7.0 DESIGN OF FINAL PLAN

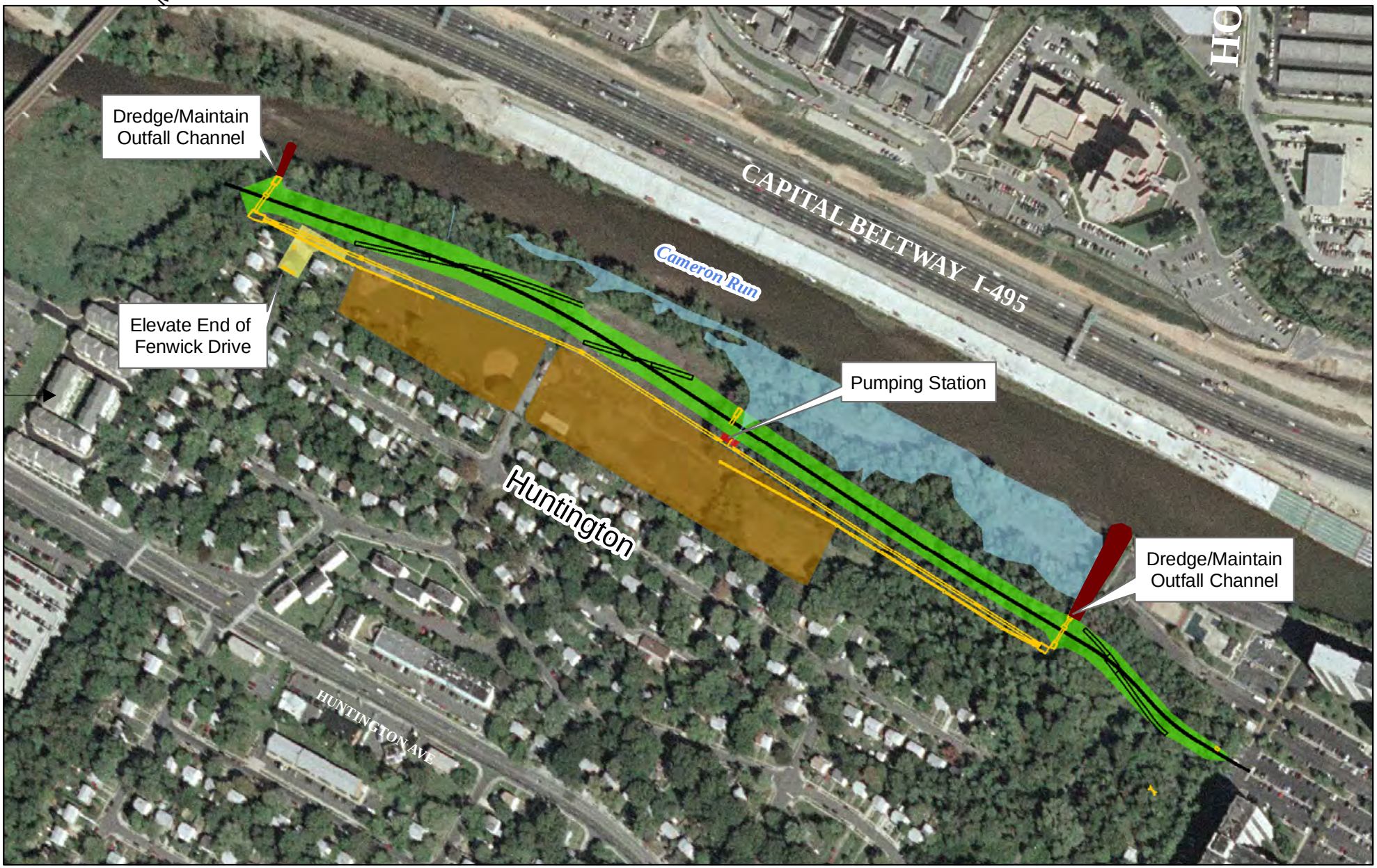
The main components of the selected plan (Final Concept Plan 2c) are a levee and a pumping station. As part of this study, the Corps developed the levee to a 65% design level. Further design of the levee will be required to take it to a 100% level so that it may proceed to construction. The Corps only designed the pumping station to a concept level design stage. This concept plan allowed the team to develop an approximate construction cost estimate. However, significant further design will be required for the pumping station and the accompanying features (such as the flow diversion pipes). The project design is described in this section. For further design details, refer to the Engineering Appendix G.

7.1 LEVEE DESIGN

Figure 7.1 shows a plan view of the project. The levee is 2,865 feet long and will tie into high ground upstream and downstream of Huntington. The project consists of two drainage structures, one near each end, to allow stormwater to flow through the levee. There are flow diversion pipes to divert flow to the pumping station during high water events, when the drainage structures are closed. There is an 8-foot wide asphalt recreational path along the top of the levee and ramps that lead over the levee for maintenance and handicap access. The detailed civil engineering drawings of the project can be found in Appendix G1.

The project is designed to prevent flood damages to the Huntington houses during the 1% annual chance flood event (100-year flood) and lower events. The crest elevation at the upstream end of the levee is 19.4 feet (4 feet higher than the 1% annual chance flood elevation); the crest elevation at the downstream elevation is 17.3 (3 feet higher than the 1% annual chance flood elevation). The additional height above the 1% annual chance flood elevation is to allow for risk and uncertainty and sea level rise (see sections below for more details). The levee height meets FEMA certification standards. The top of the levee will be approximately 10 to 15 feet above the existing ground.

A typical cross section of the levee embankment is shown on Figure 7.2. The proposed embankment has a 10-foot wide crest and 1 vertical on 2.5 horizontal (1.0V:2.5H) side slopes. A 6-foot deep trapezoidal impervious cutoff/inspection trench will be located beneath the levee along the alignment centerline. The embankment will be constructed primarily using select earth material from a borrow source obtained by the contractor. The select earth material will consist of most impervious materials available in the area. The levee will be covered with grass. A combination blanket and toe drain will be placed along the landside toe of the levee embankment. The combination blanket and toe drain will intercept seepage through the semi-pervious foundation blanket layer and will reduce potential uplift pressures along the base of the levee. The blanket drain will also provide a means of collecting any possible internal seepage and provide an exit for the collected seepage at the landside levee toe. A trapezoidal toe drain will be placed to a depth of about 5 feet below the existing ground surface at the levee toe at



US Army Corps of Engineers
Baltimore District

Figure 7.1

Levee Project
Plan View

0 300 600 1,200 Feet

1 inch = 300 feet

Aerial Photograph dated 2004 courtesy of AerialExpress

Legend

- Levee
- Elevate Road
- Excavated Area for Ponding
- Pumping Station
- Delineated Wetlands
- Dredge/Maintain Outfall
- New Drainage Pipe
- Recreation Trail
- Maintenance/Handicap Ramp

