Feature Design Memorandum No. 2
Hydrology and Hydraulics

Flood Damage Reduction Project
Saugus River & Tributaries
Massachusetts

Lynn, Malden, Revere and Saugus, MA.

US Army Corps of Engineers
New England Division

DECEMBER 1993

SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION PROJECT
LYNN, MALDEN, REVERE, AND SAUGUS
MASSACHUSETTS

HYDROLOGY AND HYDRAULICS

FEATURE DESIGN MEMORANDUM NO. 2

HYDROLOGIC ENGINEERING BRANCH
HYDRAULICS AND WATER QUALITY BRANCH
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WATER CONTROL DIVISION
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DTIC
*V: Technical Report HL-93-5, May 1993 ADA 266 336
citeseerx.ist.psu.edu/viewdoc/download PDF
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IV: apps.dtic.mil/sti/pdfs/ADA 293023.pdf
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VII ADA 341675.pdf TR CHL-98-8 Mar '98

7. TIDAL ESTUARY

a. General. The lower 4.7 miles of the Saugus River, and the entire 3-mile length of its Pines River tributary, are tidal estuaries. These estuaries and adjacent saltwater marshes cover a total area in the lower Saugus River Basin of over 1,700 acres (extending from the proposed location of the floodgate structure to the upper limit of the estuary). These extensive saltwater wetlands have hydrologic as well as environmental significance.

b. Area-Capacity Curves. The total water surface area in the estuary varies from about 260 acres at normal low tide (EL -4.5 feet NGVD) to about 700 acres at normal high tide (EL +5.0 feet NGVD). Similarly, the approximate water surface area for a spring tide ranges from 230 acres at low tide (EL -5.2 feet NGVD) to 1,440 acres at high tide (EL +5.8 feet NGVD). Under a storm tide condition, with an elevation of 8.0 feet NGVD, the water surface area (total flooded area) is about 1,800 acres.

During the design phase, detailed mapping of the tidal estuary, with topographic overlays of elevations 6, 7, and 8 feet NGVD, were developed by James W. Sewall Surveyors; these maps were used to further refine the area-capacity curves developed during feasibility studies. Developed area-capacity curves for the Saugus River estuary are shown on plate 2. These curves were developed from surveys of contour elevations 6, 7, and 8 feet NGVD, along with other available topographic mapping, in association with aerial photographs of the estuary taken under a range of known tide levels. As can be seen on the area-capacity curve, the areas at elevations 6 and 7 feet NGVD are 1,600 and 1,700 acres, respectively, significantly larger than those estimated during feasibility studies. This is due to the more detailed mapping, and results in more volume of available storage than previously estimated.

8. TIDAL HYDROLOGY

a. Astronomical Tides. In the study area (figure 1), tides are semidiurnal, with two high and two low waters occurring during each lunar day (approximately 24 hours 50 minutes). The resulting tide range is constantly varying in response to the relative positions of the earth, moon, and sun, with the moon having the primary tide producing effect. Maximum tide ranges occur when orbital cycles of these bodies are in phase. A complete sequence of tide ranges is approximately repeated over an interval of 19 years, which is known as a tidal epoch.

(2) Study Area. NOS tide gage records and high watermark data gathered after major storms have been utilized in the development of profiles of tidal floods along the New England coast. Additionally, profiles of storm tides for selected recurrence intervals have been developed, utilizing tide stage-frequency curves and high watermark information. A location map and profile for the reach of New England coast bounding the project are shown in figures 9 and 10, respectively.

Additionally, studies by the Coastal Engineering Research Center (CERC) for the adjacent Roughans Point project indicate that storm tide frequency in the Saugus and Pines River system is nearly identical to that at the Boston NOS gage. CERC simulated surges for 50 randomly selected coastal storms (maximum water levels ranged from 7.9 to 11.2 feet, NGVD) using the WIFM model and determined their frequency. Results were presented in Technical Report CERC-86-8. During more frequent storm tide events, most built-up areas are protected by localized stretches of high ground, tide gates, etc. Therefore, information from the CERC report, supplemented by knowledge of local drainage patterns and past flood high watermarks, was used to develop flood stage frequency relationships for many separate damage zones. These curves and related explanation are contained in the following section.

9. TIDAL FLOOD PLAIN ZONES

a. General. Tidal flood plain zones susceptible to flood damages in the lower Saugus River Basin are generally those coastal reaches in Revere and Lynn which have direct ocean (Broad Sound) exposure and developed areas in Lynn, Revere, and Saugus, located on the periphery of the interior tidal estuaries. Existing condition flood elevation frequency estimates were made, for use by others, in determining flood damage estimates. Frequency relations were developed for 13 different hydrologic/hydraulic zones including some subareas within zones. Location of the 13 zones, seven in Revere, three in Saugus, and three in Lynn, and respective subareas are shown on plate 4. Development of the elevation frequency relationships for each zone and subarea was not a precise analytical process. Frequency curves involved consideration of the topographic and hydraulic features of each site, available information on historic flood levels, and strong reliance on the developed ocean stillwater elevation frequency relation discussed in section 8e. -"Tide Stage Frequency." In addition, results of extensive studies by CERC, concerning storm development and wave overtopping along coastal areas, were considered when revising and updating curves during design studies. With the exception of areas where physical conditions affecting flood levels have changed, the experienced February 1978 record storm tide level was assigned a one percent annual chance of occurrence, based on statistical analysis of long term storm tide records for Boston Harbor. The Boston record included adjustment of historical data for the gradual long term rise in ocean level.

b. Recent Tidal Flood History. Ocean storm accounts for the New England coast extend over a 300-year historic period, and continuous records of ocean tides at Boston have been maintained since 1922. In that period, the greatest known tide level at Boston was 10.4 feet NGVD in February 1978. Other historic events approaching that level, or exceeding after adjustment for rise in sea level, were 16 April 1851 and 26 December 1909, at elevations 10.1 and 9.9 feet NGVD and 10.5 and 10.6 feet NGVD after adjustment, respectively. No reliable systematic high water data is available in the lower Saugus River Basin for the historic storm events. Resident recollection is generally limited to the previous 10- to 15-year period. Recent notable experienced storm tides at Boston, in addition to February 1978, are: 2 January 1987 with a level of 9.5 feet NGVD; October 1991 and December 1992, both with stillwater elevations of 9.4 feet NGVD; 25 January 1979 with a stillwater elevation of 9.3 feet NGVD; and 19 February 1972 with an elevation of 9.2 feet NGVD. Rainfall accompanying these recent highest tidal events is listed in table 3. Tide elevations listed in table 3 are adjusted for sea level rise and are slightly higher than the experienced elevations listed above.

