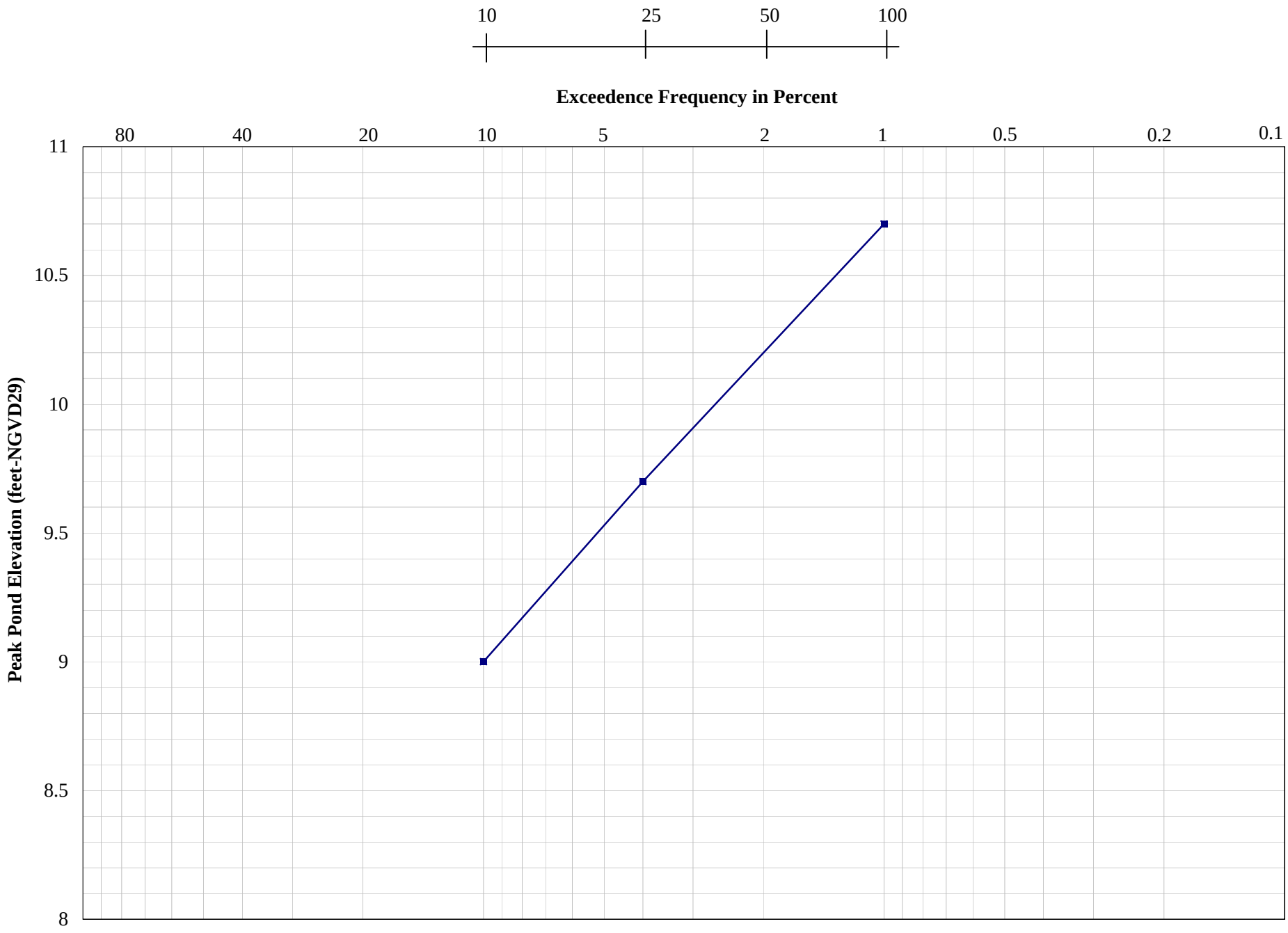


Figure 2

Huntington, VA (Cameron Run) Total Pumping Capacity