County of Cape May

Draft Hydrology Report

Cox Hall Creek Basin

June 2008

Lower Township, New Jersey

HMM 243271

Table of Contents

1.0 Introduction

The County of Cape May has engaged the services of the team of Hatch Mott MacDonald (HMM) and The Lomax Consulting Group, LLC (TLCG) to undertake investigations preliminary to the environmental restoration of the Cox Hall Creek wetland area adjacent to the Delaware River. Hatch Mott MacDonald is providing hydrology and engineering capability while The Lomax Consulting Group, LLC provides environmental permitting capability and site and project specific experience. This team brings a multi-disciplinary approach to managing a project of this magnitude and complexity. It is recognized that critically important components of this project include the watershed hydrology analysis, engineering of channels and structures, state and federal environmental regulatory agency permitting and participation in public interest and county partner participation.

The current investigations as described in this report provide for modeling the hydrology of the Cox Hall Creek watershed and design of a culvert at Clubhouse Road as an initial phase of project implementation. Work required for restoring tidal flow to the wetland area and undertaking other project improvements would be the subjects of future phases.

1.1 Location

The Cox Hall Creek wetlands basin is located adjacent to the Delaware Bay in Lower Township, Cape May County, New Jersey (See Figure 1 – Appendix B). Cox Hall Creek enters the wetland basin from the northeast and Mickels Run enters the basin from the east. These two streams cross Bay Shore Road (County Highway 603) before entering the Cox Hall Creek Wetland Project area. The limited culvert capacity at this road limits the flow into the wetland area until such time as the road is overtopped by storm flow. Surface water also enters the wetland area from the surrounding land area which includes a substantially developed residential area, particularly to the south of the wetland. The tributary area to the north of the wetland includes a former golf course which is now preserved as a wildlife management area.

Clubhouse Road crosses the creek channel leading from the main wetland area to the pump station and outfall to the Delaware Bay. The outfall culvert passes through a dune which extends north and south along the bay. Shore Drive runs parallel to and on the land side of the dune, but is interrupted at the Cox Hall Creek outfall by a strip of private land and the outfall alignment. Therefore there is no existing roadway across the Cox Hall Creek outlet to the bay. However, there is public access by foot along the shore of the Delaware Bay and from Shore Drive to the immediate north of the outfall alignment.

1.2 Background

The *Cox Hall Creek Wetland Restoration Scenarios and Feasibility Study, Volume 3 – Preferred Concept*, dated February 2004, describes the overall concepts for the restoration of the Cox Hall Creek wetland adjacent to the Delaware Bay. Previously, perhaps initially to utilize the land for non-tidal purposes and later to address tidal flooding problems affecting properties adjacent to the wetlands, a barrier was constructed on the land side of the dune at the outfall to restrict tidal flow into the wetlands. These flood control facilities, currently including tide gates and a pumping station, eliminated the periodic tidal inundation of the Cox Hall Creek wetlands and resulted in transformation of the wetland vegetation to a predominantly freshwater regime. The propagation of Phragmites australis (common reedgrass) has resulted in certain problems, including creation of a fire hazard and clogging of the internal channels that drain stormwater.

The overall goal of the complete project (including elements beyond the scope of the current authorized investigations) is to restore the quality and enhance the natural functioning of the Cox Hall Creek wetlands, to enhance both the natural and human environments. Detailed surface water and tidal hydrology analyses are required to appropriately design the project components and establish operation criteria for stormwater and tidal flow conditions. A tidal inundation hydrology/hydraulics analysis is required to appropriately size the facilities that will allow tidal flow into and out of the wetland basin, with consideration of the tidal cycle and the volume of the wetland floodplain up to the elevation of the desired daily inundation. A stormwater analysis is required to appropriately size facilities to accommodate storm flows, with due consideration of the flood storage capacity of the wetland basin. A balance is required between tidal and freshwater flow to restore tidal flow and deter Phragmites growth, yet not aggravate stormwater flooding conditions and not adversely affect the population of swamp pink that has been identified along a reach of Cox Hall Creek a short distance upstream of the wetland restoration area.

1.3 Project Concepts

As identified in the *Cox Hall Creek Wetland Restoration Scenarios and Feasibility Study, Volume 3 – Preferred Concept*, dated February 2004, the preferred wetlands restoration concept is <u>Scenario 6. Freshwater and Tidewater Ecosystem Complex.</u>

As described in the above referenced 2004 report, Scenario 6 provides for return of tidal flow to the wetlands in a controlled manner, with protection of the upstream flora and fauna of the Cox Hall Creek and Mickels Run wetlands by the installation of small berms and water control structures to maintain the upstream hydrology during low flow periods and tidal ebbs. Self-regulating tide gates were considered to allow tidal inundation to a controlled elevation twice daily, but preclude excessive tidal inundation that would adversely affect areas to remain as freshwater regimes and developed areas. Band ditches were included along the perimeter of the wetland to be returned to the salt-water regime to promote tidal circulation and inundation of the marsh.

The principal components of Scenario 6, as presented in the 2004 report, included:

1. Installation of self-regulating tide gates into a reconstructed culvert system through the beach and dune complex to reconnect the wetlands to the bay;
2. Redesign and replacement of the pumping station;
3. Reconstruction of the channel between the pumping station and Clubhouse Drive;
4. Redesign and replacement of the culverts under Clubhouse Drive;

5. Construction of band ditches along the perimeter of the wetland restoration area, with weirs to prevent complete draining of the ditches at low tide;
6. Construction of low berms and water control structures (and possibly fish ladders) at the proposed interface of tidal and freshwater regimes on the Cox Hall Creek and Mickels Run.
7. Stormwater quality facilities at the numerous storm sewer outfalls surrounding the affected wetland area.

The details of the Scenario 6 concept were reviewed with representatives of appropriate federal and state agencies at a joint meeting on April 9, 2008. Federal and State agencies represented at the meeting included the US Army Corps of Engineers, Philadelphia District; US Environmental Protection Agency (EPA); US Fish and Wildlife Service (USFWS); NOAA Fisheries; and the New Jersey Department of Environmental Protection (NJDEP) Division of Land Use Regulation and Division of Fish and Wildlife (DFW). The consensus of the opinions of the fisheries and wildlife agency personnel was that the project should not include band ditches, internal flow control structures, dams, weirs or fish ladders. They were of the opinion that the tidal flow could be adequately controlled by use of appropriate water control structures at the outlet to the Delaware Bay. They considered the band ditches unnecessary and undesirable as they preferred that the ebb and flow of tide be concentrated in the interior wetland ditches that would develop further based upon management of the tidal flow. Also, excavation for the band ditches would disturb the natural sediment layer around the perimeter which may be helping to reduce salt water intrusion into adjacent soils. DFW personnel stated that water control structures are being used to successfully manage restoration of tidal flow to wetlands in other areas under DFW management, and could be operated to limit the extent of upstream tidal flow so as to preserve the area of swamp pink population along Cox Hall Creek.

On May 29, 2008, representatives of Cape May County, Hatch Mott MacDonald and The Lomax Consulting Group accompanied representatives of NJDEP- DFW on a field visit to observe the operation of water control structures in operation at the Heislerville Wildlife Management Area, located adjacent to the Maurice River estuary in Maurice River Township, Cumberland County, New Jersey. The water control structures consisted of cylindrical chambers with aluminum stop logs and flow control flaps which could be arranged in multiple combinations, with various sized flaps, to control the flow of water in and out of the wetland basin. The wetland areas being managed were of significant size; however there did not appear to be a need to control the outflow of stormwater at the site since there were no adjacent private homes in the floodplain that would be affected by the facility operation. However, for facilities requiring accommodation of high rates of flow, multiple chambers could be employed. Further consideration will be given to use of this type of water control structure for the Cox Hall Creek project.

As a result of the recommendations at the April 9, 2008 joint agency meeting, the revised principal components of Scenario 6, as now under consideration include:

1. Installation of appropriate water control structures into a reconstructed culvert system through the beach and dune complex to reconnect the wetlands to the bay;
2. Redesign and replacement of the pumping station;
3. Reconstruction of the channel between the pumping station and Clubhouse Drive;
4. Redesign and replacement of the culverts under Clubhouse Drive;
5. Stormwater quality facilities at the numerous storm sewer outfalls surrounding the affected wetland area.

This revised list excludes the internal band ditches, weirs, and fish ladders, and includes consideration of the type of water control structures recommended by the NJDEP-DFW.

2.0 Project Description and Analyses

2.1 General

The overarching approach has entailed early consultation with the New Jersey Department of Environmental Protection (NJDEP), U.S. Army Corps of Engineers (USACOE), U.S. Environmental Protection Agency (USEPA), U.S. Fish and Wildlife Service (USFWS), and National Marine Fisheries (NOAA) to ensure that analysis, design and permitting is consistent with their expectations. This consultation has been initiated as described above.

The following design criteria that were utilized in the first phase of this project will be reaffirmed to ensure that the Cox Hall Creek hydrology management design are consistent with defined watershed priorities, the project goals and associated funding sources. These criteria include:

- Stormwater management
- Protection of local residents against flooding
- Protection of potable water supplies drawn from the Holly Beach water-bearing zone
- Ecological restoration of the Cox Hall Creek wetlands area
- Water quality enhancement
- Pest control, with special emphasis on mosquitoes and *Phragmites*, including wild fire suppression

The hydrologic analysis has been undertaken as described in this Report. Upon completion of the technical review of this Report by the County and its partners, the draft Report will be presented to the Cox Hall Creek Focus Group for their input.

Based upon the hydrology findings and the technical and public interest input; the design for the Clubhouse Road culvert and stormwater management (pollution control) structures will be completed. The final Project Report will be presented for technical and public review. This Report will include estimates of costs and potential funding sources. During the second meeting with the Cox Hall Creek Focus Group, the final plans will be presented along with the permitting review process.

The applications for permits and approvals will be submitted to the NJDEP for CAFRA, Waterfront Development, Coastal Wetlands, Freshwater Wetlands and Water Quality Certification. Concurrently an application will be submitted to the USACOE for any components on the salt water side of the existing tide gates. The Corps has advised that the Corps jurisdiction ends at the functioning tide gates and does not extend upstream of that location until tidal flow is restored, either intentionally or by failure to maintain the tide gates as functional. Those permit applications will be tracked through the regulatory review process to final decisions.

2.2 Scope of Current Investigations

The current Scope of Work essentially includes the undertaking of a hydrologic analysis of the Cox Hall Creek watershed, and preparation of a preliminary design of stormwater outfall filters and a culvert crossing of Clubhouse Road. This Repoprt addresses the stormwater and tidal hydrologic analyses and preliminary sizing of the Clubhouse Road culvert. Following final selection of the size and design of the culvert and the stormwater outfall water quality facilities, HMM in coordination with The Lomax Consulting Group (TLCG) will prepare applications for applicable NJDEP Freshwater Wetlands, Coastal Wetlands, Water Quality Certification, Waterfront Development and CAFRA permits. For this initial phase of construction, an application will not need to be submitted to the USACOE, since the Clubhouse Road culvert and upstream stormwater outfalls are not currently within Corps jurisdiction (as long as the existing tide gates are maintained and functional). HMM and TLCG will also prepare a construction cost estimate and offer references for sources of potential funding for the recommended improvements.

The following work efforts are not included in the current scope of project, but are recognized as future project activities:

- Design of the replacement pump station
- Design of the replacement outfall pipe into the Delaware Bay
- Design of the self-regulating tide gates or other water control structures
- Development of Construction Bid Documents
- Construction Coordination and Inspection
- Municipal, County and Soil Erosion Permitting
- Tidelands Grants, Licenses or Leases
- Wetland Delineations or Associated Surveys
- Sub-Surface Investigations
- Application Fees

2.3 Methodology

Hydrologic Analyses

There are two major components to the hydrologic analyses of the Cox Hall Creek watershed and wetlands under Scenario 6 (as modified by the above recommendations). The first is the analysis of the stormwater flows into the wetlands and through the outfall facilities, to provide criteria for the sizing of channels, culverts and the pumping station, and for the design of the water control structures. While the design of some of these project elements is not included under the current project efforts, the hydrologic analyses will provide criteria for the future design of such features. The second hydrologic analysis required is for the tidal flow into and out of the wetland basin through self-regulating tide gates or other water control structures, the culvert under Clubhouse Road, and the channel and outfall pipe between Clubhouse Road and the Delaware Bay. The results of both the stormwater and the tidal hydrologic analyses must be considered in evaluation of the controlling or critical condition for sizing the culvert to achieve the project objectives of restoring tidal flow to the wetland but not aggravating storm flooding in the adjacent residential area. The size of the Clubhouse Road culvert should be selected so that it does not become the controlling structure affecting tidal and storm flow, but rather that it permits flexibility in the selection and operation of the outfall facilities that will be designed in the future.

Stormwater Hydrologic Analysis

As indicated in the above referenced report, *Volume 3*, a portion of the developed area adjacent to the Cox Hall Creek wetlands will be inundated by floodwaters that rise above approximately elevation 3 to 4 feet NAVD. As stormwater enters the wetland basin, the water level in the basin will rise, depending upon the rate at which water enters and the capacity of the outfall system, including channels and culverts as influenced by the varying tide elevation, and the pumping station. Analyses of this dynamic condition necessitates the development and routing of a storm water hydrograph, representing the combined runoff from the upstream tributary drainage area.

An essential component of the analysis of this balance of inflow, storage and outflow capacity is the inflow characteristics of the tributary watershed of approximately 2,330 acres. Appropriate analyses of this southern New Jersey watershed has been based on utilization of the DelMarVa unit hydrograph. This unit hydrograph has a slower rise to peak, smaller unit peak and longer recession limb than some other unit hydrographs commonly used in hydrologic models that are more appropriate for terrain found in the northern part of the state. The DelMarVa unit hydrograph more closely represents hydrologic conditions in flat terrain areas of southern New Jersey, with significant local depressions and soil conditions that result in slower movement of surface runoff.

To accomplish the analysis of inflow, storage and outflow, a dynamic hydraulic modeling program has been employed based upon the XP SWMM computer program.