N

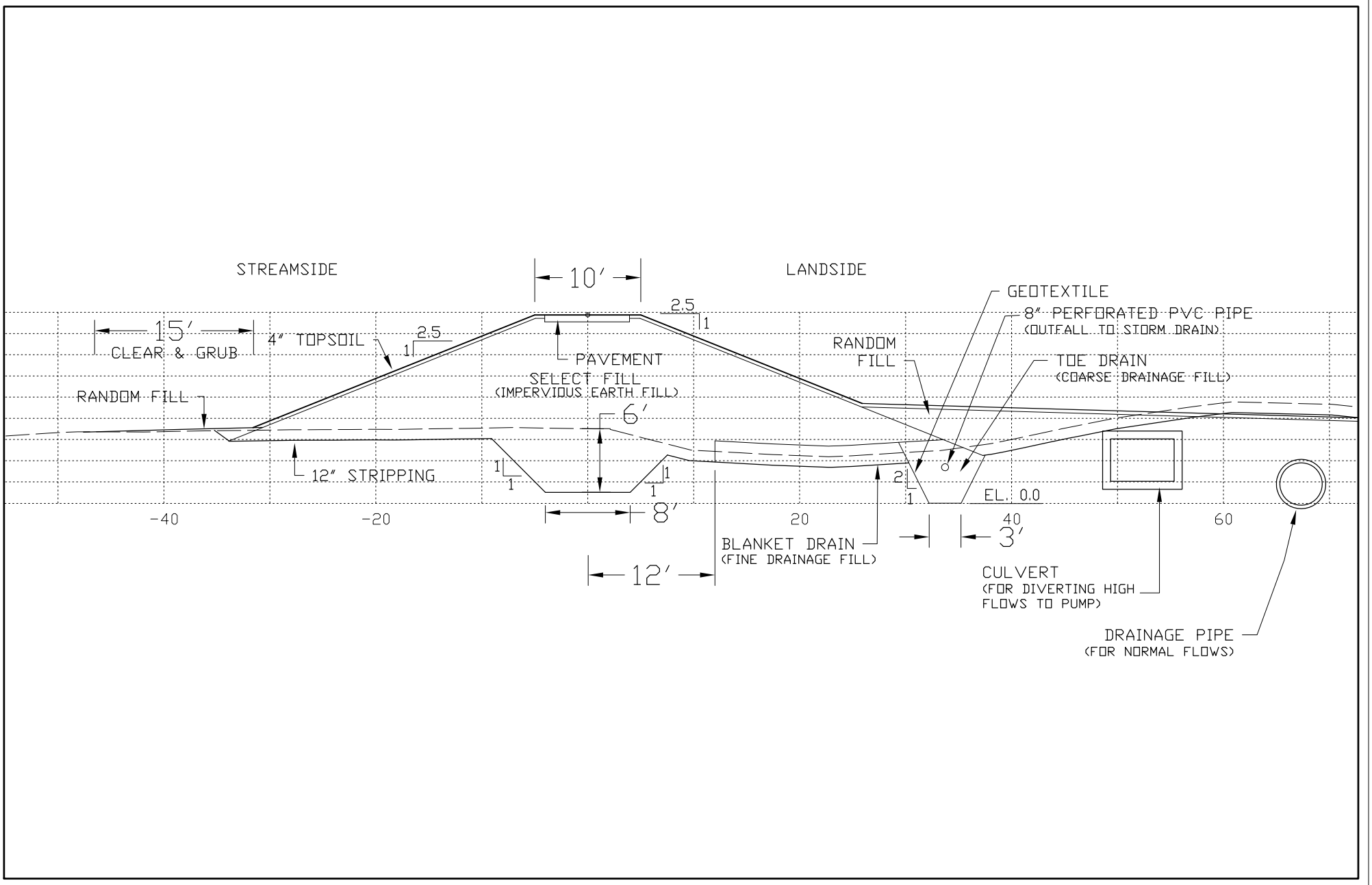


Figure 7.2 Cross-Sectional View of Levee

US Army Corps of Engineers
Baltimore District

HUNTINGTON
FLOOD DAMAGE REDUCTION STUDY
FAIRFAX COUNTY, VIRGINIA

DATE: OCT 2008
SCALE: NOT TO SCALE
FIGURE 7.2

elevation 0.0. An 8-inch perforated plastic pipe will be installed in the toe drain to collect and convey the seepage flow beyond the levee toe. The gravel toe drain will be surrounded by geotextile filter to prevent contamination of the gravel by the fine grained foundation soils. The fine and coarse aggregates for the drains will be obtained from off-site commercial sources.

The project will also include excavating part of the open space/park area approximately 1-2 feet deeper to elevation 6.0 feet to allow for more rainfall storage during a flood event. This is discussed further in the interior residual flooding analysis Section 7.3.

7.1.1 Levee Drainage Structures

Currently, the Huntington community is drained to Cameron Run by six drainage channels/pipes. It is advantageous to provide a few large gravity outlets rather than numerous smaller outlets. Two gravity drainage structures through the levee are proposed, one at each end of the levee. The two drainage structures will have flap gates and sluice gates. The flap gates are designed to close automatically when floodwaters rise. The sluice gates provide a secondary way to prevent backflow if the flap gates fail. An additional drainage structure was investigated near the middle of the levee, but it would require excavating and maintaining an outfall channel through wetlands. The County, Corps team and Corps' Wetland Regulatory personnel discussed this issue and to avoid impacts to the wetlands, it was decided that the third drainage structure would not be included in the project. A storm drain system adjacent to the levee is proposed to direct normal flows to the drainage structures. This system will be susceptible to siltation due to the mild slope, so frequent maintenance is expected.

The existing storm drain outfalls are set very low, within the tidal range. The new drainage structures will likewise be set low. Based on the Cameron Run cross-sections, it appears that the stream has silted in above the existing storm drain outfall elevations. Therefore, the area along Cameron Run near the outfalls and the storm drain outfall channels should be dredged. Since this stream rises rapidly, the system is heavily dependent on the automatic flap gates. As a result, maintenance dredging of the outfall channels down to elevation -2.5 will be required to maintain operation of the gates. This may require some maintenance dredging of Cameron Run as well.

To minimize excavation and stream impacts, the westernmost drainage structure is aligned with the existing drainage channel. For the inlet and outlet structure, non-flared walls were chosen for the same reason. The backwall of the structures are parallel to the levee contour to avoid warping the levee slope, and the box culvert will extend beyond the outlet structure to facilitate the flap gate. This design is based on similar structures used at nearby Fourmile Run.

Box culverts were chosen by the hydraulic engineer for the drainage structures through the levee. Corps' engineering guidance only discusses using circular concrete pipe for levees and also specifies using joints that are pressure rated. Since precast box culverts typically do not have pressure-tight joints, cast-in-place box culverts are assumed for the box culverts beneath the levee. Further investigation of precast box culvert joints, or alternative circular pipes, should be done during the next phase. Drainage fill is provided around the landside ends of the pipes. For

pipes *beneath the levee foundation*, outletting the drainage fill to the surface by encapsulating the next manhole in drainage fill is required. This is the case for the sanitary drainage structure.