c. CERC Studies. During the Washington level review of the feasibility report, questions arose concerning storm induced wave overtopping, flood damages, and the effects of the then authorized (now constructed) Revere Beach erosion control project. Questions centered around effects the new beach would have on various project features, interior stage frequency relationships, and overall project economics. The New England Division, therefore, decided to contract with CERC for various beach erosion and storm induced wave overtopping studies. These studies are presented in Technical Report, " Revere Beach and Point of Pines, Massachusetts, Shore Front Study, Phase II," and shown in Appendix III. Results of these studies were used to refine existing condition interior flood stage frequencies, which are considered representative of the new Revere Beach condition. Since completion of the new Revere Beach, several major storms have been experienced, namely, October 1991 and December 1992. The new beach was only minimally affected by these storms (i.e., little beach erosion); therefore, a decision was made to adopt the measured beach profiles obtained after the October 1991 storm as the initial design condition (see Appendix III for detailed discussion). CERC analyzed a total of 50 storm events ranging from the SPN to 2-year stillwater flood events, and summarized overtopping per flood zone identified on plate 4. The results of these studies were used to refine the stage-frequency relationships. This new beach condition has an effect on stage frequencies, particularly in Revere flood zones 1, 2A, 2B, 4A, 4B, 4C, and 5B (zones to be protected by the Park Dike feature). The development of stage frequencies in these zones was aided by results of the CERC modelling. As stated in the CERC reports, the new beach is very effective in reducing overtopping during coastal storms, and provides a high degree of flood protection. These statements are also supported by observed interior conditions following the October 1991 and December 1992 coastal storms. There was very little interior flooding behind Revere Beach. Experienced high water elevations and developed flood stage-frequency relations for selected flood plain zones in Revere, Lynn, and Saugus, Massachusetts are listed in table 17 and shown on plates 5, 6, and 7. Experienced high water elevations shown in table 17 are pre-Revere Beach erosion project conditions. These elevations are not a measure of current flood conditions within zones behind Revere Beach, however, show flooding potential behind the beach if it erodes back to pre-1978 conditions (i.e., no beach). Even though the new erosion control beach fill proved to be highly effective in reducing overtopping during the October 1991 event, some erosion did occur. As a result, concerns expressed on the stability of the beach included the following: 1) what happens to the beach after its 50-year project life, 2) what would happen if two major tidal events occurred back to back, leaving no time to renourish the beach. In addition, there were questions raised over the confidence in the cross shore profile response model of the newly placed beach fill at Revere Beach. Due to concerns listed above, table 17A is presented to show pre-beach fill stage-frequency relationships for the zones directly affected by overtopping along Revere Beach. These stage frequencies show the worst case flooding potential if the beach were to erode back to the pre-beach fill condition. For any likely conditions, flooding levels would be expected to fall between those presented in table 17 and 17A. The area fronting the MDC police station (Park Dike) apparently received more severe erosion during the October 1991 storm than other areas along Revere Beach. In lieu of the continued uncertainty of the beach condition in this area and the consequences of returning to the no-beach condition, a decision was made to include Park Dike as a feature of the floodgate project. This decision was made to assure the flood control integrity of the floodgate project.

TABLE 17

REVERE, MASSACHUSETTS

NATURAL AND MODIFIED

FLOOD STAGE FREQUENCIES

(feet NGVD)

Location - Condition	Annual Frequencies (%)*					Experienced**			
	0.2	1.0	10	50	90	1978	1979	1987	Others
Boston Stillwater	11.2	10.3	9.2	8.4	8.0	10.3	9.3	9.4	9.1 (72) 8.9 (67)
Zone 1 - Natural	8.6	7.0	5.0	3.8	3.5	7.0	5.6	-	4.5 (72)
Modified By Tidal Protection	8.6	5.6	4.2	3.8	3.5				
Zone 2A - Natural	8.5	7.8	6.6	6.5	6.4	11.3	8.5	8.6	7.3 (67)
Modified By Tidal Protection	6.8	6.7	6.6	6.5	6.4				
Zone 2B - Natural	6.6	5.9	4.6	3.8	3.4	9.3	-	4.7	
Modified By Tidal Protection	4.8	4.6	4.4	3.8	3.4				
Zone 4A - Natural	8.3	6.5	5.2	4.0	3.4	8.3	5.5	5.0±	
Modified By Tidal Protection	5.8	5.4	4.7	4.0	3.4				
Zone 4B - Natural	8.3	6.5	3.8	3.2	3.0	6.5	-	6.5±	
Modified By Tidal Protection	4.9	4.5	3.7	3.2	3.0				
Zone 4C - Natural	9.5	7.7	5.8	5.2	5.0	7.7	-	7.0±	
Modified By Tidal Protection	5.6	5.4	5.3	5.2	5.0				
Zone 5A - Natural	11.2	10.3	9.2	8.4	8.0	10.2	-	9.0	
Modified By Tidal Protection	7.6	7.4	7.2	7.1	7.0				
Zone 5B - Natural	11.2	10.3	8.2	7.1	7.0	11.0			
Modified By Tidal Protection	9.6	9.4	8.0	7.1	7.0				
Zone 5C - Natural	11.2	10.3	9.2	8.4	8.0	8.3	-	9.4	
and 5D Modified By Tidal Protection	7.6	7.4	7.2	7.1	7.0				
Zone 6 - Natural	11.8	10.7	9.0	7.9	7.4	10.7		9.5	
Modified By Tidal Protection	7.6	7.3	7.2	7.1	7.0				