XP SWMM is a commercial version of the EPA SWMM program, is generally accepted by permitting agencies, and has the capabilities required to simulate the anticipate project components. The hydraulic capabilities include reverse flow in pipes, automatic gates and time varying boundary conditions, which are necessary for modeling inflow and outflow through the culvert during normal dry weather operation. It also includes hydrologic analysis tools to model the interaction between the storm inflow, tidal basin and pumping station operation. The necessary hydrologic parameters such as initial abstraction, flow routing and unit hydrograph shape factors can all be input to reflect the drainage area characteristics and the DelMarVa unit hydrograph.

HMM has developed a hydrology model for the Cox Hall Creek basin based on the above-described XP SWMM computer program, incorporating the DelMarVa unit hydrograph representative of runoff characteristics in southern New Jersey, and surface runoff coefficients reflecting the soil and land use conditions in the subwatersheds. This model includes the watershed hydrology, simulation of the tide levels, and accounts for the project hydraulic features including the culverts, pumping station, tide gates and channels. Model scenarios have been developed to represent existing conditions and alternative proposed conditions, which will facilitate final determination of the appropriate sizes for the channels and culverts and the pumping station required capacity for a selected range of storms, such as the 10, 50 and 100-year storms. Comparison of the facility sizes and capacities required for this range of storms facilitates understanding of the requirements to achieve a designated level of protection and the selection of a project design storm and corresponding culvert size.

Tidal Inundation Hydrologic/Hydraulic Analyses

Periodic tidal inundation of the Cox Hall Creek wetland basin is needed to change the wetland to an estuary regime and provide natural deterrent to the Phragmites. Factors that will affect the depth of inundation under proposed conditions, with self-regulating tide gates or other flow control structures, include the volume within the wetland area to be inundated by the tidal flow, the hydraulic capacity of the tide gates and other inlet facilities under varying tidal levels, and the time-variation of the tidal elevation. If the hydraulic capacity of the inlet facilities is too small, the tidal inflow may not reach the desired elevation in the project wetland area. Also, with insufficient hydraulic capacity, the wetland may not sufficiently drain during the ebb tide before the tide shifts and starts rising again. This could result in inadequate flushing of the wetland basin and possibly result in sediment deposition in the channels.

The tide flow in and out of the wetland basin has been simulated with the XP SWWM program described above. HMM has modified the model to represent the tidal inflow and outflow under non-rainfall conditions, which will facilitate determination of appropriate sizes for facilities. In this analysis, consideration has been given to potential base flow in the tributary streams, which will dilute the brackish water entering from the bay. The sizes required for desired tidal inundation can be compared to the sizes required for stormwater management, and the controlling condition can be identified, thereby facilitating determination of design criteria for the project facilities.

If the wetlands are allowed to fill to even normal high tide elevations prior to a significant rainfall event, the backwater from the tidal inundation could result in flooding of portions of the developed areas. This mode of operation would necessitate a significantly increased capacity of the pumping station and outfall facilities in order to prevent upstream flooding. However, management of the wetland area tide structures to keep the water to a low level prior to a storm would provide a greater storage volume for detention of the stormwater in the wetlands and reduce the capacity required for the pumping station and outfall facilities. HMM will include discussion of these concepts and other facility management aspects in the final project report.

3.0 Hydrologic Analyses

3.1 Modeling Approach

The Cox Hall Creek drainage area and wetlands were modeled using XP SWMM a commercial version of EPA SWMM. SWMM was initially developed to model urban storm systems and models one dimensional gradually varied flow. It consists of three modes hydraulics, sanitary and runoff. The simulation Cox Hall Creek project will make use of the hydraulics and runoff modes. The model consists of a series of links and nodes. In the hydraulics mode links represent pipes, pumps, gates etc., and nodes represent manholes, storage areas and outfalls. In the runoff mode, nodes represent drainage areas. The output of the runoff mode can be become a flow input to the hydraulics mode, however the two are run independently. In XP SWMM the user is able to manipulate the unit hydrograph peak rate factor (referred to as the shape factor in XP SWMM) allowing the DelMarVa unit hydrograph to be incorporated. This makes is suitable choice for modeling the tidal flushing and storm response of the wetlands, in that the existing and proposed hydraulic and hydrologic features can all be incorporated into the model.

3.2 Rainfall and Tide Data

Based on the project location the design storms were developed based on the SCS Type III rainfall distribution. The total rainfall depths for the 24 hour rain events were obtain from Point Precipitation Frequency Estimates, for Cape May, from NOAA Atlas 14, see Appendix A.

TABLE 1

NOAA Atlas 14, 24 Hour Rainfall Depths

Return Period	24-Hour Rainfall Depth (in)
1-Year	2.48
2-Year	3.02
5-Year	3.93
10-Year	4.71
25-Year	5.9
50-Year	6.94
100-Year	8.11
500-Year	11.43

Tide data was obtained from the NOAA Cape May tide gage station, Station ID 8536110 (see Appendix A). Based on the Mean Higher-High Water (MHHW), Mean High Water (MHS), Mean Low Water (MLW) and Mean Lower-Low Water (MLLW) and the tidal period, XP SWMM generates a synthetic tide cycle. This synthetic tide cycle is used as the water level downstream of the outfall pipe. The tide elevation serves as the downstream boundary condition for the model. This is because the model responds to the tide elevation, but cannot affect the tide elevation i.e. flows from Cox Hall Creek will have not perceptible impact on the Delaware Bay. The tide gage also provides time series data for historic events at the gage. It was noted that the tides during April 6-9, 2008 were higher than any previous tides in 2008. The times series data from the gage was used to provide an additional "real world" tide analysis.

TABLE 2

NOAA Cape May Tide Gage Data

Station ID 8536110

Tide	Elevation (ft NAVD88)
MHHW	2.43
MHW	1.99
MLW	-2.86
MLLW	-3.02

3.3 Hydrology

As shown on Figure 2 (see Appendix B), the Cox Hall Creek watershed area is approximately 2330 acres and was delineated using the HUC-14 sub-watersheds as mapped by the NJDEP and the USGS. The area drains in a westerly direction towards the Cox Hall Creek wetland basin area before ultimately draining into the Delaware Bay via the Clubhouse Road culvert, tide gates at the existing pump station and the outfall culvert under the dune along the bay.

For the purpose of the stormwater component of the hydrologic analyses, the Cox Hall Creek drainage area was subdivided into ten (10) major sub-areas with the downstream limit set at the upstream end of the dune along the Delaware Bay. The approximate boundaries for each sub-area were based on computer generated topography from detailed survey data performed by Hatch Mott MacDonald, existing storm drain plans and configurations, and USGS topographic mapping, with consideration of flow paths and storage areas indicated by soils maps and aerial photographs. The hydrologic parameters such as the Drainage Area, Runoff Curve Numbers and Times of Concentration were estimated for each sub-area utilizing the available information from aerial photography, Cape May County soil survey, existing storm drain plans, USGS topographic mapping, etc. Table 3 below shows a detailed breakdown of each delineated sub-area used in the hydrologic analysis for the Cox Hall Creek watershed.

TABLE 3

Hydrologic Parameters for Cox Hall Creek watershed sub-areas

Location	Total Area (Acres)	TC (min)	CN
Area 1	102	45	66
Area 2	187	36	72
Area 3A	152	29	70
Area 3B	214	292	67
Area 4	426	95	63
Area 5	247	164	68
Area 6	314	125	62
Area 7	273	163	67
Area 8	382	100	73
Area 9	32	37	64

All of the Sub areas, with the exception of Sub-area 9, drain to Sub-area 8 which contains the main Cox Hall Creek wetland basin and floodplain area. Sub-areas 1, 2, and 3A to the south are substantially developed residential areas that drain to the wetland basin via storm sewer infrastructure systems. Sub-area 3B, also to the south, is primarily a residential area with several pockets of wooded or wetland areas. This area drains through Sub-area 4 and ultimately into the main wetland basin of Sub-area 8. Sub-areas 4, 5 and 6 to the east and northeast of the wetland basin are a mix of residential and/or light industrial areas and wooded areas with significant depressions that collect and attenuate stormwater runoff. Flows from Sub-areas 4 and 5 must cross

Bay Shore Road (County Highway 603) which has a limited culvert capacity before enter the Cox Hall Creek wetland basin. The flows from these two sub-areas at this road crossing were routed in the XP SWMM hydrologic model which reduced the peak flows generated from these areas into the main wetland basin and floodplain area. Sub-area 7 to the north is a substantially developed residential area with a former golf course which is now preserved as a wildlife management area. The wildlife management area has several water bodies throughout the area that would reduce the peak flow of stormwater runoff into the main wetland basin. Sub-area 9 is to the west and downstream of the wetland basin. This area was modeled with a routing to account for the small storage area within the channel between the Clubhouse Road culvert and the outfall culvert under the dune along the bay. For Sub-area 8, a stage-storage relation was developed for the wetland basin and flood plain area of the Cox Hall Creek project area. This stage-storage relation of the wetland basin and the watershed basin model were utilized to analyze the storm and tidal flow and to estimate the water surface elevations in the basin under existing and proposed conditions scenarios for the Cox Hall Creek wetland restoration project.

3.4 Existing Conditions Model

The existing conditions model of Cox Hall Creek was developed to serve as the starting point for developing proposed conditions models, and as a point of comparison when evaluating their performance. The hydrology data entered in the model's runoff mode was developed as described above. The DelMarVa unit hydrograph was specified by entering 285 for the peak rate factor, as specified by the New Jersey Department of Agriculture and State Soil Conservation Committee, see Appendix A. The existing storage area, drainage features, cross culverts and tide gates entered into the hydraulics mode were based upon field survey data. The outfall pipe and pump station configuration were taken from design drawings. The pumps were not modeled as part of the existing condition as they are not functioning. The Clubhouse Road Culvert (the culvert) and outfall pipe, including the pump station structure with tide gates, are the two hydraulic controls on the wetlands.

The existing conditions model was run for a series of storms ranging from the 1-year storm to the 500-year storm. The storm events were modeled against two conditions, the diurnal tide (a tide peaking twice a day) and storm surge condition. When the storms were run against the diurnal tide cycle, tidal flow was prevented from entering the wetlands by a tide gate on the outfall pipe. During the storm surge condition, no water was allowed to flow through the outfall pipe (simulating the effect of prolonged significant tidal elevations below the elevation of the top of the dune, with all stormwater going into storage in the wetland basin). The results of these two runs can be seen in Figure 3, Appendix B. The only gravity outlet at the pump station is a 24" diameter tide gate; therefore the outflow under existing conditions is relatively low when compared to the runoff flowing into the wetlands. Thus the difference between the storms run against diurnal tide and those run against the storm surge condition is relatively small. In general, under existing conditions, the wetlands fill during a storm and then gradually drain once the storm has passed.

3.5 Proposed Conditions Screening Model

The purpose of this screening analysis is to determine the appropriate size of the culvert under Clubhouse Road. The two performance criteria are to allow tidal flushing of the wetlands and to minimize flooding where possible. The performance of the overall system can not be evaluated until the outfall facilities connecting the pump station to the Delaware Bay are selected. However, those facilities will be developed some time in the future, therefore it was necessary to design the Clubhouse Road culvert to perform against the potential range of downstream facilities. Minimizing the hydraulic losses through the culvert when paired with a range of downstream facilities prevents the culvert from being the "weak link" to achieving tidal flushing and flood control.

To evaluate alternative sizes for the Clubhouse Road culvert, a screening analysis was developed to simulate the hydraulic losses though the culvert for a range of sizes. The actual culvert to be selected will likely consist of one or two culverts under the road. However, to facilitate analyses of a range of sizes, multiple culverts identical to the Clubhouse Road culvert were inserted in to the model; these culverts matched the existing culvert in size (4-foot diameter), length and inverts. The facilities going out to the Delaware Bay were replaced with an equal number of 4-foot diameter pipes, using the same length and inverts as the exiting outfall pipe. The existing inverts were maintained to ensure that the proposed facilities would be able to meet the existing grade restrictions for outfall to the bay. Based on the results of the screening analysis, two box culverts were selected for further consideration under the alternatives analysis.

The results of the screening analysis were evaluated on the basis of the two performance criteria; tidal flushing and flooding potential. The tidal flushing was evaluated on the basis of diminishing returns. If the wetlands were fully connected to the bay they would maintain the same water level as the bay. However, the wetlands are connected through a series of pipes and culverts which introduce friction losses and a lag in the time of rise and fall of the water in the wetland basin, which will neither rise to the level of MHHW, nor fall to the level of MLLW during a normal diurnal tide cycle. Water can only flow into the wetlands when the water surface in the bay is higher than the water surface in the wetlands. The greater the difference in water elevation between the bay and the wetlands the faster water will flow through the connecting culverts and control facilities. The time-duration that the water level in the bay is higher than the water level in the wetlands grows shorter with higher levels in the wetlands. The wetlands gradually slope so that as the water level rises the water surface area increases and the volume of stored water increases; the higher the water level in the wetlands, the more water required to raise the water surface an additional increment. Thus as more water is required to raise the water level in the wetlands there is less time for that water to flow in and less difference in water surfaces to push that water in. That means that the wetlands cannot reach the same peak elevation as the bay and that each incremental increase in water surface in the wetlands will become more expensive (i.e require a larger culvert capacity) to achieve. This can be seen in the results presented in Figure 4, Appendix B. The results of the April 6-9, 2008 tidal analysis are also presented in Figure 4. This figure provides an indication of the attenuation in tidal peak in the Cox Hall Creek wetland basin associated with various sized culvert replacements at Clubhouse Road.

For evaluation of alternative culvert sizes under storm conditions, the XP SWMM model was run for a range of sizes and a range of design storms. Again the results of the analysis presented the potential benefits of the Clubhouse Road culvert replacement with outfall facilities of similar hydraulic capacity. Much will depend on the future pump station and outfall design. The initial construction under this project will provide a culvert at Clubhouse Road that can function in conjunction with a wide range of future facilities at the outfall to the bay. Figure 5, Appendix B shows the results of the screening analysis under storm scenerios. To aid in the evaluation of the alternatives Figure 6 – Appendix B was developed to show the number of houses impacted by various water levels. This figure was developed based on topographic map contours indicating ground elevations at the houses and does not consider low opening or first floor elevations. Thus it is indicative of the number of houses where water would reach the foundation of the house, and does not indicate whether water would enter the house. However, it can serve as an indication of the potential additional benefits of reducing a given flood elevation.