7.1.2 Access Routes

The levee can be accessed from three locations as shown on Figure 7.1 and on drawings in Appendix G1 (shown as maintenance/handicap ramps). Two of the routes are to be elevated above the anticipated interior ponding elevation (8.0 feet), but the Fenwick Drive access point is below the anticipated interior ponding elevation (8.0 feet). The end of Fenwick Drive is shown to be raised to accommodate the diversion culverts; however, it will still be up to 1-foot below the ponding elevation.

Ramps are provided to the top of the levee and down to the riverside toe. To allow for recreational access, two handicap accessible ramps are provided. The surface of the ramps and levee crest will be paved. A maintenance route is shown along the riverside toe to allow for maintenance of the drainage structure outfalls.

7.1.3 Sanitary Sewer Impacts

The line of protection will cross an existing 48" sanitary sewer pipe and an existing 16" sewer pipe near the east end of the levee. The Corps' engineering manual discourages pipes beneath levees, and allows them only if they meet certain criteria. The criteria typically requires upgrading the pipe and adding provisions for emergency closure to prevent floodwaters from backing up through the pipe.

Since the 48" pipe crosses the line of protection, a drainage structure with a sluice gate would be required to prevent backflow of floodwater, assuming that the pipe would be flooded by Cameron Run. This sluice gate control manhole would need to be monitored during a flood event, and the sluice gate would only be closed if backflow was observed. Construction of a sluice gate on an existing 48" sanitary sewer pipe raises the following concerns:

1. Sewage flow would need to be maintained during construction.
2. It would require personnel on the ready to close the gate during a storm.
3. Since this stream rises rapidly, it would be difficult to close the gate in time.
4. Closing the gate would cause sewage flows to overflow into the protected area.
5. Closing the gate would impact the Jones Point Pumping Station.
6. The sewage flows may have to be considered in the stormwater pumping station design.

Due to these concerns, the assumption that the sewer would be flooded was investigated. The as-built drawings for the 48" sewer were reviewed for possible flood entry points:

- The manhole located just on the riverside of the levee is susceptible to flooding. However, it may be possible to waterproof and anchor it to avoid flooding.

- The 48” sewer transitions into an inverted siphon in order to pass beneath Cameron Run. The influent chamber to the siphon is underground, but it has an above-ground access structure with a low opening around el. 15.0. This is above the 100-year water surface profile, but below the top of protection.

- The siphon consists of ductile iron pipes encased in concrete and supported on piles as it crosses below Cameron Run. Risk of flooding appears minimal.

- The outlet chamber for the siphon is underground, but it has a 10” air vent pipe with an opening around el. 15.0. This is above the 100-year water surface profile, but below the top of protection.

- The next manhole north of the beltway is elevated above the levee, and flooding is not expected.

The risk of sewer flooding was discussed with the Corps’ engineering technical leaders, and the conclusion was that this 65% design should include a sluice gate drainage structure on the 48” sanitary sewer.

The 48” pipe beneath the levee was reviewed for conformance with the strength and watertightness requirements. As-built plans for the 48” pipe are dated 1980. The pipe is noted as ASTM C-76, Class IV RCP on a concrete cradle. It is buried 10-feet deep, and an additional 5-feet of levee will be constructed on top of it. This type of pipe meets the minimum requirements, but there is no indication of the type of joints used. Therefore, since a portion of the pipe will have to be removed anyway to construct the sluice gate structure, all of the pipe beneath the levee will be replaced. Drainage fill is provided around the landside end of the pipe. For pipes *beneath the levee foundation*, the engineering manual requires outletting the drainage fill to the surface by encapsulating the next manhole in drainage fill.

The 16” pipe could be relocated to avoid crossing the levee, or it could be modified to meet the criteria. Relocation has the advantage of less risk, but may be more costly. This was discussed with the Corps’ engineering technical leaders, and the conclusion was that the feasibility design should include relocating the 15” pipe.

The 16” sewer pipe outfalls to a pumping station (Jones Point Pumping Station) located near the east end of the levee. Based on the as-built plans, the wet well will not flood until flood waters rise above elevation 18.0; therefore, direct flooding of the well is not expected. As a precaution, backflow preventers are recommended on all sanitary house connections to prevent sewage backflow in the event that the pumping station fails; however, they are not included in the levee project or the levee project cost estimate.

The stormwater pumping system will also impact the sanitary sewer system. If interior ponding behind the levee extends over the sanitary manholes, stormwater could flood the manholes. Stormwater and sewage could then backup into basements through the sanitary house connections. It is assumed that the manholes within the ponding area are not watertight. Therefore, they should be waterproofed and modified with watertight manhole lids. They may

also need to be modified to prevent flotation. Stormwater and sewage could also backup into the basements through any house cleanouts that may exist within the ponding areas. Therefore, backflow preventers may be needed on all sanitary house connections.

7.1.4 Other Utility Impacts

As shown on the project drawings in Appendix G1, light poles and a television line will need to be reconstructed. The roadway work will also require raising some manholes, meter covers, valve covers, and a hydrant.

7.1.5 Regulatory/Compliance Considerations

Construction of the project will affect existing wetlands, streams, tidal waters, and trees. Required permits/approvals may include: erosion and sediment control, NPDES, possible forest conservation (if applicable), and state and federal authorization to work in streams and wetlands.

7.2 TOP OF PROTECTION

7.2.1 Risk and Uncertainty Analysis

At the request of Fairfax County, a risk and uncertainty analysis was performed for Plan 1c (highest levee with a pump and no dredging) and Plan 2c (highest levee with a pump and dredging) although this is currently not a requirement by FEMA for levee certification. The FEMA requirement is that the levee needs to be a minimum of 3 feet higher than the 1% annual chance flood elevation. If the county constructs this project without the Corps' involvement, they are required to meet that standard. The risk and uncertainty analysis that was conducted is a Corps of Engineers requirement for levee certification.

The purpose of a risk and uncertainty analysis is to provide decision-makers more information with which to select the appropriate size of the project. Risk and uncertainty is encountered in three areas: hydrology and hydraulics, geotechnical design, and economics. A Hydrologic Engineering Center-Flood Damage Analysis (HEC-FDA) Program is used to evaluate the risk and uncertainty in projects. The analysis attempts to describe the error based on uncertainty and present the results to the decision-maker in terms of project reliability. The FDA model is a program that calculates flood damages allowing relationships between H&H variables and economic variables to be uncertain. With regard to H&H it recognizes the uncertainty in the flow frequency relationship and the flow stage relationship. For economic magnitudes it allows for uncertainty in structure value, content value, damages that occur at each stage of flooding, and first floor and start of damage elevations. A Monte Carlo simulation is used to capture this uncertainty in the estimation of damages. The damages are then converted to an annual value. The results of the analysis are summarized below; for further details, see the risk and uncertainty sections in Appendix D (Economics) and Appendix G3 (Hydrology and Hydraulics).

HEC- FDA through a project performance analysis determines the degree of "assurance" (i.e., conditional non-exceedance probability) that each frequency event will be contained by the levee

or floodwall in each reach (and the levee will not be overtopped). A value of at least 90% assurance is required to meet levee certification requirements.

The HEC-FDA analysis was performed for Plan 1c and Plan 2c and Table 7.1 below presents results for Plan 2c.