* Annual frequencies are based on new Revere Beach condition

** Experienced elevations are based on pre Revere Beach condition (no beach)

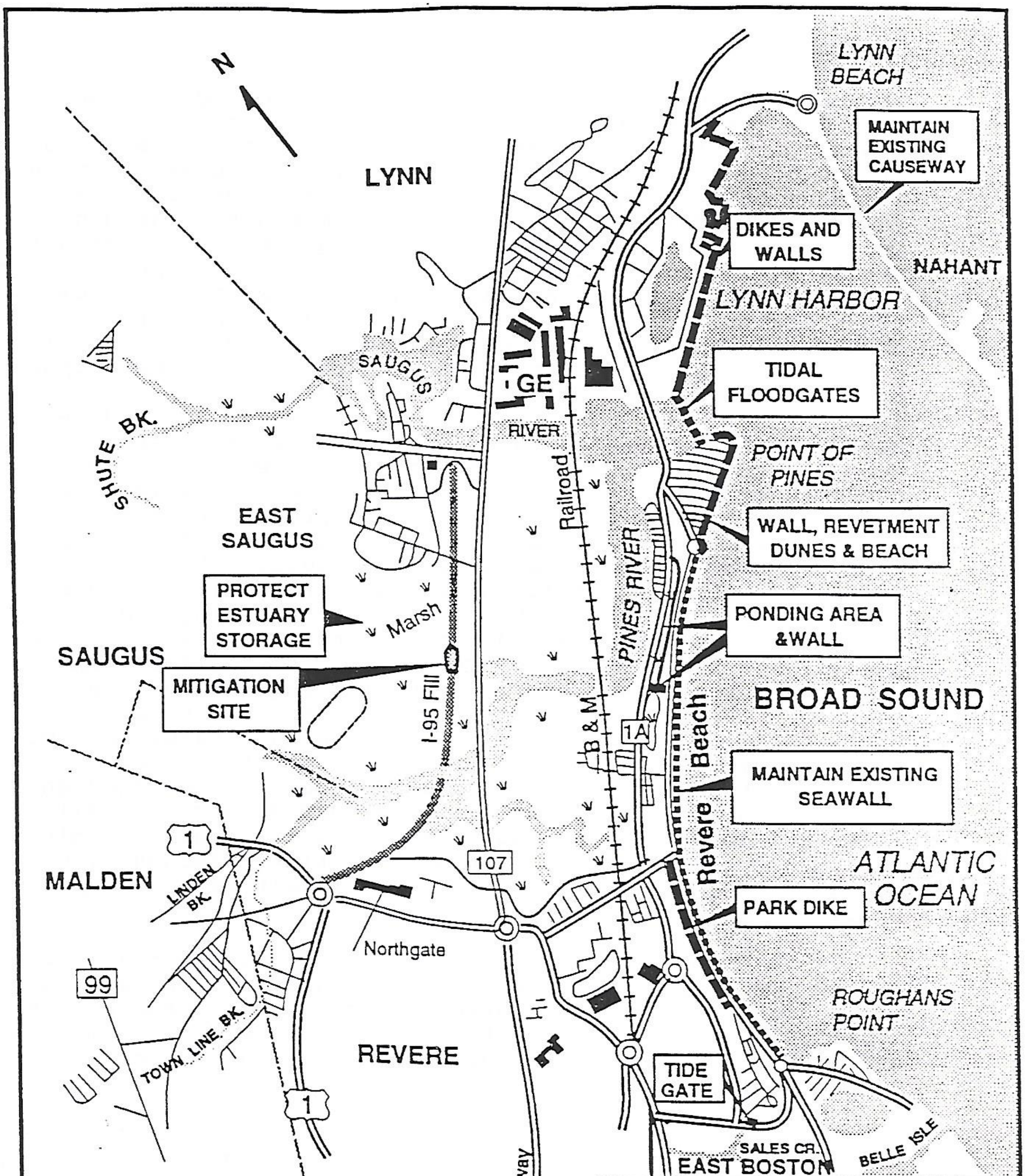
NOTE: Flood levels in Zones 1, 2A, 2B, 4A, 4B, and 5B are affected by the new Revere Beach.

TABLE 17 (cont) **LYNN, MASSACHUSETTS**

Location - Condition	Annual Frequencies (%) 0.2	1.0	10	50	90	Experienced 1978	1979	1987	Others
Boston Stillwater	11.2	10.3	9.2	8.4	8.0	10.3	9.3	9.4	9.1 (72) 8.9 (67)
Zone 1 - Natural	13.6	12.4	10.1	9.0	8.5	12.4	10.1	10.0±	
Modified By Tidal Protection	8.5	8.4	8.4	8.4	8.3				
Zone 2 - Natural	12.0	11.2	9.9	9.1	8.8	11.2	-	10.0	
Modified by Tidal Protection	8.6	8.4	8.4	8.4	8.3				
Zone 3 - Natural	11.2	10.3	9.2	8.4	8.0	10.3	9.3	9.6	
Modified By Tidal Protection	7.6	7.4	7.1	7.0	7.0				

SAUGUS, MASSACHUSETTS

Location - Condition	Annual Frequencies (%) 0.2	1.0	10	50	90	Experienced 1978	1979	1987	Others
Boston Stillwater	11.2	10.3	9.2	8.4	8.0	10.3	9.3	9.4	9.1 (72) 8.9 (67)
EAST SAUGUS									
Zone 1 - Natural	12.1	11.0	9.2	8.2	7.9	11.0	9.2	9.6	
Modified By Tidal Protection	7.6	7.3	7.1	7.0	7.0				
Zone 2 - Natural	11.9	10.7	8.3	6.0 (21%)	-	10.7	-	7.2	
Modified By Tidal Protection	6+	6-	-	-	-				
Zone 3 - Natural	11.7	10.5	8.6	7.0	6.4	10.5		8.5	
Modified By Tidal Protection	7.6	7.3	7.1	6.6	(6-)				



Saugus River and Tributaries Flood Damage Reduction Project

Lynn, Malden, Revere and Saugus, MA

US Army
Corps of Engineers
New England Division

August 1993

Figure 12

Section 11 contains a discussion of interior runoff design considerations.

c. Point of Pines Project Area, Revere--Zone 7. A separate Point of Pines Local Protection Project has been incrementally optimized by NED, and approved by HQUSACE on 24 June 1985. The hydrologic and economic justification was presented in the Detailed Project Report, dated October 1984. The Division Engineer approved the inclusion of Point of Pines with the Regional Floodgate Plan. Integration of the approved Point of Pines project with the floodgate project will affect economies to both projects. Because of the location of the floodgate, a section of the Lynn protection along the Saugus River, as well as portions of the wall required along reaches F and G (near the Yacht Club along the Saugus River), for the Point of Pines Project are not needed (see plate 10 for location of reaches). However, for the floodgate project, a revetment or wall would be required in reach F of Point of Pines. Currently the plan of improvements calls for sand dunes in reach E. The top of dune will be at elevation 18.2 feet NGVD and result in no overtopping for SPN conditions. Results of CERC studies indicate that the dunes will be stable and maintain their structural integrity, if maintained under SPN conditions (see Appendix III).

As the decision was made to include the separable Point of Pines local protection project into the floodgate project, a review of the previously developed and approved stage frequency curves for Point of Pines (reference: "Detailed Project Report, Point of Pines, Revere, MA," October 1984 Appendix A) was considered necessary. From indepth coastal studies and modelling conducted by the U.S. Waterways Experiment Station (reference: "Coastal Flooding, Roughans Point, Broad Sound, Lynn Harbor, the Saugus-Pines River System," T.R. CERC 86-8) stillwater tidal frequency elevations in the Saugus River have been refined and modified. This modification results in a change of previously developed elevation frequencies for the northern end of Point of Pines (zone 7-D, see plate 7). Current estimates of applicable natural stillwater elevations in the Saugus River adjacent to the Point of Pines area, together with previously developed elevation frequencies at zone 7-D follow. We note that the stage frequency numbering system has been changed from the DPR to this design document. Previously the Point of Pines stage frequencies were listed in Zones 1 through 4; this numbering system has been changed to Zone 7A through 7D.

elevation from the floodgate structure along reaches E (north end) and F, overtopping along these areas would be prevented. Therefore, under this condition, hydraulics of the interior area would be modified and future condition zone limits would be changed. Modified zone limits for zones 7-B, 7-C, and 7-D for the above described condition are shown in plate 8.

The results presented in this FDM are based on the combination of revetment and dune system. The current CERC results indicate a higher level of protection provided by a properly maintained total dune system as compared to a dune/revetment combination. Therefore, if further design studies are conducted and result in adopting a total dune system, the flood stage reductions attributable to a total dune system will be greater than those shown on plate 7 of this FDM, and, therefore, will provide greater benefits.