3.6 Summary of Results of Screening Analyses

The results of the XP SWMM analysis show the performance of the alternative facilities. The incremental tidal flushing increase from enlarging or adding additional facilities diminishes with each increment. This can be seen in Figure 4, for example when the number of screening culverts were doubled from six to twelve the increase in tidal flushing was only 0.1'. While the increase in tidal flushing from increasing the screening culverts from four to six is 0.2', twice as much impact as adding an additional six barrels.

When considering the results of storm analysis, incremental reduction in flood elevations due to incremental increases in facility capacity does not diminish as rapidly as it does for tidal flushing, (see Figure 5). However, the benefits of decreases in flood elevation are diminished by the decrease in the number of additional homes benefited. For example, under the 100-year storm analyses, when the number of screening culverts is doubled from six to twelve only about 2 or 3 additional houses are removed from the floodplain, and only about 4 others remain impacted. Therefore, the increase from 6 to 12 screening culverts potentially benefits only 6 houses to some degree. To further evaluate the benefits of the larger culvert capacity, detailed survey information would be required at each of these 6 houses to determine the actual ground elevation adjacent to the house and the elevation of the point of entry of water into each house.

4.0 Alternatives for Clubhouse Road Culvert

Obviously placing numerous identical culverts is not practical. The multi-barrell screening analysis was only intended to focus the alternatives modeling efforts. Based on the results of the screening analysis two culvert sizes were selected for evaluation, a single barrel 20'Wx8'H box culvert and a double barrel 20'Wx8'H box (total width 40'). The crown of the culvert would be set at elevation 3.5', just above MHHW and the bottom would be buried to maintain the existing invert. For evaluation purposes the outlet pipe through the dunes was modeled with the same dimensions as the considered road culvert. This is probably larger than any potential future outlet pipe so it will ensure that the culvert does not inhibit the performance of the final system.

4.1 Tidal Flow

The goal of tidal flushing is to inundate wetlands to the highest practical elevation without proposing excessive facilities or increasing the potential for flooding at the houses. Based on the screenings analysis providing facilities in excess of six to eight 4-foot diameter screening culverts produces no additional benefit for tidal flushing. The 20'Wx8'H culvert was estimated to be equivalent to these facilities. Figure 7 demonstrates that one 20'Wx8'H box culvert is about as effective in accomplishing tidal flushing as six to eight screening culverts. The tidal inundation of the Cox Hall Creek Wetlands is a dynamic process and has a time component as well as a peak depth. Figure 8 illustrates the duration of tidal inundation for the two alternative Clubhouse Road culverts.

4.2 Storm Flow

The goal of the storm analysis was to lower the flood elevations as much as practical and to protect as many houses as practical. Because the screening analysis showed continuing benefit from increasing facilities, both a single barrel and a double barrel 20'Wx8'H culvert were tested. Figure 7 demonstrates that one 20'Wx8'H box culvert is about as effective in accomplishing flood control as eight screening culverts, the 100-year flood is reduced to elevation 3.5'. Two 20'Wx8'H culverts result in a flood reduction well beyond that of twelve screening culverts, reducing the 100-year flood to elevation 3'.

4.3 Combination Tidal and Storm Flow

Should a rainfall event occur in conjunction with an extreme tide or storm surge preventing flow from Cox Hall Creek to the bay, the culvert facilities will have no impact the depth of flooding, with no change from the existing condition since all such stormwater would be stored in the wetland basin. In this situation the outlet would be blocked and the flood volume stored in the wetlands, thus only a change in storage volume or the addition of pumping capacity would result in a change in flood elevation in the basin. The future pump station would have an impact on this scenario, but the design of the pump station is beyond the scope of this analysis.

4.4 Culvert Selection

Based upon the results of the above described analyses, a single cell, 20' wide by 8' high box culvert at Clubhouse Road (with outfall facilities of equivalent capacity) would be equivalent to approximately an 8 to 9 barrel screening culvert. As indicated on Figure 4, the single 20' box culvert would attenuate a normal diurnal tide with a peak in the bay at El. 2.4' to a peak of El. 2.1' in the Cox Hall Creek wetland basin, and a storm tide peaking at El. 4.4' in the bay would be attenuated to El. 3.8' in the basin. Under storm conditions based upon flows in Cox Hall Creek and tributaries (not Delaware Bay flood levels), with a normal diurnal tidal cycle, the single barrel 20' wide culvert would result in a 100-year flood at approximately elevation 3.6', which is significantly lower than the corresponding existing 100-year storm flood level estimated at approximately elevation 4.9' (see Figure 3). As indicated on Figure 6, approximately 33 houses would benefit to some degree from this reduction in local flood elevations, leaving approximately 4 of these houses with surrounding grade below the reduced flood elevation. Detailed survey information would be required at each house to further evaluate the potential benefits that might be achieved by the 1.3' reduction in 100-year flood elevation and by smaller reductions in lesser floods, as well as to determine whether the 4 remaining houses are actually adversely affected by the reduced 100-year flood elevation.

The double 20' wide culvert seems to offer only small additional benefits for tidal and storm flow. The attenuation of the normal diurnal tide (El. 2.4' in the bay) is slightly less (see Figure 4), allowing the tide to rise to El. 2.3' in the wetland basin as compared to El. 2.1' for the single 20' wide culvert. The attenuation for higher tides, such as the April 6-9, 2008 peak at El. 4.4' is also improved somewhat with the twin 20' wide box sections. As indicated above, the increased benefit of the twin 20' box culvert under local storm conditions appears to be minor but would require a survey of key elevations at the 6 houses that would potentially benefit.

For the 20' wide by 8' high culvert (single or twin) at Clubhouse Road, the culvert invert was selected to match the existing 4-foot diameter culvert invert, approximately El.-3.7'. The crown of the proposed culvert at the road was set at El. +3.5' to reduce friction losses for tidal flushing by keeping the crown above the normal tidal peak (MHHW El. 2.43') with some freeboard. Thus the road culvert clear opening would be about 7.2' high. The box culvert inside height was indicated as 8' to allow for a little less than one foot of sediment to cover the concrete bottom to provide protection for fish passage and to provide a natural material on the bottom. This dimension could be increased in final design to provide additional depth of sediment if desired. The invert at the existing tide gate at El.-1.46'is about 2.2' higher than the invert of the road culvert. Therefore, there would always be a minimum of about 2' of water in the road culvert. The proposed culvert would maintain this bottom submergence condition and thus eliminate the need for a low flow or fish passage channel in the culvert. The sidewall height of the culvert can be set based upon the desired depth of sediment in the bottom.

The results of the above analyses will be reviewed with the project team prior to selection of a culvert for Clubhouse Road. Once the basic size is selected, a decision can be made as to whether it should be a single cell culvert or a twin culvert. This choice may be based upon clearances, ease of construction, hydraulic conditions at the inlet and outlet and other factors.

5.0 Hydrologic Criteria for Future Facilities

The XP SWMM model and hydrologic analyses described above can serve to facilitate the design of other project components in the future. The analyses completed for this Report serve to provide a basis for selection of the size of the Clubhouse Road culvert so that the new culvert will not become the controlling facility in the new outlet works for the Cox Hall Creek wetland. The future water control structures and pumping station should be the controlling facilities to allow for appropriate management of the tidal flow and appropriate sizing of the pump station.

The hydraulic analyses of the road culvert alternatives were based on the concept that the future outfall facilities to the bay will have hydraulic capacity compatible with that of the new road culvert. If a decision is made to construct new outfall facilities with less capacity than the road culvert, then the resulting water surface elevations in the Cox Hall Creek wetland basin will be controlled by those facilities of lesser capacity and not by the road culvert, which would permit the desired level of management.

The outfall facilities downstream of the Clubhouse Road culvert include the connecting channel, the tide gate facilities and the pumping station. Except for the pump station, these are all gravity controlled facilities.

5.1 Gravity Inlet/Outlet Facilities

Since the tide gates (or future water control structures) and the channel between Clubhouse Road and the tide gates are gravity controlled facilities, their capacity depends upon their size, invert and crown elevations, material and the differential in water surface elevation between the bay and the wetland basin. To achieve the water level elevations for storm and tidal events as indicated in this Report, these facilities will need to have hydraulic capacity compatible to that of the Clubhouse Road culvert. This does not mean that the outfall facilities have to be the same size as the road culvert; they just have to be compatible. For example, in the analyses described above with the road culvert analyzed as a 20' wide box culvert with a sediment bottom, crown El. 3.5' and invert El. -3.7', providing a clear opening of 7.2'. The outfall in the model was based upon the same width and crown elevations, but with an invert elevation similar to that of the existing outfall at El. -1.6', thus providing a clear opening of 5.1'. These elevations and culvert width may need to be adjusted in final design but serve as a guide in selecting a compatible outfall culvert.

The compatible hydraulic capacity of the gravity structures can be selected from the peak flow rates for the selected design storm, as appear in the table below. For tidal flow, the capacity should be similar to that of the road culverts as follows:

- For the single cell 20' wide culvert, peak tidal flow of about 240 cubic feet per second (cfs)
- For the twin 20' wide box culvert, peak tidal flow of about 356 cfs.

It should be noted that the peak velocity and peak rate of flow in the culvert do not necessarily occur at the same time as the peak water level. For the tidal flow, the time of peak water level in the wetland basin is actually a time of low velocity since this is the slack period when the tide is changing from inflow to outflow. The XP SWMM model can be used to determine the peak velocities and associated water levels in final design.

5.2 Pumping Station

For some storms, coincident with low or normal tide cycles, there will not be a need for pumping due to the large storage volume of the Cox Hall wetland basin, provided that adequate capacity is provided in the outfall gravity facilities as described above. However, during sustained surge tides with little flow out of the basin, stormwater pumping would be required to achieve the same level of protection as afforded by the capacity of the road culvert. The design capacity of the pump station will depend upon the selected level of protection under such surge conditions. The following table presents the peak flow at the Clubhouse Road culvert for a range of storms.

TABLE 5

Peak Storm Flows at Cox Hall Creek Wetland Basin

Storm	Peak Flow In (cfs)	1 Cell @ 20' Peak Flow Out (cfs)	2 Cells @ 20' Peak Flow Out (cfs)
2-Year	204	287	400
5-Year	376	329	467
10-Year	542	371	529
25-Year	817	433	645
50-Year	1048	484	749
100-Year	1354	554	880
500-Year	2081	665	1137

The above table requires some interpretation for understanding. For example, the peak outflow from the wetland basin can be greater than the peak inflow if the outflow occurs as the tide is falling after the basin has accumulated a significant volume of water. Thus the out rush of water can be more than the inflow peak rate, particularly for smaller storms. However, in the event of a tidal surge, there is no need to have a pump capacity equal to this peak outflow for these smaller storms since the flood levels would be below elevations of concern.

For the larger storms, the peak outflow rate achievable by gravity flow is very high for a stormwater pumping station. If there is a sustained surge tide and if the pumping capacity is not equal to the culvert capacity, the water level will increase in the wetland basin and will potentially reach the elevations indicated for existing conditions on Figure 3 for "Blocked" outlet facilities. The peak water level should be no more than under existing conditions except for the condition where tidal flow is allowed into the wetland basin prior to a storm and is at or above approximately

El. 2' when the storm flow is blocked by the tidal surge. Based upon the peak tidal inundation elevation selected for management of the wetlands, the appropriate sized pumping station can be selected to provide for pumping out of the stored water or otherwise compensating for the reduction in stormwater storage volume. The XP SWMM model can be used for this analysis.

It may be practical to establish one design storm for gravity flow and a lesser storm for pumped flow. For example, it may be desirable to design the gravity outflow facilities to match the capacity of a single 20' wide culvert at Clubhouse Road, providing for 100-year storm protection for most of the houses near the wetland basin, but have a smaller design storm or higher allowable flood elevation for pumping required in conjunction with a tidal surge. Again, the XP SWMM model can be used to further analyze alternatives. However, for refinement of allowable flood elevations, survey data should be obtained for the flood entry points at the 20 or so lowest houses (grade at or below El 4.5').

6.0 Conclusions and Recommendations

(Preliminary pending review meeting of June 18, 2008)

6.1 Clubhouse Road Culvert

Based upon the results of the XP SWMM model analyses of the Cox Hall Creek basin, it is suggested that further consideration be given to selecting a 20' wide culvert as described above (or the equivalent) for the replacement of the existing 4-foot diameter culvert at Clubhouse Road. This size should be sufficient to allow flexibility in optimizing the size of the outfall facilities, potentially providing for 100-year local flood protection for adjacent houses except under conditions of coincidental above normal tidal surges in the bay. Final selection of the dimensions of this culvert is subject to other considerations, such as site constraints, ease of construction, clearances, and erosion control at the inlet and outlet. It may be preferable to provide a twin barrel culvert of equivalent capacity, rather than a single cell 20' wide box section.

6.2 XP SWMM Model

The XP SWMM model developed for analysis of the Cox Hall Creek wetland basin can be further utilized and modified to refine the design criteria for other project components, including the outfall culvert, tide control structures and pumping station. To facilitate further analyses, a design storm should be selected for gravity outflow. It may be desirable to select a lesser storm for pumped outflow. The allowable flood elevations for these analyses should be based upon survey elevations of the point of water entry into the lower 20 or so houses in the area bordering the wetland basin. Once these elevations are determined, they can be compared to the peak water elevations indicated by the XP SWMM model.

The XP SWMM model can also be utilized to evaluate a selected combination of outlet culverts and water control structures to develop operations criteria so that the existing swamp pink population along Cox Hall Creek is not affected by salt water allowed to flow into the wetland area. For evaluation of this condition, surveyed elevations of the lowest portion of the swamp pink will be required and a reasonable freeboard or margin of safety will need to be established to set the allowable tidal flood elevation.