Table 7.1: Risk and Uncertainty Results for Plan 2c

Risk and Uncertainty Analysis Results						
For Plan 2c						
	Conditional Non-Exceedance Probability by Events					
	10%	4%	2%	1%	0.4%	0.2%
Plan 2c	0.9999	0.9997	0.9986	0.9921	0.9783	0.9705

As shown in the table, the non-exceedance probability for the 1% flood is 99.21% for Plan 2c. The conditional non-exceedance probability is above the required 90% for the stream reach for Plan 2c for the 1% (or 100-year) flood. Therefore, the project meets the levee certification requirements for assuring that it provides a 100-year level of flood protection. The non-exceedance probability for Plan 2c is much larger than required, which indicates that a lower top of protection may be possible.

At the time of the analysis, Plans 1c and 2c were still being considered. It was decided to investigate the possibility of decreasing the top of protection profile for Plan 1c (levee and dredging) by 0.5 feet. The FDA model was run with this new lower levee and it showed that the conditional non-exceedance probability for the 1% annual chance event would be 96.7%. The original Plan 1c had a conditional non-exceedance probability for the 1% annual chance event of 99.5%. It is assumed that Plan 2c would have a similar reduction in probability.

To meet Corps levee certification requirements for projects, levees that are 3 ft or higher above the 1% chance event water surface elevations must have a non-exceedance probability of at least 90%; whereas, those with less than a 3 ft increment must have a non-exceedance probability of at least 95%. The non-exceedance probability for the 1% event of the lower top of protection plan 1c is 96.7%, which still meets the requirement of 95% for levees with less than a 3 ft increment above the 1% chance exceedance event water surface elevation. It is likely that if Plan 2c was lowered by 0.5 feet, that it would also have a probability greater than 95%. Therefore, the decreased top of protection would likely still meet the risk and uncertainty requirements for levee certification by the Corps. However, concurrent with the risk and uncertainty analysis, a sea level rise analysis was being conducted that also affected the levee crest elevation.

7.2.2 Sea Level Rise Analysis

After the team conducted the risk and uncertainty analysis and determined that the levee could be lowered by 0.5 feet so that the downstream end would be 2.5 feet higher than the 1% annual chance flood and the upstream end would be 3.5 feet higher than the 1% annual chance flood, the team conducted an analysis of the impact sea level rise (SLR) might have on the final design

crest height of the levee. To determine this impact, existing HEC-RAS models were re-run to measure the impact that raising the downstream section water level might have on the upstream area where the levee sits. Impacts were measured for the 1% annual chance flood, which is the project design flood, for both sea level rise over a 50-yr period and a 90-yr period. A 50-year period was chosen because that is typically the duration that is used when determining the economic costs and benefits of a Corps flood damage reduction project. The project team also wanted to look at the impact over a longer period of time and since the National Research Council (NRC) report (NRC, 1987) only makes SLR predictions to the year 2100, we selected a 90-year period (assumes the year 2010 to year 2100). Current Corps standards call for consideration of local historic rates, projected NRC rates, and projected Intergovernmental Panel on Climate Change (IPCC) rates (IPCC, 2007) when planning and designing projects that could be affected by sea level rise. Estimates for both the 50-yr and 90-yr SLR rates were made from each of these reports and then run in HEC-RAS to determine their impact on future water levels at the levee. Details of the sea level rise analysis can be found in Appendix G4.

The best sources of information for historic sea level trends are local tide gauges. For this study, data from the nearest NOAA tide gauge, Station ID 8594900, in Washington D.C. were used to estimate SLR for Huntington. According to this gauge, mean sea level has risen roughly 0.933 ft per 90-yrs based on monthly mean sea level data from 1924 to 2006. Likewise, the 50-yr local historic SLR estimate is 0.52 ft per half century.

The 1987 NRC report and its estimates are based on the assumption that there is a high probability that global SLR will greatly accelerate with time and that there are uncertainties associated with these estimations. To account for the uncertainties of future SLR accelerations, the NRC report examines three eustatic rises to the year 2100 from 1987: 1.64 ft (0.5 m), 3.28 ft (1 m), and 4.92 ft (1.5 m). The Corps chose the highest and lowest NRC predictions (Curve 1 and Curve 3 from the NRC report) for this analysis.

The 2007 IPCC report examines six climate evolution scenarios. However, only the rise and rate of rise for two scenarios (B1 and A1F1, smallest and greatest, respectively) were evaluated as part of this analysis. Based on the IPCC rates, A1F1 provides sea level rises of 1.94 ft per 90-yrs and 1.08 ft per half century. Likewise, B1 provides 1.25 ft per 90-yrs and 0.69 ft per half century.

The impact of SLR was incorporated into the HEC-RAS models by simply adding the SLR values to the downstream known water surface elevation at the mouth of Cameron Run and the confluence with the Potomac River. The steady flow model was then run for the 1% annual chance flood event and the resulting water level compared to the previously accepted plan in which the downstream end of the levee is at elevation 16.8 ft (2.5 ft above the 1% annual chance elevation) and the upstream end is at elevation 18.9 ft (3.5 ft above).

The SLR values used were the minimums and maximums from the NRC report and the IPCC report for both the 50-yr and 90-yr periods. Since the local historic estimates are less than both the NRC and IPCC estimates, they were not included in the HEC-RAS modeling. Table 7.2 below shows the resulting change in water surface elevation (WSE) at both the upstream and downstream ends of the Huntington levee for each SLR value. This change is measured with

respect to the WSE for the current plan as described above. Note that only the SLR of 4.66 ft by year 2100 from Curve 3 in the NRC report causes any increase in the predicted WSE at the levee. This increase is 0.3 ft at the upstream end of the levee and 0.35 ft at the downstream end.

Table 7.2: NRC and IPCC SLR Rates used for HEC-RAS Modeling and their Corresponding Impact on 1 % Annual Chance Flood WSE Upstream Near the Levee along Cameron Run

Method		Sea Level Rise (ft)		WSE Change at Levee (ft)	
		50-yr	90-yr	50-yr	90-yr
NRC	Curve 1	0.46	1.51	0	0
	Curve 3	1.38	4.66	0	0.35 D/S, 0.3 U/S
IPCC	A1F1	1.08	1.94	0	0
	B1	0.69	1.25	0	0

As described above, the Corps and County evaluated a levee that is 3 feet above the 1% annual chance flood elevation at the downstream end and 4 feet above at the upstream end. Based on the results of the risk and uncertainty analysis, the county was considering lowering the top of levee to 2.5 feet above the 1% annual chance flood elevation at the downstream end and 3.5 feet above at the upstream end. However, based on the sea level rise analysis, Fairfax County decided to increase the top of levee back up to the original elevations (3 feet above and 4 feet above at the downstream and upstream ends, respectively). This is a conservative decision, as the sea level rise analysis showed that only the most extreme prediction (4.66 feet rise in 90-years) has an impact at Huntington, and the impact is only 0.35 feet. Table 7.3 shows the final top of levee elevations at various cross sections.