11. INTERIOR RUNOFF DURING STORM TIDES

a. General. With a Regional Saugus River Tidal Floodgate Project, probable modified flood level frequencies in the estuary would be a function of probable volume of inflow during periods of gate closure for storm tides. Consideration was given to this runoff in estimating probable modified interior flood level frequencies. However, potential interior runoff during storm tides was also considered in assessing project design capability and project operating procedures.

b. Probable Interior Runoff Coincident With Storm Tides. Methods for analyzing coincident interior runoff frequencies are presented in Corps of Engineers EC 1110-2-247, "Hydraulic Analysis of Interior Areas," dated 23 September 1983. If the occurrence of tidal and interior storm runoff events were completely independent, then an inflow duration curve (runoff rate versus percent of time) could be developed and this curve separated into a series of percent chance runoff events, i.e., 10 different runoff rates for each 10 percent time duration. The 10 different runoff rates could then be used to compute 10 different resulting interior storage levels, and these levels could be averaged to arrive at the most probable average interior storage level. For further discussion of this approach, reference is made to EC 1110-2-247.

The above method is most applicable to the analysis of probable coincidence of completely independent meteorological events. However, storm tides and rainfall-runoff are not entirely independent. Storm tides are generally associated with coastal storms and/or hurricanes with accompanying rainfall. Therefore, it would be expected that runoff-duration

- Sensitivity testing, using SBEACH and the Halloween storm data set, indicated that beach fill with median grain sizes above approximately 0.40 mm at Revere Beach and Point of Pines exhibit significantly higher erosive resistance compared to natural beach material (median grain size of 0.21 mm).

- Longshore movement appeared to be a dominant process distributing sediment along Revere Beach and Point of Pines, and aided in buffering the POP reach during the Halloween storm because of the predominant northerly transport at the site.

- Longshore variations in sediment transport during the Halloween storm limited calibration and verification of SBEACH, and analysis of cross-shore erosion along Revere Beach, was hindered due to an assumed longshore gradient for all storms tested. Uniformity of longshore transport seemed better satisfied at Point of Pines, allowing profile response simulations to be completed.

- Wave transformation in the lee of Nahant Peninsula is quite complex, and variations in wave height and direction have the most influence along the northern reach of Revere Beach in the vicinity of Carey Circle.

- The existing Revere Beach coarse-grained beach fill appears highly effective in mitigating overtopping of seawalls relative to pre-fill conditions.

- Bore runup overtopping conditions are dominant along Revere Beach for present profile conditions, with water levels associated with the storm data base below the beach/seawall intersection elevations; bore runup conditions greatly reduce predicted overtopping volumes for the storm set.

- Broken wave overtopping conditions are dominant along Revere Beach for pre-fill profile conditions, with water levels associated with the storm data base above the beach/seawall intersection elevations; broken wave conditions greatly increase predicted overtopping volumes for the storm set relative to bore runup conditions.

- Dune optimization indicated that properly maintained dunes, with a crest width of 30 feet or greater and crest elevation of 16.5 feet NGVD (21 feet MLW), (18.5 feet NGVD near Carey Circle, profile 6) or greater at Point of Pines, are extremely resistant to erosion and overtopping associated with severe storms, including the SPN.

- The 1 on 3 sloping revetment design evaluated at Point of Pines, with a crest elevation of 16.0 feet NGVD appeared effective in mitigating overtopping for all but the most extreme storms when fronted with a coarse-grained beach fill with a berm elevation of 6.0 feet NGVD.

- In the absence of protective beach fill, the revetment design evaluated at Point of Pines with a crest elevation of 16 feet NGVD, appeared effective in mitigating overtopping for only high frequency storms.

- Post-Halloween storm (November 1991) profiles along Revere Beach maintained a high level of flood protection according to simulations using post-storm profile data and the Halloween storm, which is indicative of the erosive resistance of the coarse-grained fill.

Consideration should be given to construction of a full dune system of constant elevation at POP. Assuming a design storm, with a 100-year return period and possibly higher, it is evident from results that design dunes with coarse-grain size (0.49 mm) are resistant and would experience minor damages. Additionally, total project site analyses indicate a buffering of the beach at POP with introduction of material from the updrift beaches, and lack of substantial offshore movement associated with the typical severe storm events as indicated by profile response simulations. The dune system at POP has historically proven effective, but it remains totally dependent upon the condition to where it is maintained; results discussed herein are likely negated by failure to sustain a beach/dune at or near design conditions.

Further study, based on results of the CERC studies at Point of Pines, recommended evaluation of dune versus revetment options. Evaluation should include cost comparison, level of protection, confidence in design, maintenance requirements, environmental, aesthetic, public acceptability, real estate, etc. Results of this evaluation and final selection of protection will be presented in future design studies.

(3) <u>Hydraulic Design of Park Dike</u>

(a) <u>General</u>. To protect against flood damage, a dike was proposed for construction on the west side of Revere Boulevard along a reach of Revere Beach, as local residents were interested in developing the dike as a city park. Detailed physical model studies were conducted at CERC for the design of Park Dike, and are documented in detail in Appendix IV. Following is a general overview of the study and findings.

(b) <u>Scope of the Park Dike Study</u>. Overtopping values needed to design the dike could not be accurately estimated due to the geometric complexities of the beach, seawall, Revere Boulevard toe wall at the "Park" Dike, and the dike. CERC was therefore requested to determine overtopping rates for the proposed dike through physical model tests. To assist NED in determining the level of non-Federal funding, CERC was also tasked with determining a minimum configuration, rubble-mound structure that would provide for flood control caused from overtopping of the existing seawall. The study included determination of overtopping rates for the proposed park dike, as well as the minimum sized rubble-mound dike when fronted by profiles from both the 1978 and 1991 surveys, and subjected to severe storm conditions (SPN).

(c) <u>Revere Beach Conclusions</u>. The proposed park dike, with the crest lowered to provide only a 2.5-foot rise from toe wall to crest, was found to be sufficient to prevent nearly all overtopping during the design storm event with the 1991 beach bathymetry. Revere Boulevard, of course, would be completely flooded. Waves overtopping the seawall and crossing the boulevard would flow part way up the park dike in a solid sheet of water across the width of the flume. As runup decreased, the sheet of water would be reduced to a few "fingers" or thin streams of water flowing much further up the slope of the dike. At no time did the solid sheet of runup reach the crest. It is anticipated that prototype runup on a park dike, covered with vegetation and paths, would be less than observed in the wave flume. If the profile in front of the seawall remains at a bathymetry similar to the 1991 survey, it appears that the park dike, with a crest elevation at a minimum of plus 19.5 feet NGVD (24 feet MLW), should be adequate to prevent nearly all overtopping during the design storm event. It should be recognized, however, that in any random sea event there is a possibility of an event occurring that exceeds the conditions tested in the physical model. During development of the embankment FDM, the height requirements of Park Dike, which currently has an elevation of 23.0 feet NGVD, will be further evaluated.