\\MIL-DATA\projects-inc\243271 Cox Hall Creek\6.1 Report 0806\WorkingDraft Hydrology Report 6-17-08.doc

Appendix A – Hydrology Data

1. NOAA Atlas 14 Cape May Rainfall Data
2. NOAA Cape May Tide Gage Data
3. New Jersey Department of Agriculture State Soil Conservation Committee – Technical Bulletin 2004-2.0

NOAA Atlas 14 Cape May Rainfall Data

POINT PRECIPITATION FREQUENCY ESTIMATES FROM NOAA ATLAS 14

CAPE MAY 3 W, NEW JERSEY (28-1351) 38.9536 N 74.9358 W 0 feet

from "Precipitation-Frequency Atlas of the United States" NOAA Atlas 14, Volume 2, Version 3
G.M. Bonnin, D. Martin, B. Lin, T. Parzybok, M.Yekta, and D. Riley
NOAA, National Weather Service, Silver Spring, Maryland, 2004
Extracted: Mon Jan 7 2008

Confidence Limits | Seasonality | Location Maps | Other Info. | GIS data | Maps | Help | Docs | U.S. Map

Precipitation Frequency Estimates (inches)

ARI* (years)	5 min	10 min	15 min	30 min	60 min	120 min	3 hr	6 hr	12 hr	24 hr	48 hr	4 day	7 day	10 day	20 day	30 day	45 day	60 day
1	0.34	0.54	0.68	0.93	1.16	1.36	1.50	1.87	2.22	2.48	2.87	3.13	3.62	4.09	5.53	6.96	8.89	10.69
2	0.40	0.64	0.81	1.12	1.40	1.66	1.83	2.27	2.68	3.02	3.48	3.80	4.35	4.91	6.59	8.25	10.49	12.58
5	0.48	0.76	0.96	1.37	1.75	2.11	2.33	2.87	3.41	3.93	4.52	4.89	5.51	6.12	7.97	9.82	12.29	14.55
10	0.53	0.85	1.08	1.56	2.04	2.48	2.74	3.39	4.07	4.71	5.42	5.82	6.49	7.12	9.08	11.07	13.66	16.02
25	0.60	0.96	1.22	1.81	2.41	2.95	3.30	4.12	5.02	5.90	6.77	7.20	7.93	8.57	10.65	12.79	15.47	17.89
50	0.66	1.05	1.32	2.00	2.70	3.35	3.76	4.75	5.87	6.94	7.95	8.40	9.17	9.77	11.90	14.14	16.84	19.27
100	0.71	1.13	1.42	2.18	3.00	3.75	4.24	5.41	6.80	8.11	9.26	9.72	10.52	11.06	13.21	15.51	18.18	20.58
200	0.76	1.20	1.51	2.36	3.31	4.16	4.75	6.12	7.83	9.43	10.74	11.18	12.00	12.44	14.55	16.89	19.48	21.82
500	0.82	1.29	1.63	2.59	3.72	4.73	5.45	7.14	9.34	11.43	12.96	13.35	14.19	14.46	16.43	18.77	21.17	23.37
1000	0.87	1.36	1.71	2.77	4.05	5.19	6.02	8.02	10.69	13.16	14.88	15.19	16.02	16.15	17.90	20.21	22.40	24.46

Text version of table

* These precipitation frequency estimates are based on a partial duration series. ARI is the Average Recurrence Interval.
Please refer to the documentation for more information. NOTE: Formatting forces estimates near zero to appear as zero.

* Upper bound of the 90% confidence interval
Precipitation Frequency Estimates (inches)

ARI** (years)	5 min	10 min	15 min	30 min	60 min	120 min	3 hr	6 hr	12 hr	24 hr	48 hr	4 day	7 day	10 day	20 day	30 day	45 day	60 day
1	0.37	0.59	0.74	1.02	1.27	1.50	1.66	2.06	2.46	2.70	3.11	3.38	3.89	4.37	5.84	7.31	9.30	11.15
2	0.44	0.71	0.89	1.23	1.54	1.83	2.02	2.50	2.96	3.29	3.79	4.12	4.69	5.24	6.94	8.66	10.98	13.12
5	0.52	0.84	1.06	1.50	1.93	2.32	2.56	3.16	3.77	4.27	4.91	5.28	5.93	6.51	8.39	10.30	12.86	15.17
10	0.59	0.94	1.19	1.72	2.24	2.72	3.02	3.73	4.50	5.11	5.87	6.26	6.97	7.57	9.57	11.59	14.29	16.68
25	0.67	1.06	1.35	2.00	2.66	3.26	3.62	4.53	5.56	6.37	7.30	7.73	8.50	9.08	11.21	13.38	16.16	18.62

50	0.73	1.16	1.47	2.22	3.00	3.70	4.13	5.24	6.53	7.50	8.55	8.99	9.81	10.36	12.52	14.79	17.59	20.07
100	0.79	1.26	1.59	2.43	3.35	4.17	4.67	6.00	7.61	8.74	9.94	10.40	11.25	11.72	13.89	16.23	18.98	21.44
200	0.85	1.35	1.70	2.64	3.71	4.64	5.24	6.82	8.80	10.13	11.51	11.94	12.85	13.20	15.32	17.68	20.35	22.76
500	0.92	1.46	1.84	2.93	4.20	5.33	6.06	8.03	10.62	12.23	13.86	14.28	15.15	15.35	17.31	19.65	22.14	24.42
1000	0.99	1.55	1.95	3.16	4.61	5.89	6.74	9.09	12.23	14.06	15.91	16.27	17.17	17.34	18.88	21.20	23.48	25.60

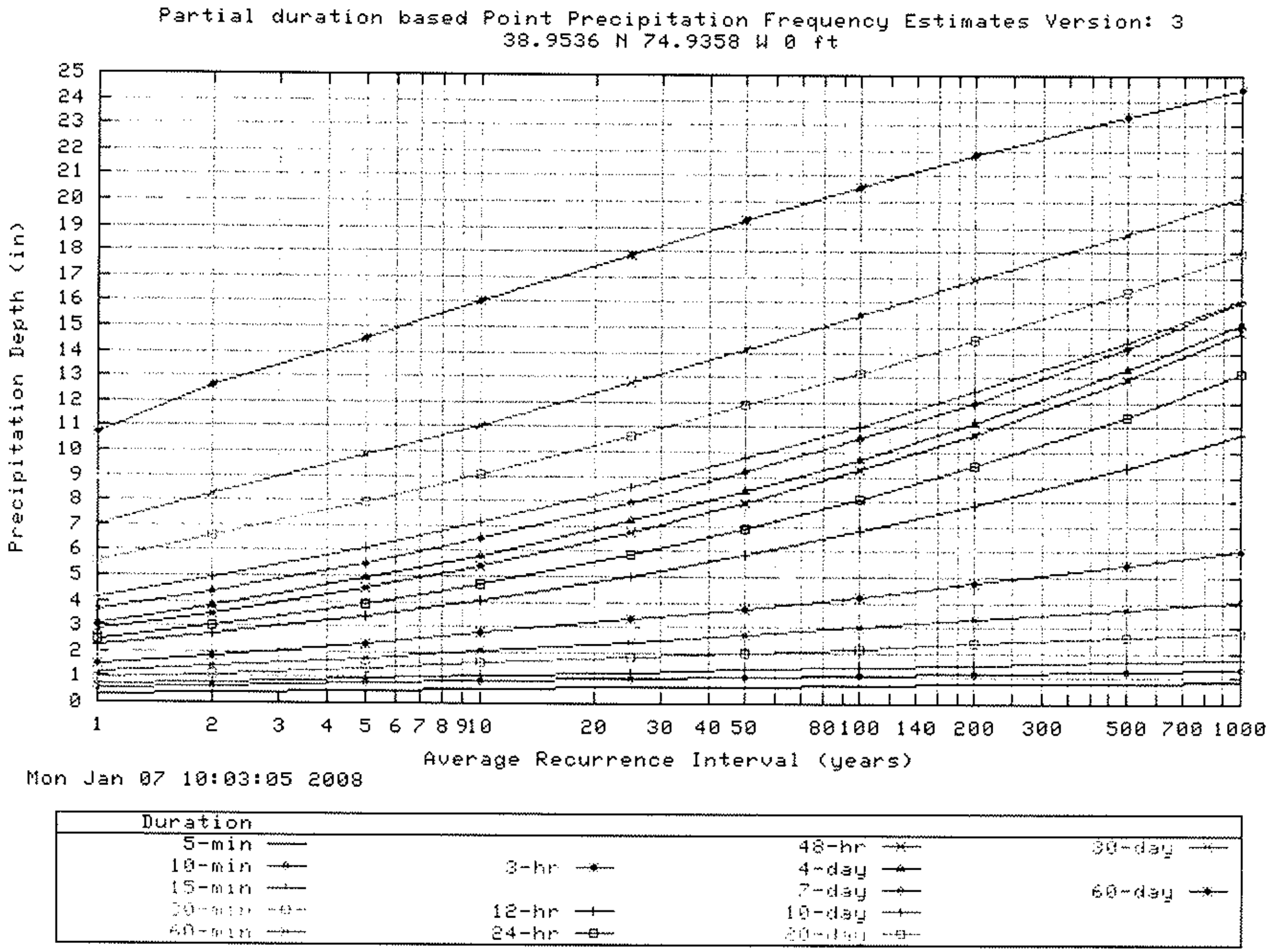
* The **upper** bound of the confidence interval at 90% confidence level is the value which 5% of the simulated quantile values for a given frequency are **greater** than.
** These precipitation frequency estimates are based on a partial duration series. ARI is the Average Recurrence Interval.
Please refer to the documentation for more information. NOTE: Formatting prevents estimates near zero to appear as zero.

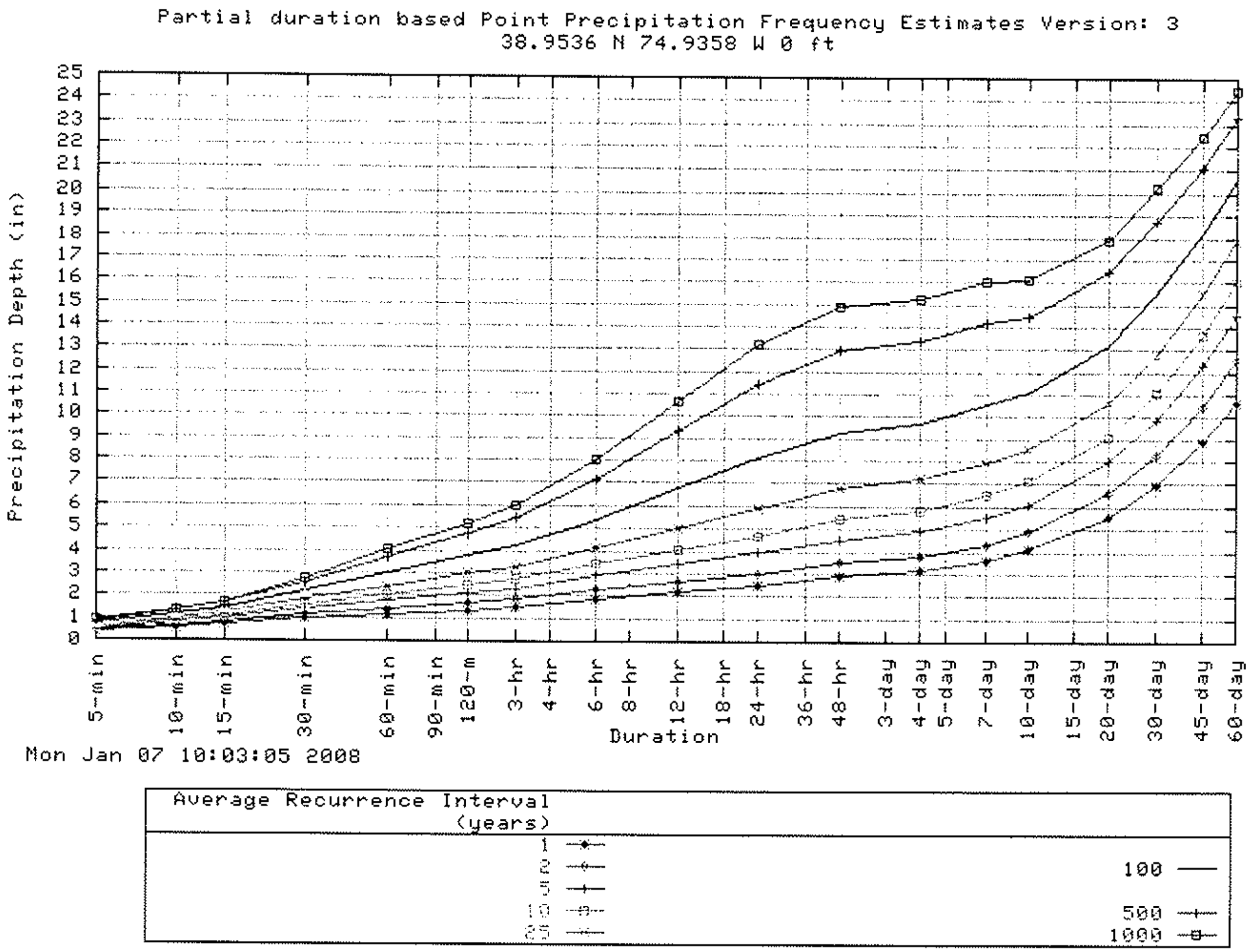
*** Lower bound of the 90% confidence interval**
Precipitation Frequency Estimates (inches)