Table 7.3: Levee Top of Protection Elevations

River Cross Section	Levee Crest Elevation (Top of Protection), feet (NGVD 29)
1389	19.4
1240	19.0
1100	18.6
1000	18.3
860	17.9
760	17.6
660	17.3

7.3 HYDRAULIC IMPACT OF PROJECT

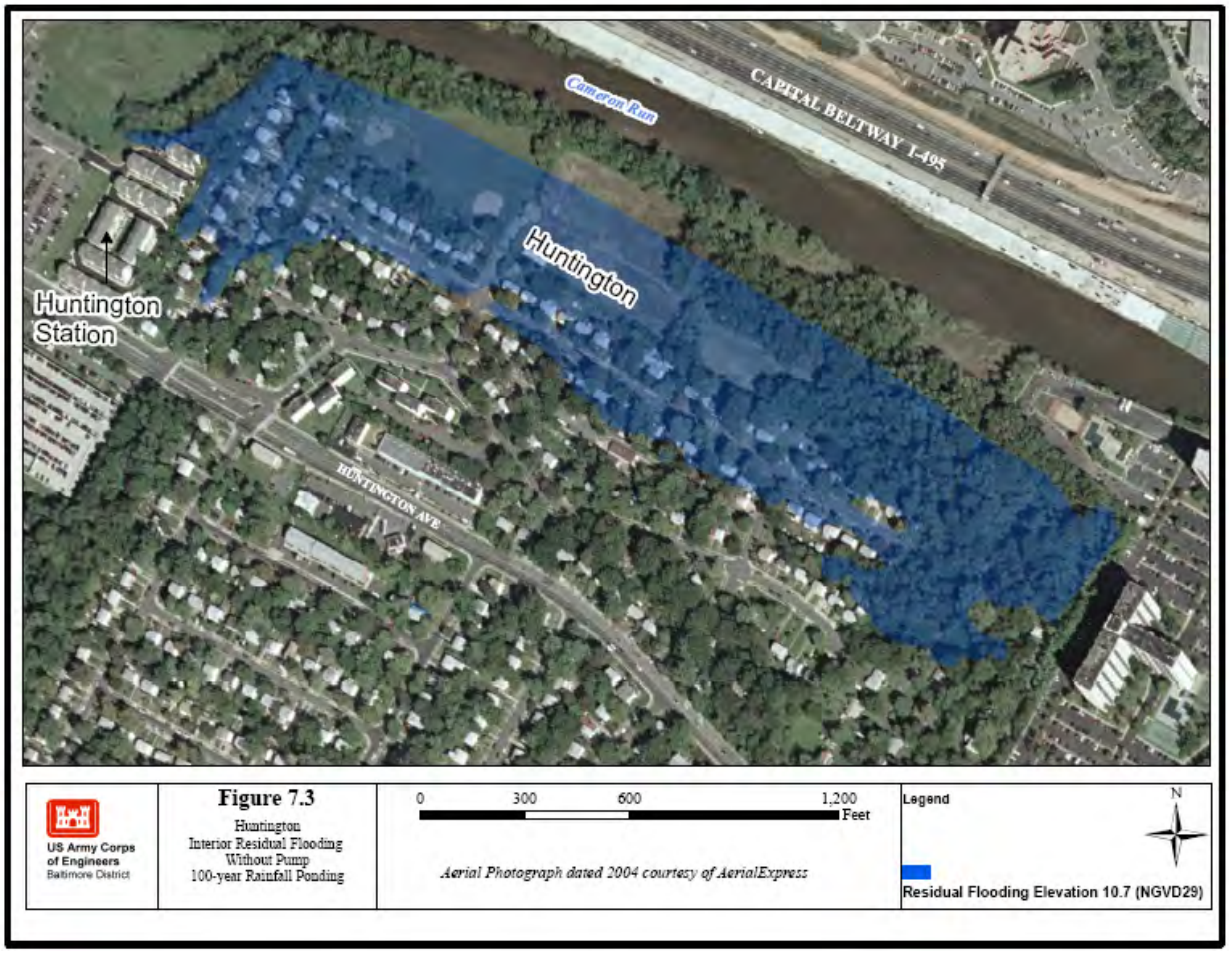
The selected levee plan will have an impact on the 1% annual chance flood elevations adjacent to and upstream of the levee. Based on hydraulic modeling, this project will increase the 1% annual chance flood elevation by up to 0.6 feet just upstream of the project (See Appendix G3). These impacts are the same as the Final Concept Plan 2c impacts; see Section 6.3.1 for further information on these impacts.

7.4 INTERIOR RESIDUAL FLOODING ANALYSIS

When the levee is constructed and the storm drain pipes are closed during high water events (to keep the flood waters from Cameron Run from backing up through the pipes and flooding Huntington), rainfall runoff will have nowhere to go. This rainfall runoff will pond on the landward side of the levee. A preliminary interior residual flooding analysis was conducted as part of this study to determine how high the water will pond behind the levee and to determine what size pumping station would be needed to pump the water over the levee into Cameron Run. The details of this analysis can be found in Appendix G5.

The preliminary analysis determined that if the flow of water in Cameron Run is higher than the storm drain outfalls (and the flap gates close) and a 100-year frequency rainfall occurs at Huntington, the water behind the levee will pond to a peak elevation of 10.7 feet (Figure 7.3). This would cause a number of houses in Huntington to flood from this interior drainage. Fairfax County decided to design a pumping station that would handle the 100-year rainfall without causing any damages to structures. The lowest low opening into a house is elevation 9.1 feet. The Corps and County evaluated various alternative methods for reducing the peak interior ponding elevation below elevation 9.1 feet. Initially, it was decided that the pumping capacity should be determined to lower the peak pond elevations to 9.0 feet. 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 9.0 feet is 60,000 gallons per minute (gpm) (this was size of pump assumed for the final concept plans). However, there would still be significant ponding in yards and streets. Later during the 65% design, it was decided to reduce the peak ponding water surface elevation to a maximum of 8 feet to keep the flood waters further from the houses. There would still be some ponding along yards and roads. The required pumping capacity to maintain a maximum peak elevation of 7.8 feet was determined to be 140,000 gpm.