* maintain Revere Beach design profile

Decreasing the elevation of the ocean fronting seawall lowered the rate of overtopping over the park dike. With the toe wall maintained at a constant elevation, lowering the freeboard of the seawall increased the freeboard of the toe wall over the seawall by the same amount, thereby, increasing the effectiveness of the toe wall. Although there was more overtopping of the lower seawall, the increased effectiveness of the toe wall resulted in less water overtopping the dike. Tests conducted to determine effects of a failure of the seawall, to elevation of the roadway, found overtopping rates lower than with the seawall in place.

In lieu of the recreational park dike, a small rubble mound with an impervious core was found effective in preventing overtopping when tested with the 1991 beach profile. Roughness of the stone structure quickly halted the runup, and a much smaller structure than the park dike was found to be sufficient. With a crest elevation of plus 20.9 feet NGVD (25.4 feet MLW), there was no overtopping during design storm conditions, with the exception of a minor quantity of splashing.

Overtopping was observed on tests for both park dike and the rubble mound structures when beach erosion occurred in front of the seawall, with increased stillwater level from sea level rise, or on tests with the 1978 beach profile.

Minor displacement of armor stones occurred on the rubble-mound structures with stones averaging 337 lbs. during tests with the 1978 beach profile. Because wave action on the dike with the 1991 profile was less than the 1978 one, and assuming that stones would be seated during storms of less severity than the design event, the 337-lb stones are considered sufficient if the 1991 beach profile is maintained.

If the beach profile is not maintained and returns to a bathymetry similar to the 1978 profile, then both park dike and rubble-mound dike will be overtopped during the design storm event, and under less extreme conditions. Because sea conditions varied for the various tests conducted under this research effort, it is difficult to compare overtopping rates for the different profiles and structure options at Revere Beach. However, the following comparison, based on conditions at the peak of the SPN, may be instructive.

Table 19 lists several tests of profile 2, tested at the SPN peak with a stillwater of plus 12.0 feet NGVD, and a wave period of 15.9 seconds. Both 1978 and 1991 profiles are included, as are both park dike and rubble-mound dike (19.5 and 20.9 feet NGVD, respectively) as well as overtopping rates without a dike.

By far the highest overtopping rate was found with the 1978 profile and no dike. With the addition of beach fill (1991 profile), the overtopping rate was reduced by about 65 percent, even with a wave height that was one-half again as high as the 1978 profile. The addition of overtopping flow is predominantly laminar once it crosses Revere Beach Boulevard.

d. Design of Saugus River Tidal Floodgate

(1) Background. The Saugus-Pines River System (figure 15) is located approximately six miles north of Boston Harbor. The Saugus River, and its Pines River tributary, have a watershed area of approximately 47 square miles. The Saugus River originates about 13 miles inland, and flows in a southeasterly direction before discharging into Broad Sound, between the cities of Lynn and Revere. The downstream 4.7 miles of the Saugus River is tidal. This portion of the system, including the Pines River system, is comprised of multiple storm-drained coastal urban areas and extensive tidal marshland. The entire three miles of the Pines River system is tidal. The rivers have a relatively small watershed; therefore, freshwater discharge is low in comparison to tidal flows. The river systems are classified as well mixed. Several cities and towns, including Revere, Lynn, Saugus, and surrounding areas are subjected to periodic flooding from strong northeasterly storms and hurricanes.

The New England Division, Corps of Engineers (CENED) was authorized to develop a plan consisting of seawall and revetment construction, sand dune reconstruction, and a Saugus River tidal floodgate and protection/acquisition of the estuary flood storage to protect these communities from periodic flooding. The central feature of the protection scheme is a floodgate structure in the entrance of the Saugus-Pines River system.

The plan is to construct a floodgate structure at the confluence of the Pines and Saugus Rivers near Broad Sound. The structure is to be located about 600 feet seaward of the existing General Edwards Bridge, just beyond the Point of Pines Yacht Club. The floodgate will be constructed to maintain safe navigation for commercial and recreational vessels, natural tide levels, and estuary flushing patterns. During storm events that are expected to cause significant damage, the floodgates will be closed. All other times they will remain open.

The U.S. Army Engineer Waterways Experiment Station (WES) was requested to assist CENED in design of the structure. Two types of models, physical and numerical, were used throughout the course of the investigation. A three-part model study was developed. Part one employed a global numerical model using the USACE TABS-MD modeling system. The numerical report is located in Appendix XV. This model was used to evaluate overall effects on circulation, tide levels, and sedimentation. It was also utilized to generate boundary conditions for the physical model, the second part of the overall study. The physical model was used to obtain detailed information on current velocities and current patterns in the immediate vicinity of the proposed structure, for use in the structure design and the numerical model study. The physical model report is located in Appendix VI. The third part, a navigation study using the physical model, evaluated navigation safety for several proposed plans and also for various construction stages of the structure. The navigation model report is located in Appendix VII. Current velocity data generated from the physical model were used in the riprap design analysis. More detail on the models is discussed in the following sections.

(2) Objective. The model study is to provide the most cost effective and environmentally sound design configuration for the Saugus River floodgates. Cost effectiveness is defined in this study minimizing the number of gates necessary to provide safe and effective operation of the structure. Evaluation focused on impacts of the structure on: (a) basin water surface elevations, (b) water circulation, (c) current velocities, (d) sedimentation and erosion, and (e) navigation. In addition, water surface elevations and current velocities resulting from a 1.0-foot rise in sea level are discussed.

(3) Navigation Model. The complete navigation model report is included at Appendix VII. Following is a broad summary of study and findings:

(a) Need and Purpose of Model Study. Although preliminary design of the proposed floodgate structure, presented in the feasibility study, was based on sound theoretical design practice and experience, navigation conditions through the navigation gate of the structure could be expected to be complex. As navigation conditions vary with location and flow through the structure, it was necessary to perform model studies that could account for these effects. Therefore, comprehensive numerical and physical model studies were considered necessary to:

- Determine effects of the proposed structure on navigation through the study reach.

- Develop modifications to design of the structure that could improve navigation.

- Develop the project construction sequence that would provide satisfactory navigation conditions for small crafts.

- A system of current vanes, or some device to indicate current direction and magnitude to boaters, may improve the way mariners might interpret and react to flow conditions. This would be particularly helpful during the construction of the project. Detailed design of aids to navigation for construction and permanent features will be further coordinated with the U.S. Coast Guard and local marine interests as well.

(4) Numerical Model. Following is a broad summary of numerical modelling for the proposed floodgate. The detailed numerical report is included in Appendix V.

(a) General Objectives and Approach. The numerical model study was conducted to examine far field effects of the plan in particular, to provide flood damage reduction against the Standard Project Northeaster (SPN) event. The principal component of this plan is construction of tidal floodgates at the mouth of the Saugus River. These floodgates prevent tidal surges from entering the river; thereby, reducing flooding within the study area. The floodgates will be constructed to maintain both safe navigation and natural tide levels and flushing patterns in the estuary under normal conditions. The gates will be closed only when projected tide levels are expected to cause significant damage.