ARI** (years)	5 min	10 min	15 min	30 min	60 min	120 min	3 hr	6 hr	12 hr	24 hr	48 hr	4 day	7 day	10 day	20 day	30 day	45 day	60 day
1	0.31	0.49	0.61	0.84	1.05	1.23	1.37	1.70	2.02	2.30	2.64	2.92	3.39	3.87	5.25	6.62	8.51	10.26
2	0.36	0.58	0.73	1.01	1.27	1.50	1.67	2.05	2.44	2.80	3.21	3.54	4.08	4.63	6.26	7.84	10.04	12.05
5	0.43	0.69	0.87	1.24	1.59	1.90	2.11	2.60	3.10	3.63	4.16	4.55	5.17	5.77	7.56	9.33	11.75	13.93
10	0.48	0.77	0.98	1.41	1.84	2.23	2.46	3.06	3.67	4.34	4.97	5.39	6.06	6.70	8.61	10.51	13.06	15.31
25	0.55	0.87	1.10	1.63	2.17	2.64	2.95	3.69	4.51	5.39	6.18	6.63	7.37	8.02	10.06	12.12	14.75	17.09
50	0.59	0.94	1.19	1.79	2.43	2.98	3.34	4.22	5.22	6.31	7.22	7.68	8.48	9.11	11.21	13.37	16.02	18.39
100	0.63	1.01	1.27	1.95	2.69	3.33	3.74	4.77	5.97	7.31	8.35	8.82	9.66	10.25	12.40	14.60	17.24	19.59
200	0.67	1.07	1.35	2.10	2.94	3.65	4.15	5.33	6.76	8.40	9.61	10.05	10.94	11.45	13.59	15.83	18.42	20.72
500	0.72	1.14	1.43	2.28	3.27	4.11	4.70	6.10	7.87	10.03	11.44	11.87	12.75	13.16	15.23	17.49	19.93	22.11
1000	0.76	1.19	1.49	2.42	3.53	4.47	5.13	6.77	8.83	11.40	12.99	13.37	14.25	14.57	16.48	18.76	21.01	23.08

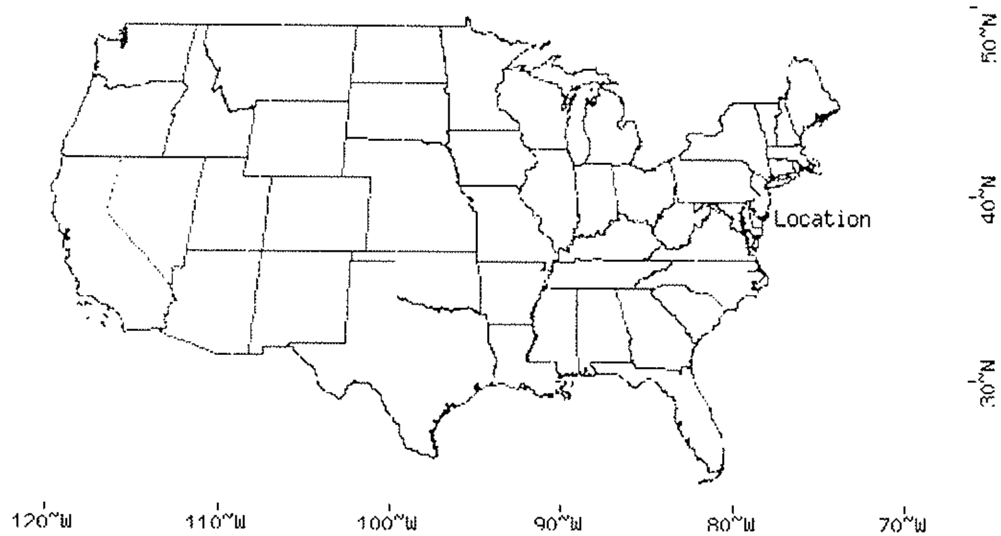
* The **lower** bound of the confidence interval at 90% confidence level is the value which 5% of the simulated quantile values for a given frequency are **less** than.
** These precipitation frequency estimates are based on a partial duration maxima series. ARI is the Average Recurrence Interval.

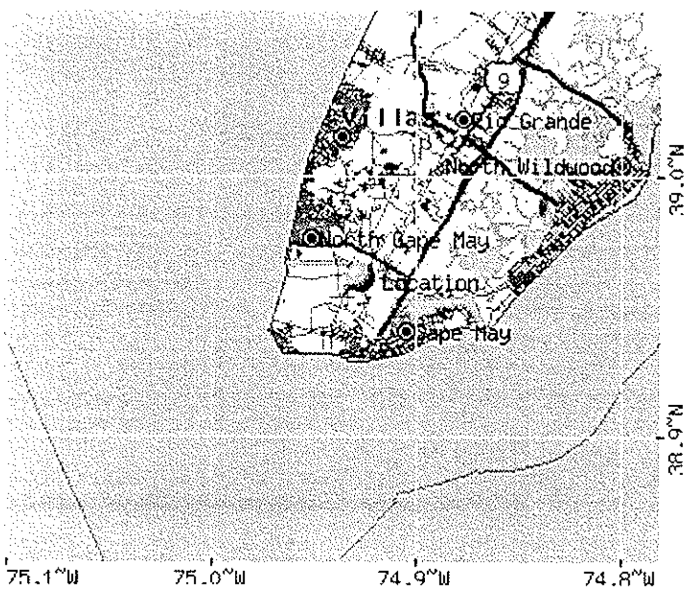
Please refer to the documentation for more information. NOTE: Formatting prevents estimates near zero to appear as zero.


Partial duration based Point Precipitation Frequency Estimates Version: 3
38.9536 N 74.9358 W 0 ft
Precipitation Depth (in)
Average Recurrence Interval (years)
Mon Jan 07 10:03:05 2008
Duration
5-min
10-min
15-min
30-min
60-min
3-hr
12-hr
24-hr
48-hr
4-day
7-day
10-day
20-day
30-day
60-day


Partial duration based Point Precipitation Frequency Estimates Version: 3
38.9536 N 74.9358 W 0 ft
Precipitation Depth (in)
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
25
5-min
10-min
15-min
30-min
60-min
90-min
120-m
3-hr
4-hr
6-hr
8-hr
12-hr
18-hr
24-hr
36-hr
48-hr
3-day
4-day
5-day
7-day
10-day
15-day
20-day
30-day
45-day
60-day
Duration
Mon Jan 07 10:03:05 2008
Average Recurrence Interval
(years)
1
2
5
10
25
100
500
1000

Maps -





These maps were produced using a direct map request from the U.S. Census Bureau Mapping and Cartographic Resources Tiger Map Server.

Please read disclaimer for more information.

LEGEND

State	Connector
County	Stream
Indian Resv	Military Area
Lake/Pond/Ocean	National Park
Street	Other Park
Expressway	City
Highway	County

Scale 1:228583
0 2 4 6 8 mi
0 2 4 6 8 10 km
*average--true scale depends on monitor resolution

Other Maps/Photographs -

View USGS digital orthophoto quadrangle (DOQ) covering this location from TerraServer; **USGS Aerial Photograph** may also be available from this site. A DOQ is a computer-generated image of an aerial photograph in which image displacement caused by terrain relief and camera tilts has been removed. It combines the image characteristics of a photograph with the geometric qualities of a map. Visit the USGS for more information.

Watershed/Stream Flow Information -

Find the Watershed for this location using the U.S. Environmental Protection Agency's site.

Climate Data Sources -

Precipitation frequency results are based on data from a variety of sources, but largely NCDC. The following links provide general information about observing sites in the area, regardless of if their data was used in this study. For detailed information about the stations used in this study, please refer to our documentation.

Using the **National Climatic Data Center's (NCDC)** station search engine, locate other climate stations within:

+/-30 minutes ...OR... +/-1 degree of this location (38.9536/-74.9358). Digital ASCII data can be obtained directly from NCDC.

Hydrometeorological Design Studies Center
DOC/NOAA/National Weather Service
1325 East-West Highway
Silver Spring, MD 20910

(301) 713-1669
Questions?: HDSC.Questions@noaa.gov

NOAA Cape May Tide Gage Data

Station Information
Tide / Water Level Data
Tide Predictions
Current Data
Meteorological Observations
Conductivity
PORTS
Operational Forecast System
Bench Mark Sheets
Datums
Harmonic Constituents
Sea Level Trends

Cape May, NJ

Cape May, NJ: Data Inv

Station ID: 8536110

Pag

Datums

Click HERE for printable version

Data Units: (•) Feet ( ) Meters [Apply Change]

Jan 7 2008 10:31 — ELEVATIONS ON STATION DATUM
National Ocean Service (NOAA)

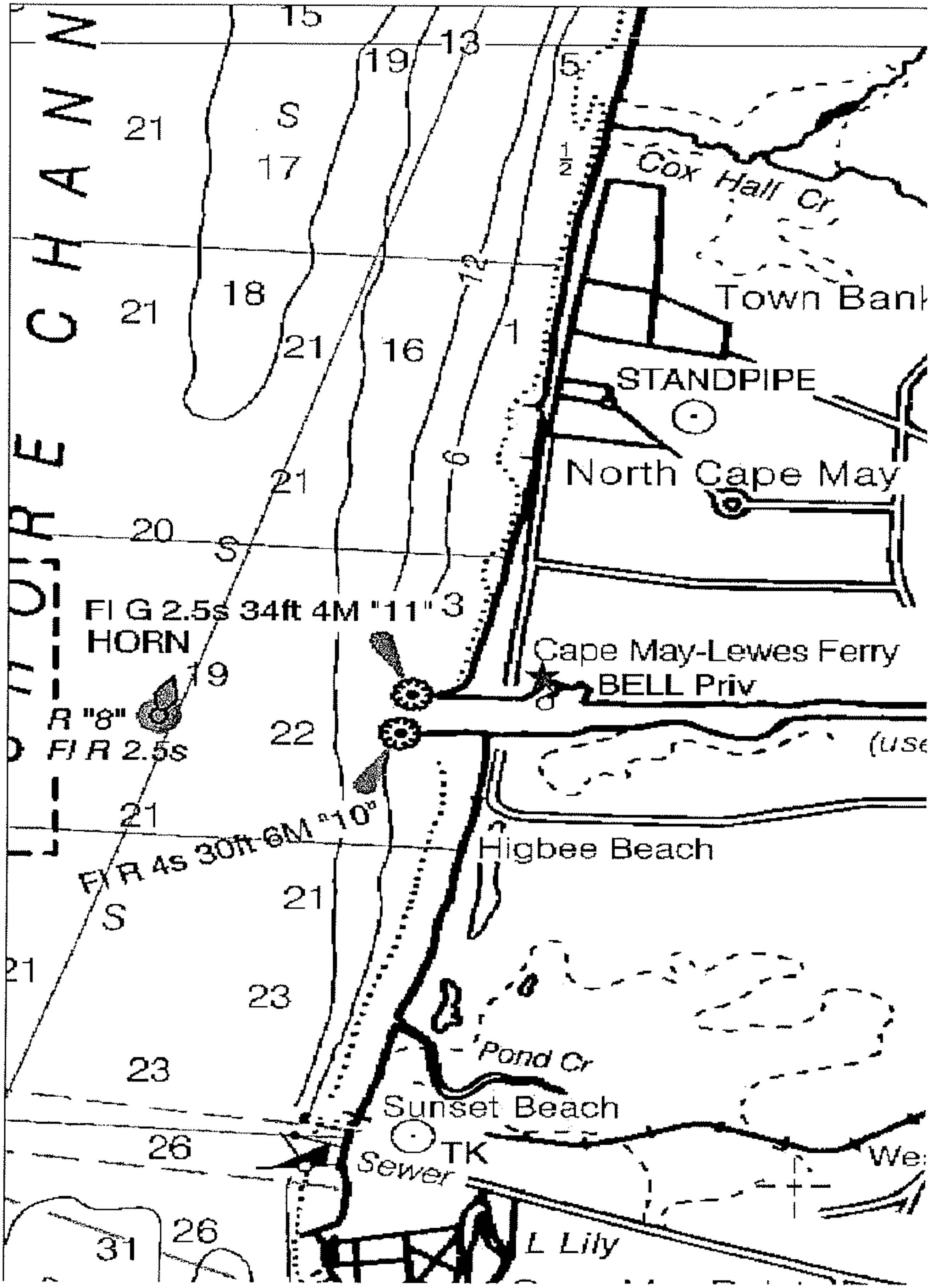
Station: 8536110 — T.M.: (
Name: CAPE MAY, CAPE MAY CANAL, DELAWARE BAY, NJ — Units: Fe
Status: Accepted — Epoch: 1983-2(