Figure 7.3: Interior Residual Flooding Without Pump, 100-year Rainfall

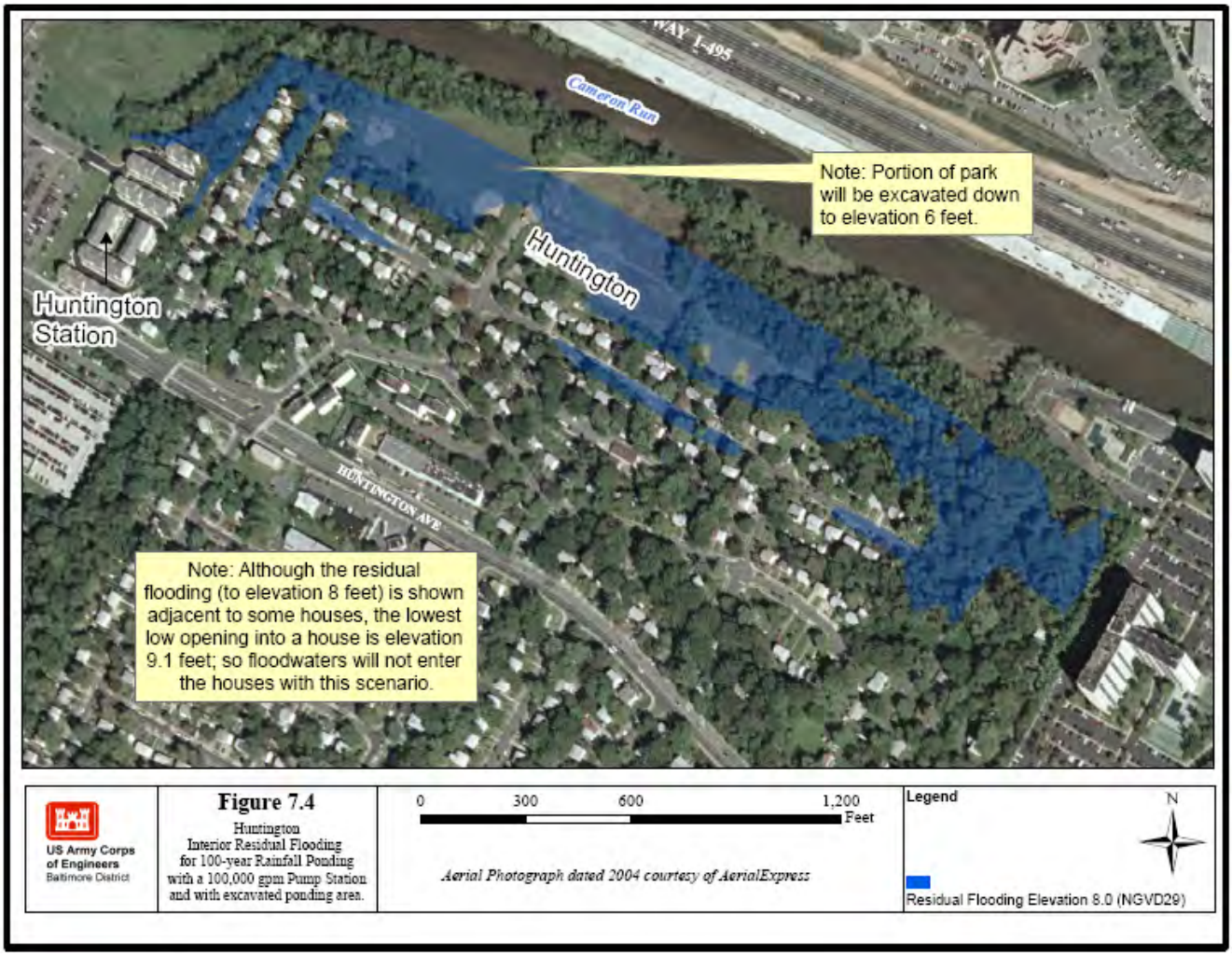


In order to reduce this large pumping requirement and gain more pond storage capacity, the county requested that the Corps investigate excavating down part of the open space/park area. The excavated area was limited to the park adjacent to the levee, but does not extend into the wooded areas. Geotechnical limitations required keeping the bottom at elevation 6.0 feet or greater. Higher uplift gradients caused by seepage beneath the levee are a concern if the area is excavated any deeper. Due to this, the only way to provide significant storage is to grade the entire area down to elevation 6.0 feet. As a result, the area will drain poorly, and will likely not be suitable for park activities. The area that will be excavated lower is shown on the levee plan, Figure 7.1. At the request of the County, this excavated ponding area is part of the final project and is included in the cost estimate. The interior residual flooding analysis was re-run with this area excavated down to elevation 6 feet and it was determined that the total pumping capacity necessary for the project is now 100,000 gpm (Table 7.4). Figure 7.4 shows where the water will pond up to elevation 8 feet during a 100-year rainfall.

Table 7.4: Pumping Capacities for Various Interior Drainage Alternatives

Interior Drainage Alternative	Pumping Capacity Required (gpm)
Maintain Peak Pond Elevation at 9 Feet	60,000
Maintain Peak Pond Elevation at 7.8 Feet	140,000
Excavate Open Space to Elevation 6 Feet and Maintain Peak Pond Elevation at 8 Feet	100,000

Figure 7.4: Interior Residual Flooding for 100-year Rainfall with 100,000 gpm Pump and with Excavated Ponding Area



7.5 HIGH FLOW DIVERSION AND PUMPING STATION CONCEPT PLAN

During a high-water event, the drainage structure gates will be closed, and flows will be diverted to a centrally located pump station via a separate drainage system. Both ditches and pipes were

considered for the diversion system, and conceptual plans were developed. A ditch would be a significant feature in this park, and it would remain wet due to the mild slope. Therefore, a piped system was chosen. Precast box culverts are acceptable for the diversion system. The diversion culverts were offset from the levee toe so that the trench excavation line would not affect the levee toe. However, the culvert was located near the toe at the westernmost end to avoid private property impacts (at the request of Fairfax County).

The pumping station was not detail-designed for this study, and an alternatives analysis was not done. Pumping stations can discharge over the levee or through the levee. For the purpose of developing a conceptual cost estimate, it is assumed that the pumping station will receive drainage from multiple pipes, and they discharges through the line of protection by way of a discharge chamber and gravity pipe. To avoid excavating and maintaining a discharge channel through the wetland, the discharge pipe will be elevated a few feet above the wetland. Energy dissipation should be incorporated into the discharge outlet.

The diversion system was designed to begin diverting flows to the pumping station when water in the main outfall channels reaches elevation 3.5. Mean higher high water in Cameron Run is approximately elevation 2.7 feet (USACE, AB Consultants, Inc & RK and K, LLP, 2008), so flow will likely be diverted to the pumping station when a storm event coincides with a significant flow in Cameron Run. To reduce the pumping frequency, gates could be installed on the inlet end of the diversion system and opened only during a significant storm event. If this is done, these gates should be opened at the beginning of the significant storm event due to how quickly rainfall runoff will reach the proposed levee.

As discussed in the Interior residual flooding analysis Section, it was determined that a pumping capacity of 100,000 gpm will be required to maintain a peak ponding elevation of 8 feet during a 100-year rainfall if the pond storage area is excavated to elevation 6.0. The concept design of this pump station includes three pumps that start when the water in the pumping station reaches differing levels:

1st pump is 20,000 gpm with a pump start elevation of 6.0 feet.
2nd pump is 40,000 gpm pump with a pump start elevation of 6.5 feet.
3rd pump is 40,000 gpm pump with a pump start elevation of 7.0 feet.
Total pumping capacity of the pumping station is 100,000 gpm

Figures 7.5 and 7.6 show concept drawings of the pumping station. The multiple chambers shown on Figure 7.5 that will receive flow from the box culvert flow diversion will actually be underground. Figure 7.6 shows the river side of the pumping station, where the discharge pipe will be located. The structure will be made of reinforced concrete and will be approximately 50 feet by 46 feet in size, not including the built out embankment that is required for vehicular access to the overhead door. The pump station will be located adjacent to the levee on the landward side. Access to the station will most likely be along the levee toe. The roof of the pump station is assumed to be constructed with steel framing members that support a light gage metal deck. It was assumed that adequate communications lines would be made available for connection to the pump station. For the concept plan, it was estimated that the sump will be approximately 18 feet deep.