A field investigation by the New England Division showed that the **phragmites** reed, which indicates deterioration in saltwater wetlands, may be expanding in the northwest corner of the Pines River marsh. Due to Federal, State, and local interest in preserving and restoring this wetland by breaching the abandoned I-95 embankment (figure 22), a breaching plan was developed by NED to restore degraded wetlands and increase tide levels, resulting in increased flushing of nearly 500 acres. The I-95 embankment is an abandoned highway fill remaining from uncompleted roadway construction activity. The plan includes breaching the I-95 embankment at the east branch of the Pines River, and widening the existing river opening in the I-95 embankment. Although the Corps has no present plans to implement this breaching scenario, the effects of this possible undertaking by others in the future must be considered when designing the proposed tidal floodgates located at the mouth of the Saugus River.

problems. However, to quantify this assessment, a sensitivity study, using a sediment transport model, was performed to evaluate shoaling potential.

(b) Conclusions. The RMA-2V model was successfully verified to limited field measurements, including a 14-hour field survey of water levels and velocity measurements. Comparisons of computed water levels and velocities to field measurements were good. At locations in some marsh areas, computed water levels were slightly higher by about 0.2 foot and the ebb tide fell about 1 hour faster than the field data. The model error was suspected to be the result of difficulties in properly representing the storage of water in marshes, and interconnected mosquito ditches. In the Pines River, velocities were underpredicted by about 0.3 fps for peak flood and overpredicted about 1.0 fps for peak ebb. These variations were caused by the elements that are removed when dry and added when wet again. These differences were small and not expected to significantly impact the use of model results for the intended purposes.

The study provided boundary conditions for the physical model study under existing and base conditions for both spring and neap tides (existing and base conditions are described in section 12d. For existing spring tide conditions, flows used for the physical model study were 29,000 and 22,000 cfs for the flood and ebb tides, respectively. The corresponding water levels for these flows were 3.48 and 4.81 feet NGVD for the flood and ebb tides, respectively. For existing neap tide conditions, flows were 12,300 and 11,300 cfs for flood and ebb tides, respectively. The corresponding water levels were 1.46 feet and -0.30 foot, respectively. For base spring tide conditions, flows were 27,100 and 28,200 cfs for flood and ebb tides, respectively. The corresponding water levels were 6.24 and 2.70 feet, respectively. For the base neap tide conditions, flows were 11,900 and 10,900 cfs for flood and ebb tides, respectively. The corresponding water levels were 1.46 feet and -0.31 foot, respectively.

Breaching the abandoned I-95 embankment and widening the Pines River opening on I-95 will increase tidal flow in marsh areas upstream from the I-95 embankment. Water levels in marsh areas will increase about 0.5 foot at peak tide under a spring tide condition. Time lag of peak water levels, between Broad Sound and the upper marsh areas beyond the I-95 embankment, was reduced from 2 hours to 1 hour.

Plan 2C+7, described in Appendix VII under description of alternatives evaluated in the navigation model, will not cause measurable change of water levels in the Pines and Saugus Rivers under normal tide conditions. It will protect study areas from flooding during storm events.

Water levels in the marsh areas under Plan 2C+7 will increase about 1.0 foot at peak flood and ebb tides for the 1-foot rise in sea level.

The proposed floodgate will not alter sediment deposition or scour pattern in the estuary under the normal tide condition, but local scour near the floodgate piers may occur.

(5) Physical Model. Following is a broad summary of the physical model study. Detailed model report is included at Appendix VI.

(a) General Objectives and Description. The primary objective of the physical model was to provide detailed hydrodynamic information for the structure design, and detailed water surface elevation and current velocity data on the three-dimensional flow field in the immediate vicinity of the structure. This information could be used in the numerical model part of the study, and in design of riprap protection. The physical model was also used to conduct the navigation study.

The limits of the physical model are shown in figure 23. The model was approximately 110 feet long and 80 feet wide at the widest point (about 4,500 square feet). It reproduced an area beginning about 600 feet south of Point of Pines; encompassing a portion of Lynn Harbor to a point on the north shore about 2,000 feet northeast of the General Edwards Bridge. Upstream limits of the model extended from the confluence of the Saugus and Pines Rivers, approximately 1,200 feet upstream from the bridge. Installed in the model to model scale were piers of the General Edwards Bridge and Point of Pines Yacht Club, the MDC public fishing pier (removed for the plan testing), the General Electric pipeline pier, and the pier on the south shore, immediately upstream from the General Edwards Bridge. The model was constructed to an undistorted scale of 1:50.

(b) Discussion of Test Results. The final design configuration (number and location of floodgates, and dredged approach areas) was determined during the course of the navigation study. Safe navigation, in conjunction with cost effective and environmental sound design, was the primary element in selection of the final plan. The details of each plan investigated, involving navigation safety, are addressed in the navigation study report (Appendix VII).

Plan 2C+7, recommended as the best in respect to navigation safety, meets the other objectives and was selected as the final plan.

This report will not address individual data generated by the many plans investigated throughout the physical model investigation. However, all hydraulic data, collected during the course of the physical model study, are presented in the form of tables and figures. The discussion of model test results, for the purpose of this report, will concentrate on the recommended scenario, Plan 2C+7. Two different scenarios, the base spring tide and existing spring tide conditions, as described in Section 12d(3)(b), were first evaluated. The only difference is that the I-95 embankment is breached under the base condition and not under the existing one. An evaluation of the project with an additional 1 foot of sea level rise is also discussed.

Under base spring tide conditions, water surface elevations were influenced very little by installation of Plan 2C+7. The difference in water surface elevations during floodflows from station 3 (headbay A) to station 1 (Pines River) was 0.08 foot for base conditions (see figure 23 for location of specific stations and gates). With Plan 2C+7 installed, this difference increased to 0.15 foot, for an increase in head loss of 0.07 foot. The difference in water surface elevations for ebb flow conditions from station 1 to station 3 was 0.07 foot, and 0.10 foot with Plan 2C+7 in place, or an increase in head loss of 0.03 foot. The head loss across the structure (ranges 1.43 to 2.44) during floodflows varied from a minimum of 0.00 foot at stations A and B to a maximum of 0.08 foot at stations C, D and F. Head loss during ebb flow varied from 0.00 foot at station A to 0.05 foot at station F. Head loss at other stations varied from 0.02 to 0.03 foot. It is important to note that these losses are at maximum flow. During slack water at high and low tides, differences between base and plan would be nearly impossible to measure.

Current velocities under base spring tide conditions, were influenced to a greater degree than water surface elevations. The largest increase in current velocity during plan floodflow conditions occurred at gate A (stations 2.43A and 2.44A). At station 2.43A depth averaged current velocity magnitude increased from 1.2 to 2.1 fps, and from 1.4 to 2.4 fps at station 2.44A. In the navigation opening, gate B, depth-averaged flood direction current velocities increased from 1.3 to about 1.8 fps at station 2.43B, and from 1.5 to 2.3 fps at station 2.44B. Overall, Plan 2C+7 resulted in an increase in velocity magnitude both oceanward and landward of the structure. Exceptions to this general trend, as expected, were noted at the four northern-most stations where the access roadway embankment had been constructed.