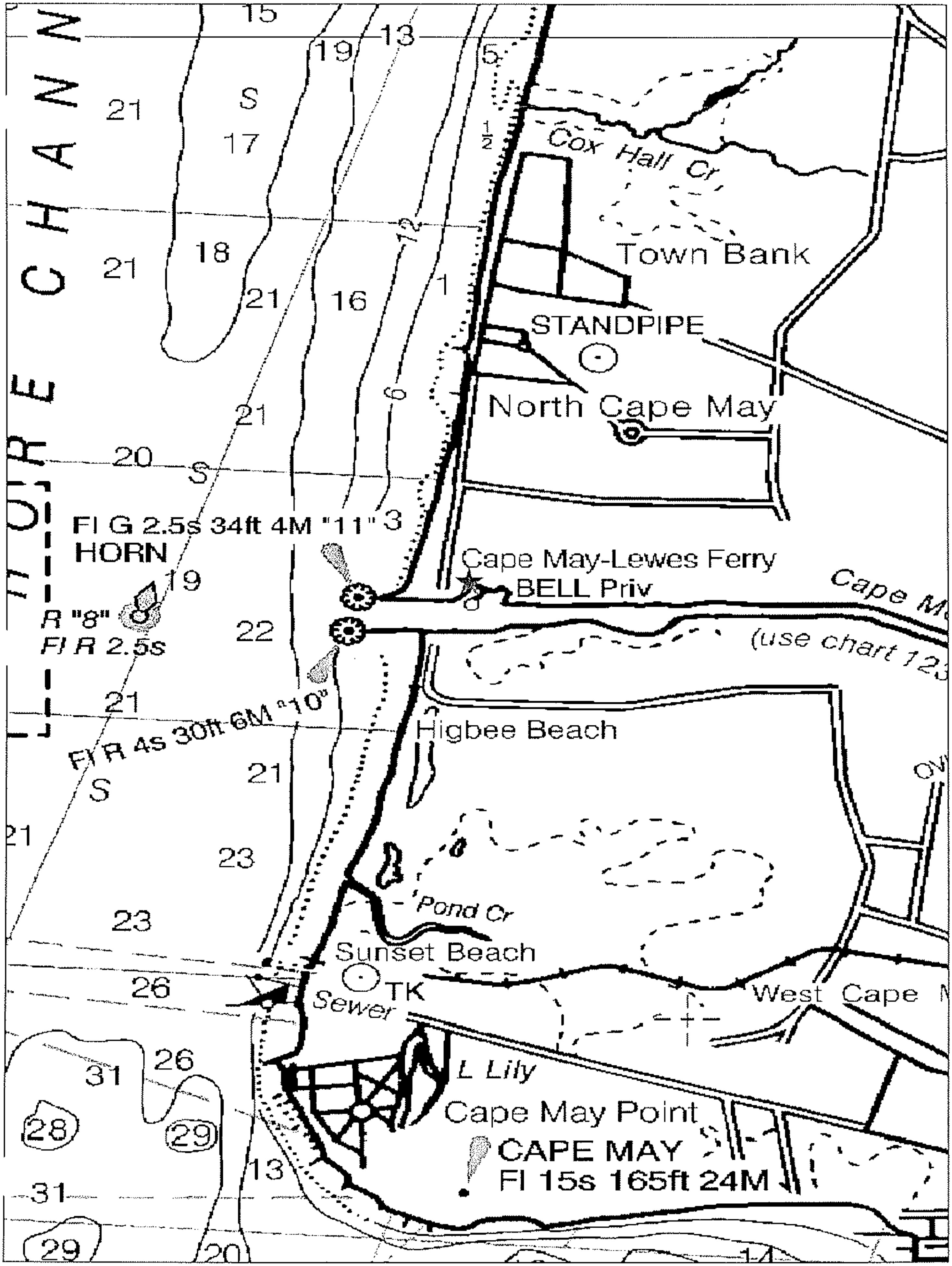
NAVD	Datum	Value	Description
2.43	MHHW	7.87	Mean Higher-High Water
1.99	MHW	7.43	Mean High Water
	DTL	5.14	Mean Diurnal Tide Level
	MTL	5.01	Mean Tide Level
-0.45	MSL	4.99	Mean Sea Level
-2.86	MLW	2.58	Mean Low Water
-3.02	MLLW	2.42	Mean Lower-Low Water
	GT	5.44	Great Diurnal Range
	MN	4.85	Mean Range of Tide
	DHQ	0.44	Mean Diurnal High Water Inequality
	DLQ	0.16	Mean Diurnal Low Water Inequality
	HWI	1.01	Greenwich High Water Interval (in Hours)
	LWI	7.07	Greenwich Low Water Interval (in Hours)
	NAVD	5.44	North American Vertical Datum
	Maximum	11.23	Highest Water Level on Station Datum
	Max Date	19850927	Date Of Highest Water Level
	Max Time	08:00	Time Of Highest Water Level
	Minimum	-0.60	Lowest Water Level on Station Datum
	Min Date	19710128	Date Of Lowest Water Level
	Min Time	03:12	Time Of Lowest Water Level

To refer Water Level Heights to a Tidal Datum, apply the desired Datum Valu

Click HERE for further station information including New Epoch products.

To refer Water Level Heights to either
NGVD (National Geodetic Vertical Datum of 1929) or
NAVD (North American Vertical Datum of 1988), apply the values locate
National Geodetic Survey


CHANN
Cox Hall Cr
Town Bank
STANDPIPE
North Cape May
Fl G 2.5s 34ft 4M "11"
HORN
R "8"
Fl R 2.5s
Cape May-Lewes Ferry
BELL Priv
Fl R 4s 30ft 6M "10"
Higbee Beach
Pond Cr
Sunset Beach
TK
Sewer
L Lily


Cox Hall Cr
Town Bank
STANDPIPE
North Cape May
Fl G 2.5s 34ft 4M "11"
HORN
Cape May-Lewes Ferry
BELL Priv
R "8"
Fl R 2.5s
Fl R 4s 30ft 6M "10"
Higbee Beach
Pond Cr
Sunset Beach
TK
Sewer
L Lily
Cape May Point
CAPE MAY
Fl 15s 165ft 24M

TIDES & CURRENTS
Search
Home | Products | Programs | Partnerships | Education | Help

Station Information
Tide / Water Level Data
Tide Predictions
Current Data
Meteorological Observations
Conductivity
PORTS
Operational Forecast System
Bench Mark Sheets
Datums
Harmonic Constituents
Sea Level Trends

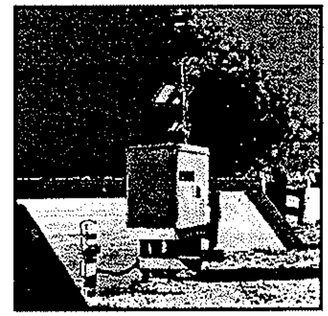
Cape May, NJ

Station ID: 8536110

Station Information

Latitude: **38° 58.1' N**
Longitude: **74° 57.6' W**
Established: **Oct 25 1965**
Present Installation: **Aug 22 1990**
NOAA Chart #: **12316**
Time Meridian: **75**

Mean Range: **4.85 ft.**
Diurnal Range: **5.44 ft.**



Click image for larger image.

Minimum Water Level:
-3.02 ft. below MLLW
(01/28/1971)

Maximum Water Level:
3.36 ft. above MHHW
(09/27/1985)

Data Types Available:

Primary Water Level
Backup Water Level
Wind
Air Temperature
Water Temperature
Barometric Pressure

Station and Bench Mark Drawing

Click HERE for Drawing
(Not for navigational use)

Station Location Chartlet

Click HERE for Map
(Not for navigational use)

EPOCH Update Information:

EPOCH Datum Comparison: **Click HERE -** check datum differences between the old epoch (1960-1978) and the new epoch (1983-2001)

Superceded Bench Mark Data Sheet:	**Click HERE** -	bench mark sheet on the old Tidal Datum Epoch (1960-1978)
Superceded Datums:	**Click HERE** -	datums on the old Tidal Datum Epoch (1960-1978)
Mean Sea Level Differences List:	**Click HERE** -	mean sea level differences between the two epochs for all stations.

Mean Sea Level Difference: for 8536110 Cape May, NJ	**1983-2001**	**1960-1978**	**Difference:**
	4.99 ft.	4.74 ft.	0.25 ft.

Location:

To reach the tidal bench marks from U.S. Highway 9 just north of Cape May Village, proceed west on Lincoln Boulevard for approximately 4.0 km (2.5 mi) and then turn left into the Cape May-Lewes Ferry Terminal. The bench marks are on the grounds of the terminal and along U.S. Highway 9. The tide gage and staff are at the south end of the dock between Ferry Slip # 1 and Ferry Slip # 2.

Cape May, NJ

Station ID: 8536110

Cape May, NJ: Data Inv

Pag

Data Inventory

Note: First/Last report will not show any breaks in data

First		Last	Data Type
11/07/2001 13:00	-	01/07/2008 15:24	**Wind**
03/06/2002 16:06	-	01/07/2008 15:24	**Air Temperature**
05/22/1997 15:00	-	01/07/2008 15:24	**Water Temperature**
08/16/2002 16:18	-	01/07/2008 15:24	**Barometric Pressure**
01/01/1996 00:00	-	12/31/2007 23:54	**Ver 6-Minute Water Level**
01/01/2001 00:00	-	01/07/2008 15:24	**Preliminary 6-Minute Water Leve**
06/13/2007 15:42	-	01/07/2008 15:18	**6-Minute Water Level**
11/21/1965 00:00	-	12/31/2007 23:00	**Ver Hourly Height Water Level**
05/06/1980 13:06	-	12/31/2007 23:54	**Ver High/Low Water Level**
12/01/1965 00:00	-	12/31/2007 23:54	**Ver Monthly Mean Water Level**

Note: "Ver" means verified.

New Jersey Department of Agriculture State Soil Conservation Committee – Technical Bulletin 2004-2.0

NEW JERSEY DEPARTMENT OF AGRICULTURE
STATE SOIL CONSERVATION COMMITTEE
Chapter 251, PL 1975 as amended,
Engineering Policies- Technical Bulletin

Technical Bulletin: 2004-2.0	**Adoption Date:** July 12, 2004
Subject: NRCS change in stormwater modeling for the NJ Coastal Plain	**From:** Hunter Birckhead, P.E., Section Chief

1.01 PURPOSE

To distribute revisions to the NRCS stormwater modeling runoff procedures utilizing the the DelMarVa unit hydrograph.

1.02 SUMMARY

NRCS runoff modeling procedures utilize a dimensionless unit hydrograph in the computation of runoff rate and volume. Until recently, the hydrograph officially used by NRCS in New Jersey was the "Standard" unit hydrograph, which represents "average" conditions. This hydrograph is characterized by a unit peak discharge factor of 484, which is more representative of the Piedmont areas of the State.

Due to recent studies by NJDA as well as an accumulation of other data, NRCS-NJ has instructed its field staff to utilize the DelMarVa unit hydrograph for runoff estimation on agricultural lands which are located within the coastal zones identified on the attached map and NRCS bulletin. This hydrograph is characterized by a peak rate factor of 285, and will predict a substantially lower peak discharge than that of the Standard hydrograph. Volume of runoff will not be affected by this change.

DelMarVa hydrograph usage is primarily restricted to areas that have slopes less than 5%, permeable soils and are characterized by "ponded" topography capable of capturing and holding some degree of precipitation prior to runoff occurring. It should not be utilized for design in areas where the predeveloped conditions are heavily urbanized and the drainage area is characterized by significant areas of impervious cover. The Standard hydrograph should continue to be used in these areas.

Districts located within the specified zones are to use and require usage of the DelMarVa unit hydrograph in the preparation of Chpt. 251 applications where it is necessary to compute runoff rate and volume with NRCS modeling methods.

This requirement is effective 60 days from the date of this bulletin.

State of New Jersey
DEPARTMENT OF AGRICULTURE
HEALTH / AGRICULTURE BUILDING
JOHN FITCH PLAZA
PO BOX 330
TRENTON NJ 08625-0330

MES E. MCGREEVEY
Governor

CHARLES M. KUPERUS
Secretary

To: Distribution List

From: Hunter Birckhead, P.E., Section Chief
State Soil Conservation Committee

Re: USDA-NRCS, Technical Bulletin NJ210-3-1

Date: July 12, 2004

The SSCC is transmitting this bulletin to advise you of the changes outlined in NRCS-NJ Technical Bulletin NJ210-3-1, issued September 8, 2003 regarding the Dimensionless Unit Hydrograph (copy enclosed).

Bulletin NJ210-3-1 describes the application of the DELMARVA unit hydrograph for use in modeling surface runoff in certain areas of the South Central flat inland and New Jersey Coastal Plain.

This change in modeling procedures is important to the State Soil Erosion and Sediment Control Act program. The use of the DELMARVA hydrograph should be applied to stormwater management runoff design for pre and post developed conditions in the applicable portions of the state identified in the attached documents. In general, flatter terrain, ponded areas and swales, wooded conditions and highly pervious soils characterize these areas. The DELMARVA unit hydrograph should not be utilized for design in areas where the predeveloped conditions area heavily urbanized and the drainage area is characterized by significant areas of impervious cover. The Standard NRCS hydrograph should continue to be used in those areas where the Delmarva unit hydrograph does not apply.

In order to implement this change consistently in the affected areas of the state, soil conservation districts will anticipate the use of the DELMARVA hydrograph in soil erosion and sediment control site plan designs, within sixty days of this letter. The SSCC is requesting your cooperation by advising your clientele of this bulletin.

This office is facilitating the distribution of the NRCS bulletin to Soil Conservation Districts, municipal, county and state government agencies and groups. Please feel free to contact this office if you have questions regarding this change.

Thank you for your cooperation and assistance.

Enclosure

Distribution:

Anthony Castillo, P.E., NJ Society of Professional Engineers
Richard Moralle, P.E., NJ Association of Municipal Engineers
Denis Sedaille, P.E., NJ Association of County Engineers
Anthony Di Lodovico, NJ Builders Association
Robert Kirkpatrick, Jr., P.E., NJ DCA - RSIS
Steven Jacobus, NJDEP
Larry Baier, NJDEP
Kiong Chan, NJDOT
Charles Horner, NJ Pinelands Commission

c: David Lamm, P.E., USDA-NRCS
Members, State Soil Conservation Committee
Charles M. Kuperus, Secretary of Agriculture
Monique Purcell, Director, Division of Agricultural & Natural Resources
Jim Sadley, Executive Secretary, SSCC
John Showler, SSCC
Frank Minch, SSCC
Chairmen and Managers of Mercer, Burlington, Ocean, Camden, Gloucester, Freehold, Cumberland, Salem and Cape-Atlantic Districts

United States Department of Agriculture

NRCS

Natural Resources Conservation Service
220 Davidson Ave, 4th Floor
Somerset, NJ 08873

Telephone: 732-537-6040
Fax: 732-537-6095
Web site: http://www.nj.nrcs.usda.gov

September 8, 2003

NEW JERSEY BULLETIN NO. NJ210-3-1

SUBJECT: ENG – Engineering Field Handbook Supplement
Dimensionless Unit Hydrograph

Purpose: To distribute a supplement to Chapter 2 of the Engineering Field Handbook regarding use of dimensionless unit hydrographs for modeling agricultural watersheds.

Effective Date: Effective upon receipt.

With the enhancement of NRCS modeling tools commonly used in the hydrologic design of conservation practices including EFH2 and WinTR55, it is now easier to vary the dimensionless unit hydrograph as a model input. Historically, the Standard Unit Hydrograph has been applied throughout New Jersey. Now, the Delamrva Unit Hydrograph may be used in modeling gricultural watersheds in the Coastal Plain that are characterized by flat topography (average watershed slope less than 5 percent), low relief, and significant surface storage in swales and depressions. Use of the Delamrva Unit Hydrograph will not affect the determination of runoff volume, but should result in lower peak discharges when compared to the Standard Unit Hydrograph.

Filing Instructions: Insert the attached supplement at the end of Chapter 2 in the Engineering Field Handbook.