Needed to Reach Zero Damage

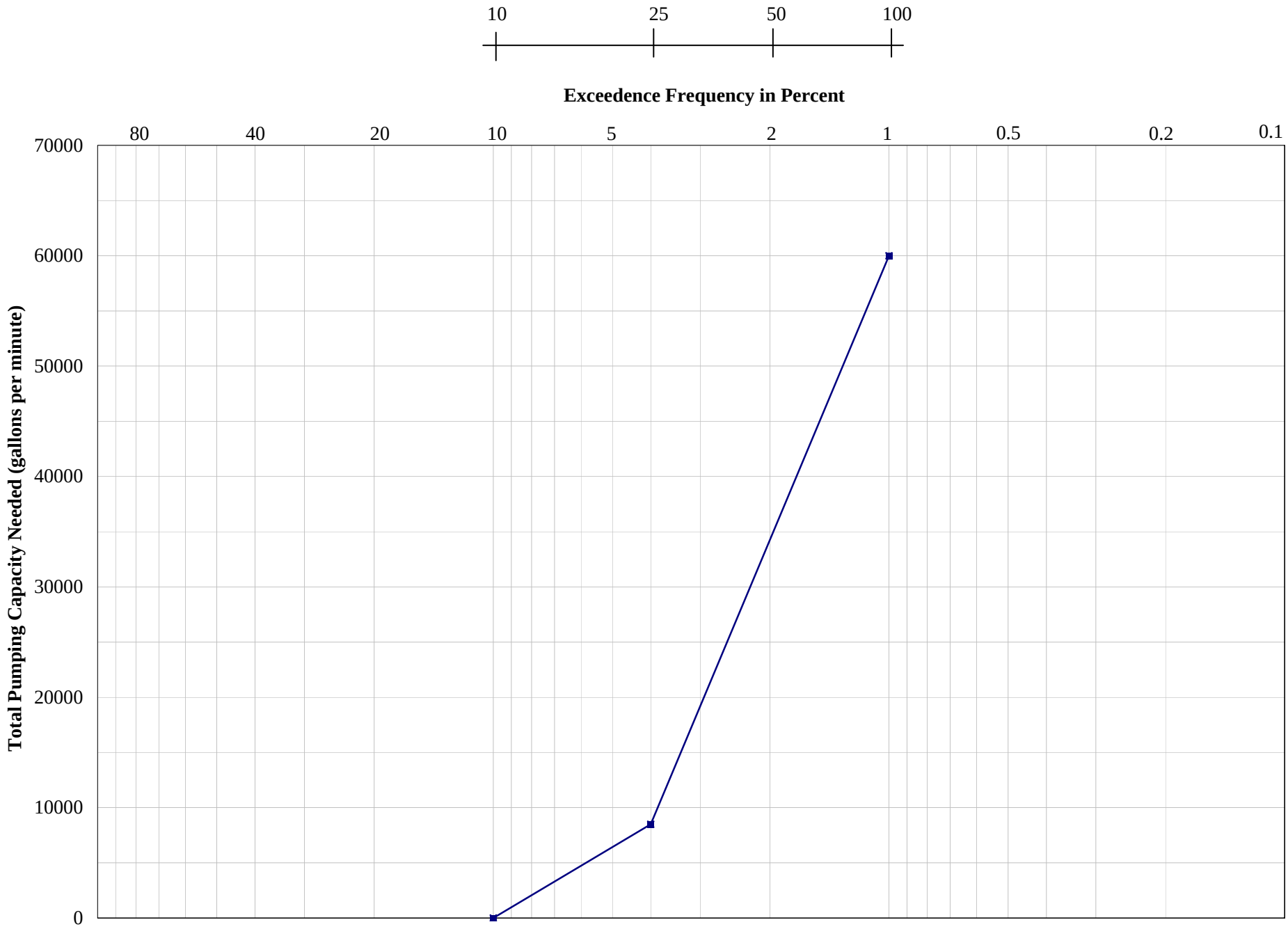
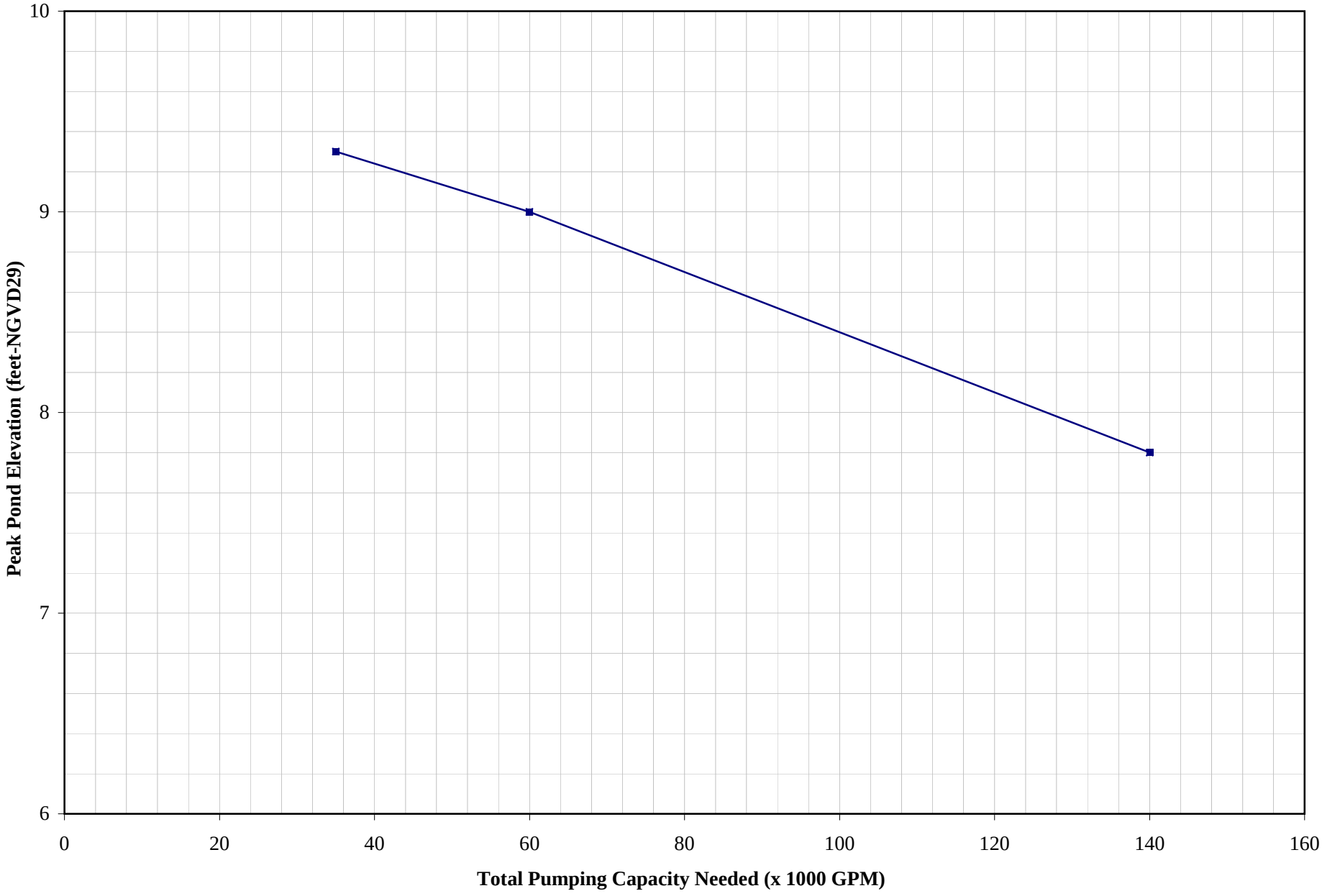


Figure 3

Huntington, VA (Cameron Run) Pumping Station Capacities Versus Peak Pond Elevations