Current velocity for base spring tide condition changes, noted during ebb flow conditions, were generally less than observed during flood flow testing. The general trend of increased velocity magnitude was again noted at stations adjacent to the gate openings, likewise stations located adjacent to the north roadway embankment showed a decrease. The largest increase in current velocity during ebb flow occurred at station 2.43C, where current velocity was increased from 1.6 to 2.6 fps with the plan installed. There was little change noted at station 2.44C. Depth averaged current velocities in the navigation channel increased at both stations 2.43B and 2.44B, by 0.8 and 0.2 fps, respectively. Current velocity magnitude at stations H through K were generally reduced about 1.0 fps to 1.5.

Under the existing spring tide conditions, water surface elevations were changed very little by Plan 2C+7. Head loss from headbay A (station 3) to the Pines River (station 1) during floodflow conditions varied from 0.17 foot, with existing conditions, to 0.15 foot, with Plan 2C+7 installed. Head loss during ebb flow conditions from stations 1 to 3 changed from 0.10 to 0.15 foot with Plan 2C+7 in place. Differences in water surface elevations for existing conditions and with Plan 2C+7 installed, were less than 0.10 foot at other monitored stations. Again, it should be noted that these differences are during peak flow periods.

Depth averaged current velocities for existing spring flood conditions at stations 2.43A and 2.43B (navigation channel) were increased by 0.5 and 0.2 fps, respectively, with the plan installed. Depth averaged current velocities on range 2.43, stations C,D,E, F, and G each resulted in decreased velocities. The maximum decrease of 0.6 fps occurred at station 2.43E. Flood direction depth averaged current velocities on the landward side of the structure (range 2.44) were generally increased by the plan. The greatest increases, (0.9 fps) were observed at stations A and E. Current velocity through the navigation channel (station B) were increased 0.5 fps with the plan installed.

Ebb flow depth averaged current velocities for the existing spring tide condition, followed the same general trend as was observed for floodflow conditions. Current velocities on range 2.43 (oceanward of structure), stations A, B, C, and F were increased by the plan. There

d. <u>Interior Storage Capacity</u>. A necessary component of the project is preservation of adequate storage capacity, in the tidal estuary, for storage of design interior runoff volume and residual tidal overtopping, while the floodgates are closed. As set forth in ER 1105-2-20, "Project Purpose Planning Guidance," local assurances would be needed to protect the Federal investment and assure the achievement of expected project benefits throughout project life. Accordingly, ample estuary lands for interior storage capacity would need to be preserved. Design criteria for the interior storage capacity assumes that a 1 percent (100-year) interior runoff event is coincident with an SPN tidal event.

During the design phase, detailed mapping of the tidal estuary, with topographic overlays of elevations 6, 7, and 8 feet NGVD was undertaken. These maps were used to further refine the area-capacity curves that were developed for the feasibility report. Developed area-capacity curves for the Saugus River Basin estuary are shown on plate 2. This relationship was also used in the HEC-1 routings. As can be seen on the area-capacity curve, areas at elevations 6 and 7 feet NGVD are 1,600 and 1,710 acres, respectively. This is approximately 540 and 270 acres greater than reported in the feasibility report, which is due to more detailed mapping. This increased area results in more volume of available storage than previously estimated.

There presently exists sufficient estuary area for safe storage of interior runoff during periods of floodgate closure, eliminating the need for a costly pumping station. However, assurances of functional integrity of such storage is critically important as stated in EC 1110-2-247, "Hydrologic Analysis of Interior Areas," dated January 1984. It was determined, using the refined area-capacity curves for the Saugus-Pines River estuary, preserving all lands, at and below +6.0 feet NGVD in the estuary, would assure 6,190 acre-feet of storage capacity between +2.0 and +8.0 feet NGVD, assuming a vertical rise between elevations 6.0 and 8.0. Elevation 8.0 has been determined as the start of tangible flood damages. The required lands for flood control and storage capacity concepts are pictured on figure 25. Plate 13 shows the area of estuary guide taking line (elevation 6) and the area to the start of tangible flood damage (elevation 8). A storage capacity of 6,190 acre-feet is approximately 30 percent greater than the required capacity shown for design conditions, and represents a considerable "factor of safety" in the event of sea level rise and/or significant erosion at Revere Beach. Such beach erosion would affect wave overtopping volumes. It was judged impractical to consider a guide-taking elevation less than 6.0 feet NGVD, because elevation 5.0 represents only tidal ditches, and increments less than a foot (i.e., 5.6 feet NGVD) were eliminated based on judgement and workability issues. Local assurances would need to stipulate that no filling (loss of storage) be permitted below elevation 6.0 feet NGVD, without compensating storage between elevations 5.0 and 6.0 feet NGVD. Executive Order 11988 requires the Corps to avoid inducing development in the base flood plain if there is an alternative (ER 1105-2-20). In developing areas, the local sponsor is required, to the extent legally empowered, to adopt flood plain management programs to ensure wise use of the flood plains in, as well as adjacent to, the project area (ER 1105-2-20). To limit potential flood-prone development and adverse impacts on project operation, local assurances should stipulate that any new developments, bordering directly on the estuary, require minimum lot elevations not less than +8.5 feet NGVD, with first floor grades not less than +9.0 feet NGVD. Although elevation 6 has been adopted as the minimum elevation needed to assure sufficient land is available for flood control, other issues have to be considered before adopting a final guide taking elevation. Real Estate personnel have expressed concerns relative to workability issues of elevation 6 and have suggested elevation 7 is a more practical guide taking elevation. Elevation 7 was also reported in the Feasibility Study and may result in a faster permitting process. Adopting elevation 7, would result in slightly more available storage than elevation 6, and Water Control Division has no objections.

e. <u>Residual Interior Drainage Needs</u>. In addition to, but separate from the Saugus and Pines Rivers estuary, drainage in localized low areas in Revere and Saugus is presently dependent on localized ponding and flap-gated drains to the estuary. The tidal floodgate project cannot be operated to provide continuous gravity drainage for these low areas and some residual local drainage problems may persist in the absence of tidal flooding. Local assurances will need to stipulate that all existing tide gate structures to the estuary be maintained in good operating condition. Further, all proposed new developments must be reviewed with regards to their potential impact on existing drainage conditions.

All needed improvements for handling interior storm drainage, in the absence of tidal flooding, will be a non-Federal responsibility in accordance with ER 1165-2-21, "Flood Damage Reduction Measures in Urban Areas," dated October 1980.