DAVID LAMM, PE
State Conservation Engineer

Enclosure

Dist: O

Dimensionless Unit Hydrograph

The dimensionless unit hydrograph is one of several watershed related parameters incorporated into NRCS hydrologic modeling procedures. The unit hydrograph influences the shape of the runoff hydrograph generated by the model, particularly the peak rate of discharge. It does not affect the volume of runoff, which is determined by curve number. Unit hydrographs vary by watershed based on many factors including watershed size, slope and length; geomorphic and geologic characteristics; amount of storage; and degree of urbanization. A standard unit hydrograph has typically been used that represents an average condition for much of the country. It has been felt to be sufficiently accurate for the hydrologic design of conservation practices. Detailed studies, however, have been conducted in some watersheds or regions to develop more representative dimensionless unit hydrographs. With the enhancement of NRCS modeling tools, it is now easier to incorporate these unique unit hydrographs into more routine hydrologic analyses.

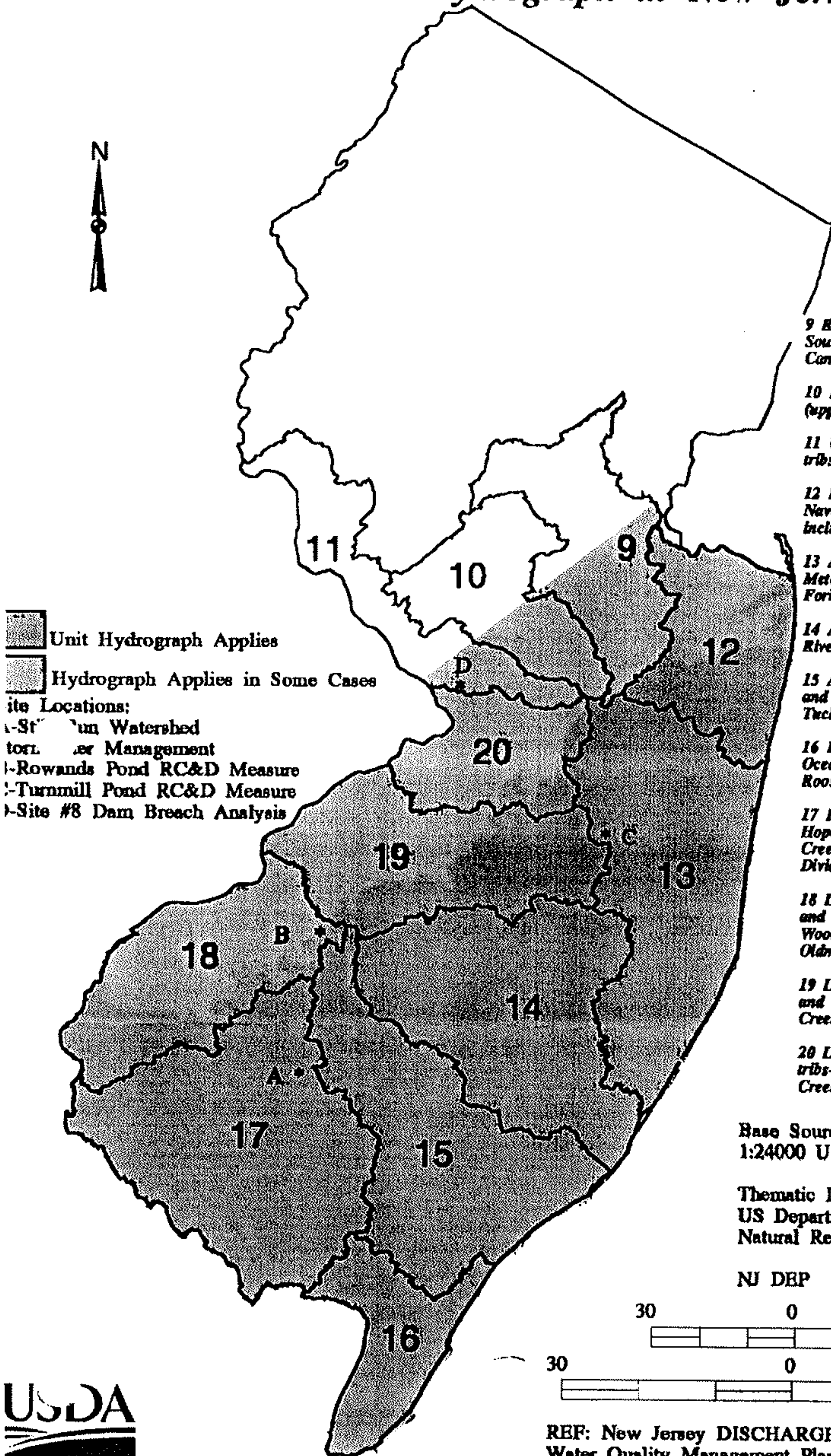
The following dimensionless unit hydrographs are applicable to New Jersey:

Delmarva Unit Hydrograph: Applies to watersheds in the Coastal Plain physiographic region that are characterized by flat topography (average watershed slope less than 5 percent), low relief, and significant surface storage in swales and depressions.

Standard Unit Hydrograph: Applies to watersheds in all other physiographic regions and to watersheds in the Coastal Plain that are not characterized by the Delmarva Unit Hydrograph.

When supported by detailed watershed studies, other unit hydrographs may be used. Study procedures are discussed in Chapter 16 of National Engineering Handbook Part 630, Hydrology.

Guide on Where to Use Delmarva Penninsula Unit Hydrograph in New Jersey



9 Raritan River Mainstem, Matchaponix Brook, South River, Green Brook, Middle Brook, D & R Canal (lower part)

10 Millstone River, Stony Brook, D & R Canal (upper part)

11 Upper Delaware (lower part of Zone 1E) and tribs-Lockatong, Alexauken Creek, Assunpink Creek

12 Raritan Bay and tribs-Shrewsbury River, Navesink River & Atlantic Ocean & tribs including Shark River

13 Atlantic Ocean and tribs-Manasquan River, Metedeconk River, Toms River, Barnegat Bay, Forked River, Little Egg Harbor, Tuckerton Creek

14 Atlantic Ocean and tribs-Mullica River, Wading River, Great Bay, Little Bay, Doughty Creek

15 Atlantic Ocean and tribs-Great Egg Harbor River and Bay, Peck Bay, Reeds Bay, Absecon Bay, Tuckahoe River

16 Delaware Bay (part of Zone 6) and Atlantic Ocean and tribs-Cape May County (south of Roosevelt Blvd, Ocean City), East and West Creeks

17 Delaware Bay (part of Zones 5 & 6) and tribs-Hope Creek, Stow Creek, Cohansey River, Back Creek, Cedar Creek, Nantuxent Creek, Dividing Creek, Maurice River

18 Lower Delaware (lower part of Zone 3, Zone 4, and part of Zone 5) and tribs-Big Timber Creek, Woodbury Creek, Mantua Creek, Raccoon Creek, Oldman's Creek, Salem River, Alloways Creek

19 Lower Delaware River (lower part of Zone 2 and upper part of Zone 3) and tribs-Rancocas Creek, Pennsauken Creek, Cooper River

20 Lower Delaware (upper part of Zone 2) and tribs-Crosswicks Creek, Doctors Creek, Blacks Creek, Crafts Creek, Assiscunk Creek

Base Source:
1:24000 USGS orthophoto quads, 1986

Thematic Data:
US Department of Agriculture
Natural Resources Conservation Service

NJ DEP

30 0 30 Kilometers

30 0 30 Miles

REF: New Jersey DISCHARGER Vol 4. No 4. Spring 1997
Water Quality Management Planning Basins

USDA
Natural Resources Conservation Service

March 1997

Appendix B - Figures

COUNTY OF CAPE MAY
CAPE MAY COUNTY, NEW JERSEY

5000' 0 5000' 10000'
SCALE IN FEET

ENVIRONMENTAL RESTORATION OF THE
COX HALL CREEK WETLAND AREA

FIGURE 1 - LOCATION PLAN

Source: U.S.G.S. 7.5 Minute Series
Topographic Mapping
CAPE MAY, NJ Quadrangle

Hatch Mott MacDonald

Certificate No. 24GA28075000

P:\243271 Cox Hall Creek\6.1 Report 0806\Figures\Location Plan.dwg 11/9/07 11:24 am

P:\243271 Cox Hall Creek\5.50 b H-H Design\Drainage Area_ACAD2000.dwg.dwg 6/10/08 3:54 pm

COUNTY OF CAPE MAY
CAPE MAY COUNTY, NEW JERSEY

ENVIRONMENTAL RESTORATION OF THE
COX HALL CREEK WETLAND AREA

FIGURE 2 - DRAINAGE AREA

SCALE IN FEET
1500' 0 1500' 3000'

Hatch Mott MacDonald

Figure 3 - Cox Hall Creek Elevation Upstream of Clubhouse Rd
Existing Condition Various Tides Conditions and Initial Wetlands Level

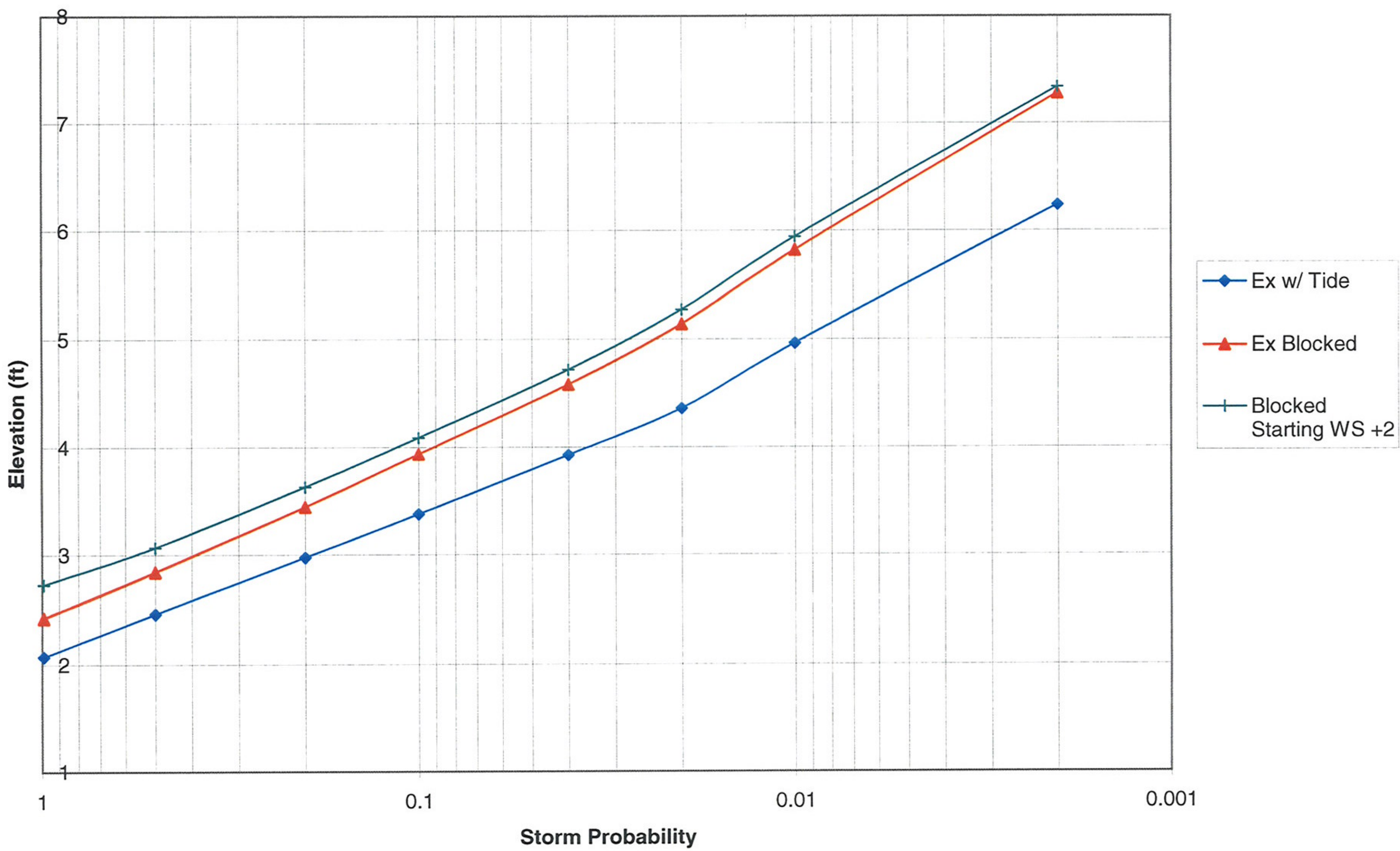


Figure 4 - Cox Hall Creek Wetlands Water Level During the Normal Diurnal Tide and During the April 6-9, 2008 Tide