Figure 7.5: Pump Station Concept Drawing (Landward Side)

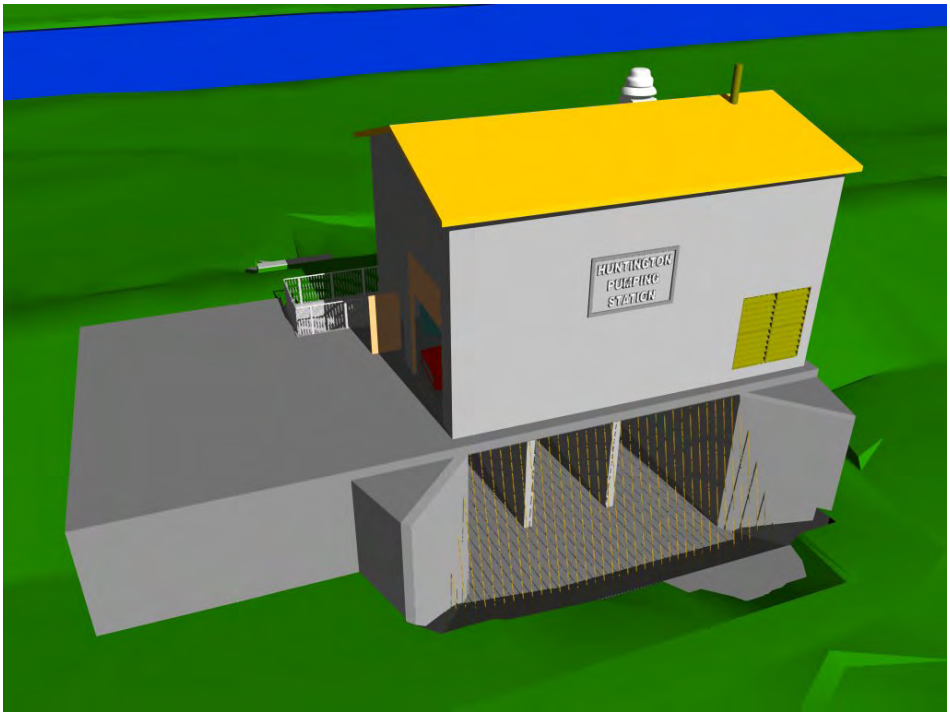
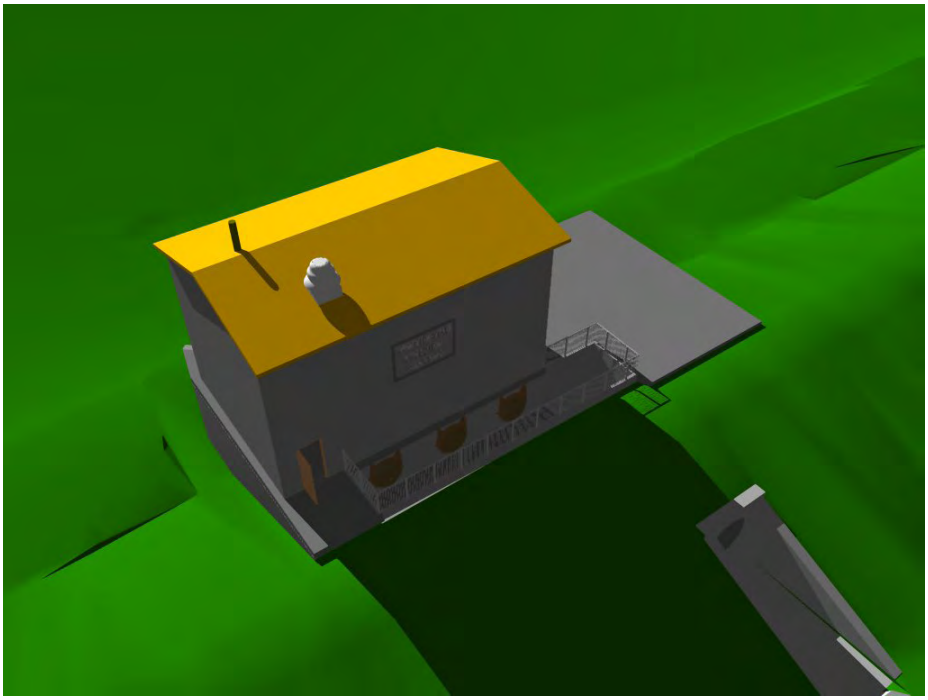


Figure 7.6: Pump Station Concept Drawing (Riverside View)



The cost estimate assumed that the pump station will be constructed on a mat foundation. No geotechnical design was performed for the pump station; however, based on the one boring in the area, it appears that a mat foundation would be suitable. Depending on the final depth of the sump, dewatering may be required to construct the foundation mat. Additional drilling and testing will be required to complete the foundation design for the pump station.

According to the available maps, there is three phase aerial power in the area. If the overhead lines are not large enough for the significant pumping station load, the local power company will have to upgrade their system.

There are two options for back-up power to the pumping station should power be lost during the flood event. One option is to have an emergency generator, and the other is to provide an alternative service feeder.

An alternative service feeder has not been investigated as part of this study. Depending on the distance and cost of running an independent feed it might be more cost effective than the generator option. If the generator is not being used, the automatic transfer switch would be eliminated and quite possibly the motor reduced voltage starters. But a primary selective switch would have to be added to the transformer (or a stand alone primary selective switch) to switch the pumping station between the two incoming feeder sources.

For the purposes of this pump station concept plan and cost estimate, it was assumed that an emergency generator would be part of the design instead of using an alternative service feeder (but this should be investigated during the next phase of study). Having a generator will require solid state reduced voltage starters for 2-500 HP and 1–300 HP motors (controls require time delay between motor starts) as well as an automatic transfer switch, which are quite expensive.

The pumps are assumed to be 460 Volts. There might be a cost savings by having the motor utilization be at a higher voltage (e.g. 4160 Volts).

7.6 RIP RAP ANALYSIS

An analysis to determine the need for erosion protection for the Huntington project was conducted using the with-project Plan 1C and Plan 2C HEC-RAS models for the design event (100-year). The water depth and velocity information was determined at each cross section along the proposed project. The levee side slope of 2.5 horizontal to 1 vertical was used.

The average channel velocities along the proposed project range from 5.4 fps to 6.8 fps for Plan 1C and 5.9 fps to 7.2 fps for Plan 2C. Riprap16 models were developed for the cross sections where the channel velocity was greater than or equal to 5.7 fps. Riprap requirements are not determined solely on velocities, but also the results of the Riprap16 model which takes into account channel side slope, flow depth and the curvature of the river. Results from the Riprap16 analysis showed that no riprap protection was required. Cameron Run is fairly straight and the levee slope is located away from the main channel, this coupled with the reasonable channel velocities contributed to the lack of required riprap protection.

To check how the project affects channel velocities upstream and downstream, a comparison was performed between the channel velocities from the Plan 1C and Plan 2C HEC-RAS model results and the channel velocities from the existing conditions HEC-RAS model results for the 100 year event. There was no increase in channel velocity upstream or downstream of the proposed levee, therefore, no riprap protection is required upstream or downstream of the tie outs.

7.7 REAL ESTATE REQUIREMENTS

Fairfax County, the Fairfax County Park Authority and the Fairfax County Water Authority own the majority of lands between the Huntington subdivision and Cameron Run. A new residential development is in planning stages along what is currently known as Hunting Creek Road on the eastern end of the study site. Private parcels at the western end of the project are located within the flood plain and are, as such, not developable.

The placement of a levee along Cameron Run will be located primarily on existing county land with tie-ins at both ends to private residential or residential development lands. Access would be by public roads Fenwick Drive and Mt. Vernon Drive in the Huntington subdivision and via an access road through the proposed residential development at the eastern end.

Construction of the levee and associated ponding area, and access through the development land on the east will require the acquisition of fee interests in private residential or residential development lands, temporary easements for construction access and staging areas, utility relocations and jurisdictional transfers of portions of county park lands to a common county entity for operation and maintenance, and permanent easements for access. Based on a preliminary analysis, it appears the project will require six (6) transfers of jurisdiction among county entities if they so choose, the acquisition of a minimum six (6) fee parcels plus one (1) easement from two private residential owners and two (2) commercial land development companies. There are public utilities in the area including telephone, electric, water and sewer. The extent of utility relocations has not yet been determined. Pending final plans and specifications, additional land may be required for temporary construction access and or staging areas, although considerable county land is available in the immediate vicinity of the project. Additionally, it is not known at this time if offsite material disposal areas will be required. Additional information regarding the real estate requirements can be found in Appendix E.

7.8 OPERATION AND MAINTENANCE

Operation and maintenance (O&M) of a levee project is critical in order for it to be effective in preventing and/or reducing flood damages. In general, the operation and maintenance of a levee project includes:

- Formation of an organization responsible for the operation and maintenance of the project.
- Inspection of the project every 90 days, and at the beginning of the flood season.
- Preparation of a semi-annual report.

- Maintenance of the easements on both sides of the levee, clear of any vegetation other than grass.
- Maintenance of healthy grass cover on the levee and regular mowing.
- Operation and maintenance of the drainage structure gates every 90 days.
- Maintenance of the drainage structure outlet channels, including regular dredging to keep the gates operating freely.
- Maintenance of a reserve supply of materials for emergencies.
- Operation and maintenance of the pump station.

Regulations pertaining to the operation and maintenance of federal flood control projects is covered in the code of Federal Regulations, Title 33-Navigation and Navigable Waters, Chapter II, Corps of Engineers, Department of the Army, Part 208-Flood Control Regulations, Maintenance and Operation of Flood Control Works (regulation can be found at the end of Appendix G1). It is recommended that Fairfax County follow these regulations even if it is not a federal flood control project. A detailed O&M manual should be prepared based on the above regulations, as well as the guidance found in the Corps of Engineers ER 1130-2-530.

7.9 PROJECT CONSTRUCTION COST ESTIMATE

A project cost estimate was developed for Plan 2c, which includes a levee with a top of protection elevation of 19.6 feet at the upstream end, the excavation of part of the park area down to elevation 6 feet to allow for a ponding area during flood events, and a 100,000 gpm capacity pumping station that will maintain a maximum interior ponding water surface elevation of 8 feet during a 100-year rainfall. The total project cost is estimated to be $20,230,000. The project cost estimate is based on 1 October 2008 costs, escalated into the future. The Preconstruction Engineering and Design (PED) costs are escalated to the year FY10 and construction is estimated to take place between FY11 and FY13, so the construction costs were escalated to a midpoint of FY12. The construction duration is estimated to be approximately 2 years.

The levee and pump station costs are separated into two line items since the levee has been designed to a 65% design level and the pump station has only been designed to a concept level. The pump station cost includes the pump station building and all accompanying features of the interior drainage system such as the flow diversion and the excavated pond area. A 25% contingency has been used for the levee cost estimate and a 30% contingency has been used for the pump station cost estimate. A summary of the cost estimate is shown in Table 7.5; the detailed cost estimate information can be found in Appendix G6.

The cost to operate and maintain the project each year could vary significantly based on the age and condition of the pump station and drainage structures, and the severity of floods and their impact on the levee and channels. For cost estimating purposes, the operation and maintenance cost is estimated to be $150,000 annually.

Table 7.5: Summary of Project Costs (Plan 2c)

Project Feature	Cost (escalated to FY 2012 $)
Levee	$6,795,000
Pump Station and Features (Flow Diversion)[1]	$9,900,000
Total Construction	**$16,695,000**
LERRs[2]	$75,000
Preconstruction Engineering and Design (PED)[3]	$1,475,000
Construction Management[4]	$1,985,000
Total Project Cost	**$20,230,000**
Annual Operation and Maintenance Cost	**$150,000**

[1]Pump capacity is 100,000 gpm to maintain maximum pond elevation of 8 feet (with excavation of park area)
[2]LERRs stands for lands, easements, rights of way, and roadway requirements
[3]PED is to complete the final design and prepare the plans and specifications, escalated to FY 2010 costs
[4]Construction Management is estimated to be 10% of the project construction cost (plus contingency and escalation)

7.10 ECONOMIC ANALYSIS OF FINAL PLAN

A final economic analysis was conducted on the selected plan. The expected annual benefits of Plan 2c are $540,000 and the expected annual costs are $1,230,000 for a 50-year project period. The benefit-to-cost ratio is 0.4. The project has a 99% chance of containing the flooding event with a 1% chance of occurrence. This project is expected to prevent nearly all of the annual flooding damage at an investment or first cost of $20.2 million (or $20.9 million including interest during construction). See Appendix D for further details regarding the economic analysis.

7.11 FLOOD RISK WITH THE LEVEE PROJECT

The Huntington levee project has been designed to prevent damages to the Huntington houses and community center from a 1% annual chance flood. According to the risk and uncertainty analysis, there is a 99% chance that the levee will not be overtopped during a 1% annual chance event. However, even with the levee, the community is not without risk. If a larger flood event should occur, say the 0.4% annual chance flood (250-yr) or 0.2% annual chance flood (500 yr), the levee could be overtopped and the community could experience flooding. Also, if the project is not operated and maintained properly, as described in Section 7.8, the levee system could fail during any flood event and the community could incur damages. It is critical that the levee and pump station system be operated and maintained as necessary to minimize the risk to the community.

It is important to note that although the levee is designed to prevent Cameron Run from flooding the community during a 1% annual chance, there will still be water ponding on the landward side of the levee from the interior stormwater runoff during a high water event. The pump station will pump this water over the levee to Cameron Run, however, depending on the amount of rainfall, there could be standing water throughout the community, particularly in the park area and along the roads. Although the pump station is designed to maintain a maximum pond elevation of 8 feet for a 100-year rainfall, which should prevent damages to houses and the community center, this residual flooding could cause some damages to the community.