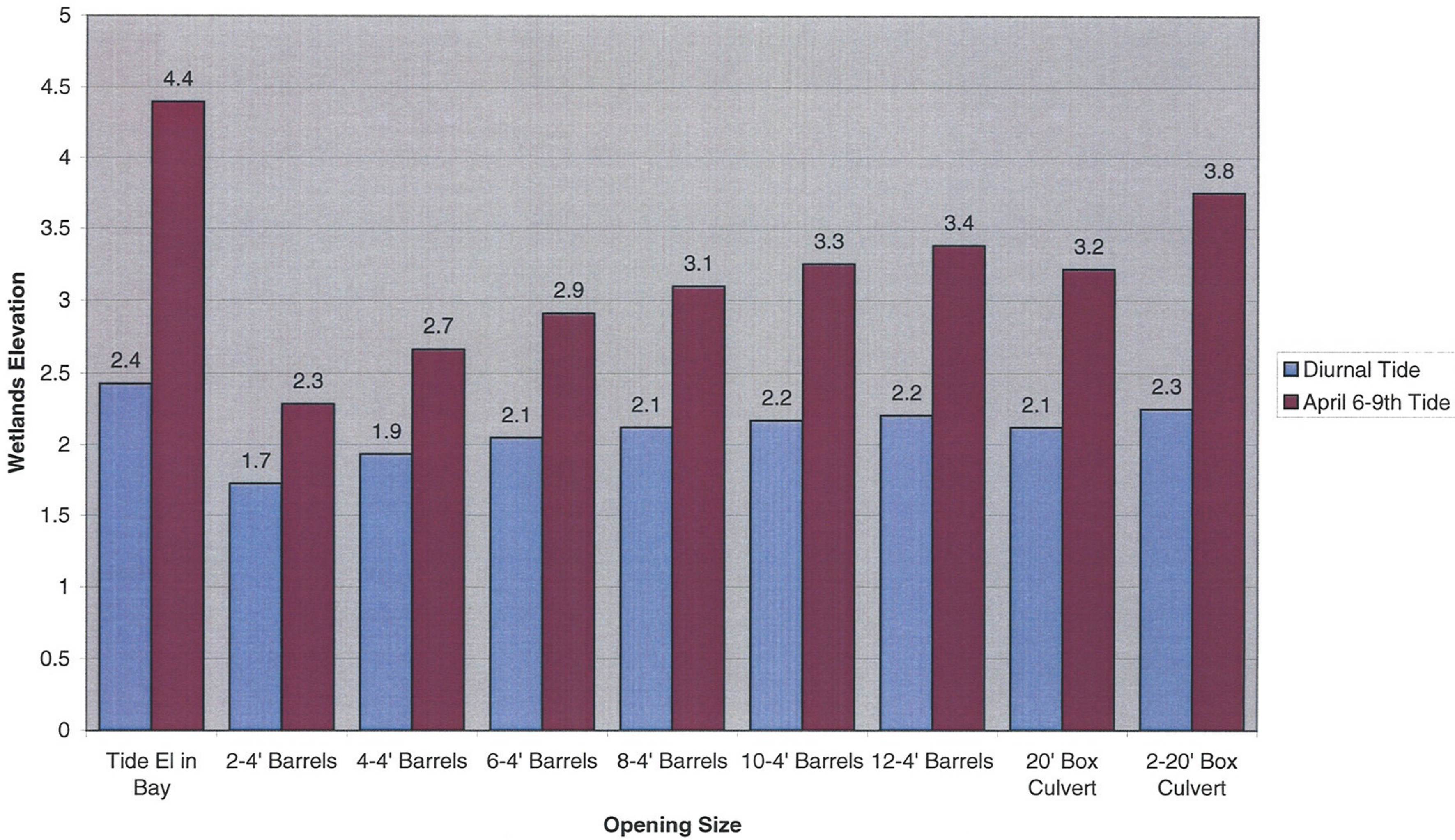


Figure 5 - Cox Hall Creek Wetlands Elevation
Multiple Barrel Screening

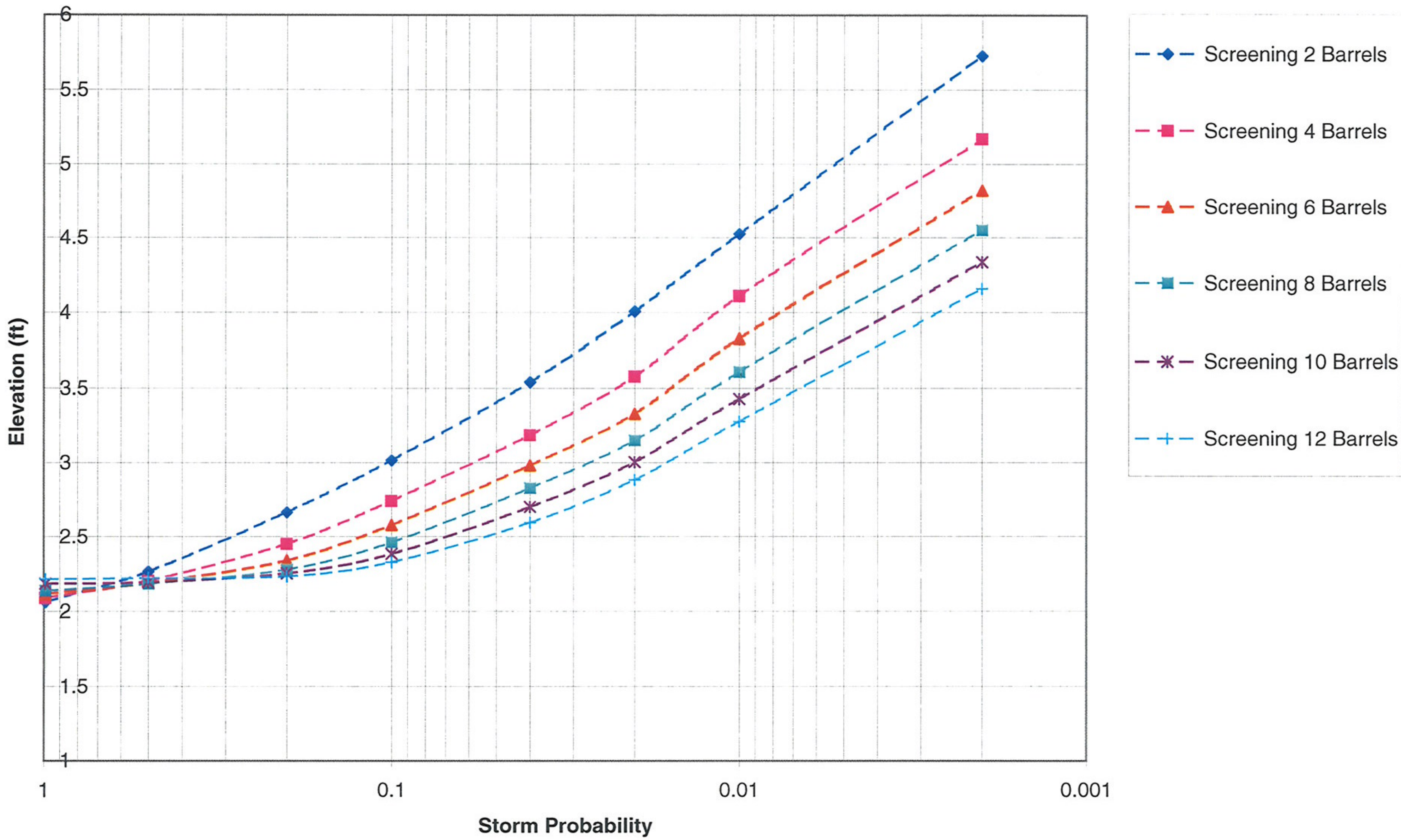


Figure 6 - Cox Hall Creek House Vs. Elevations

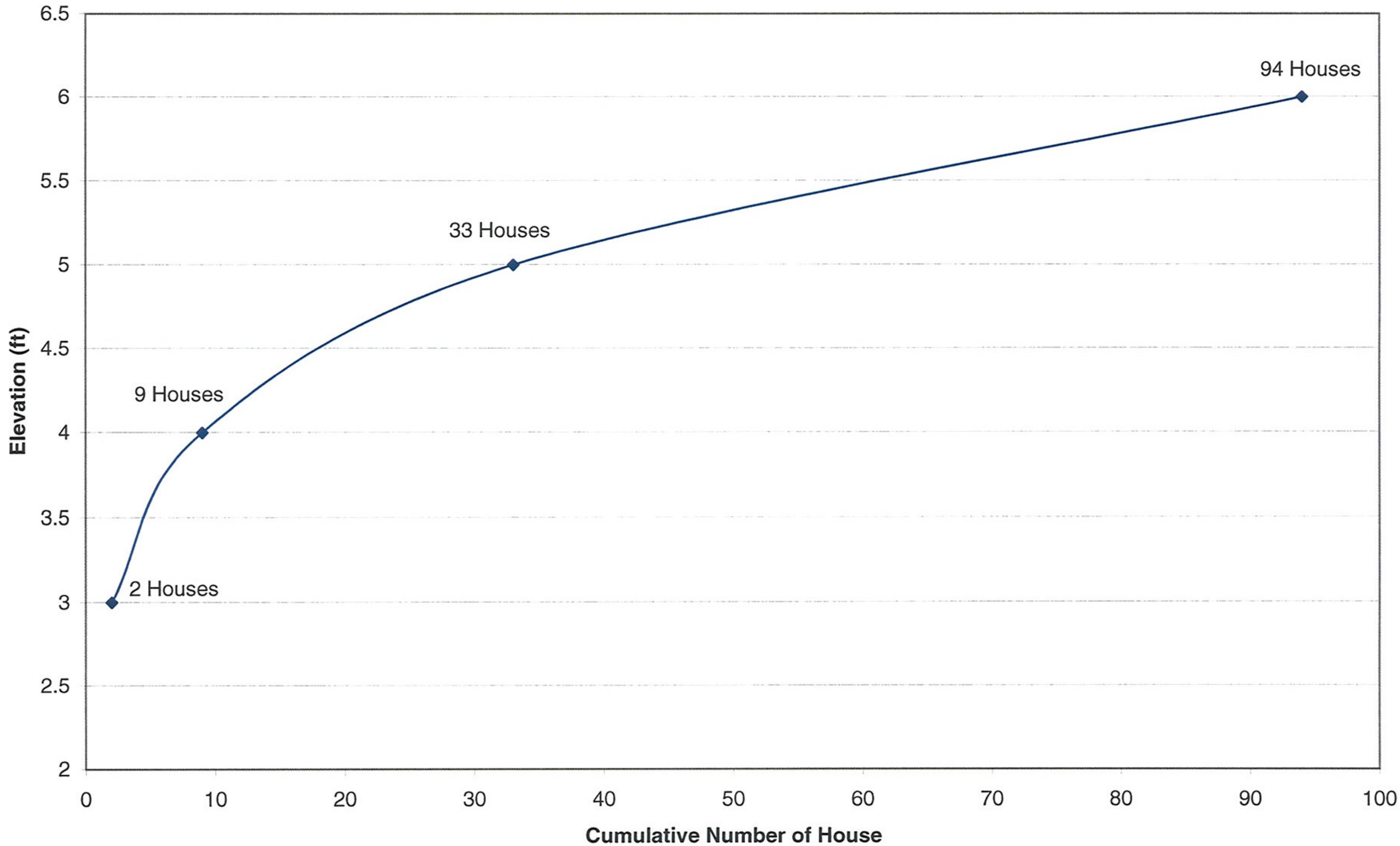


Figure 7 - Cox Hall Creek Wetlands Elevation
Proposed 20' Box Culverts

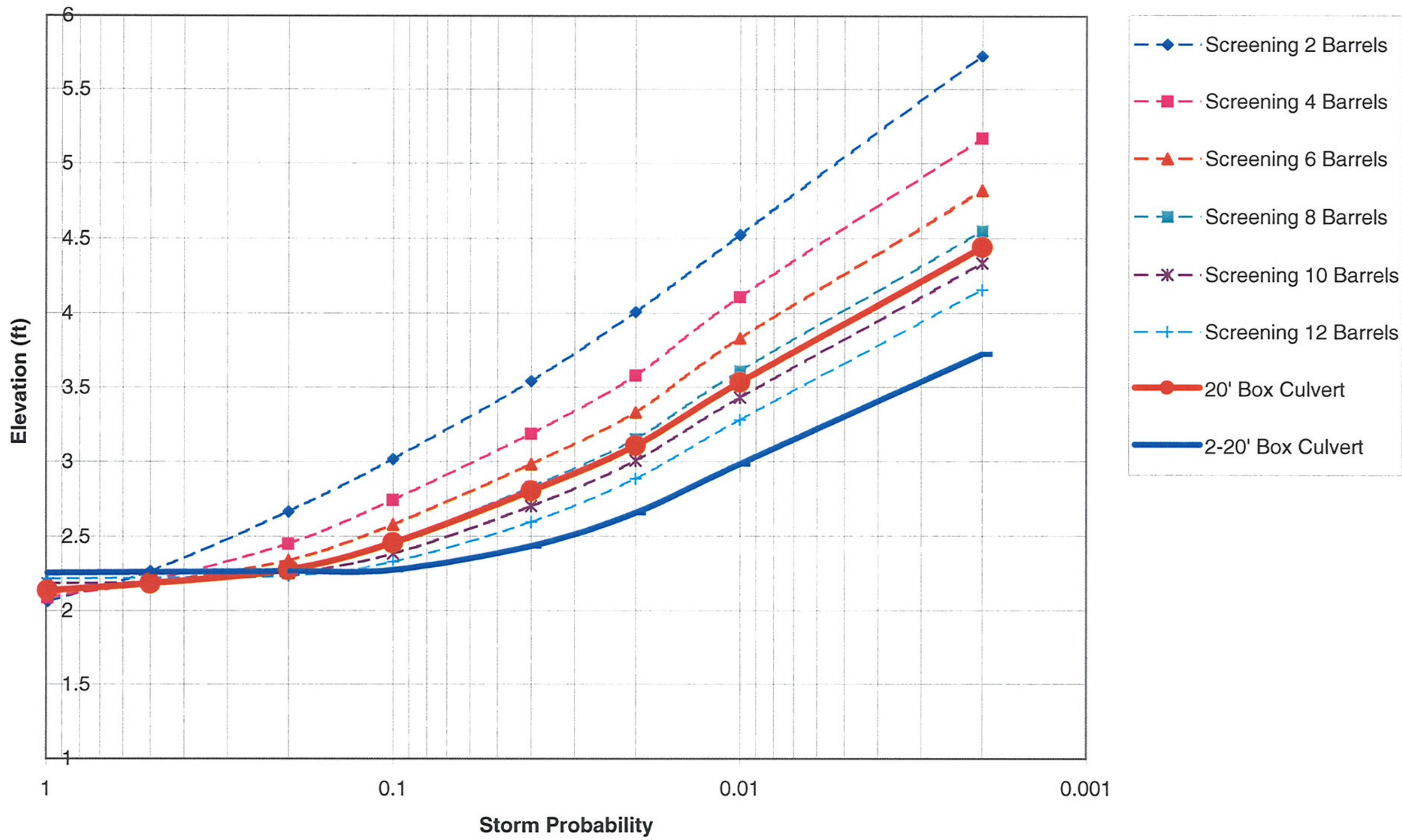


Figure 8 - Cox Hall Creek Wetland Inundation Duration