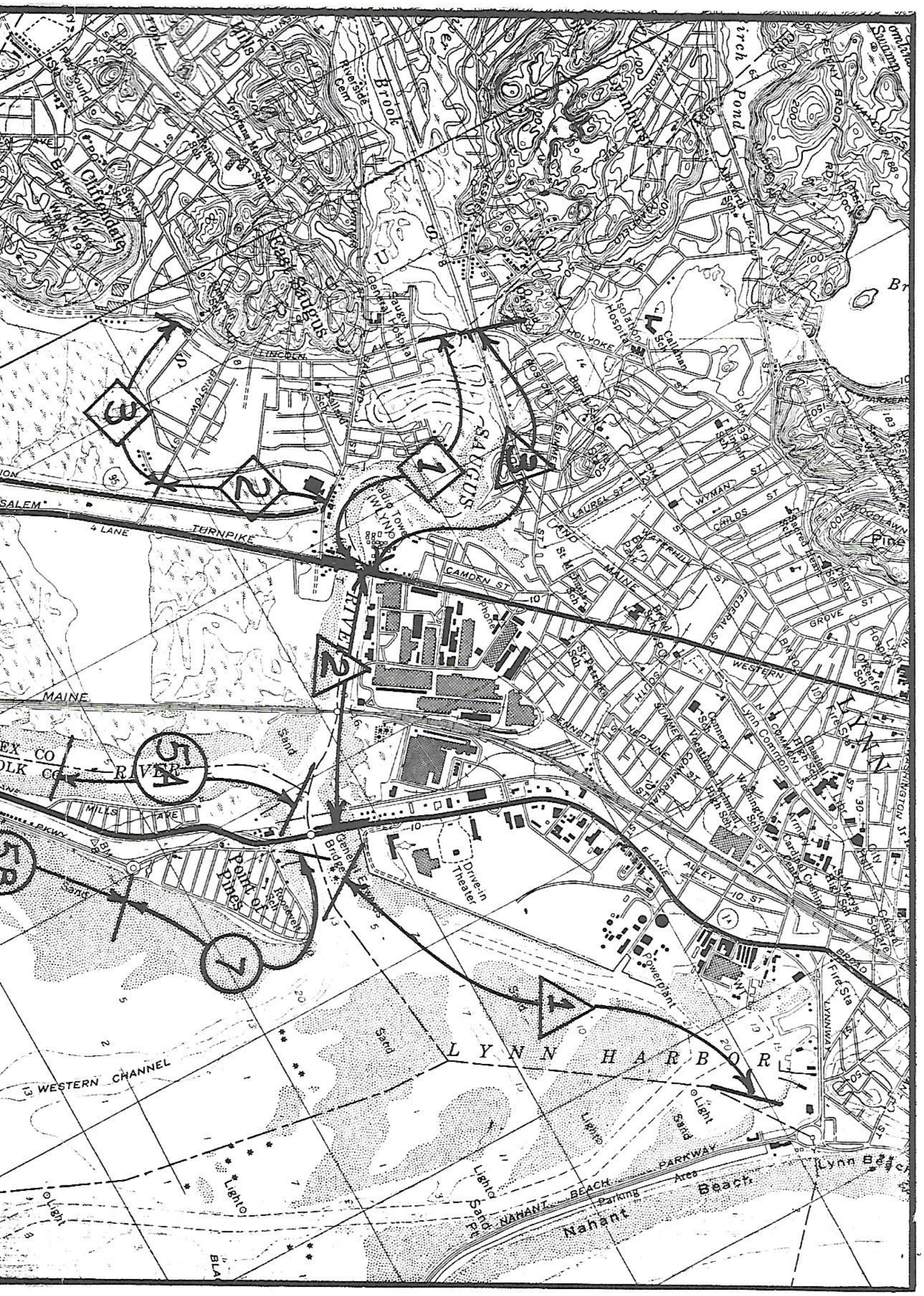
f. <u>Design Height of Protection Criteria</u>. See section on Tidal Hydraulics.

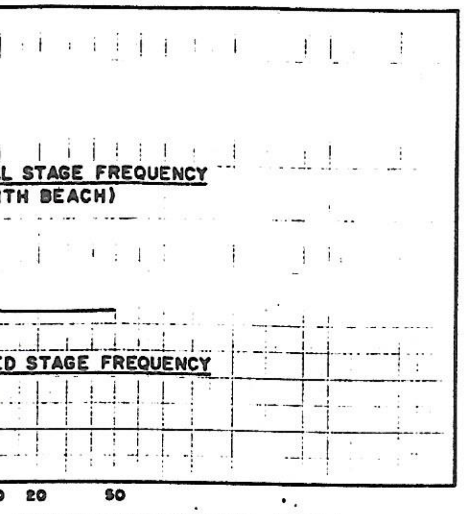
PLATE 4

SAUGUS RIVER BASIN
AND VICINITY

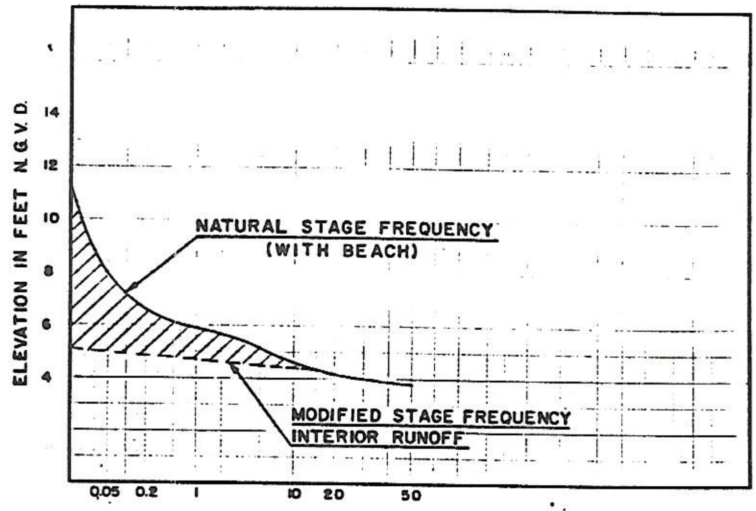
TIDAL FLOOD ZONES
REVERE, LYNN &
SAUGUS MASS.

HYDRO.ENGR. MAR. 1993

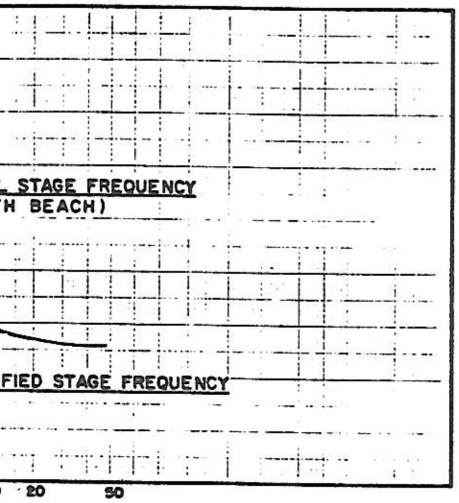

East Saugus
Cliftondale
SAUGUS
TURNPIKE
4 LANE
MAINE
RIVER
Point of Pines
Drive-in Theater
Powerplant
LYNN HARBOR
WESTERN CHANNEL
Lynn Common
LYNN
Nahant Beach
NAHANT BEACH PARKWAY
Parking Area
Lynn Beach
Sand
Light
CAMDEN ST
LAUREL ST
WYMAN ST
CHILDS ST
GROVE ST
WESTERN
HOLYOKE
FEDERAL ST
BROAD
10 ST
6 LANE
General Edwards Bridge
Isolation Hospital
Radio Tower (WLYN)
Saugus General Hospital
Riverside Cem
Pond



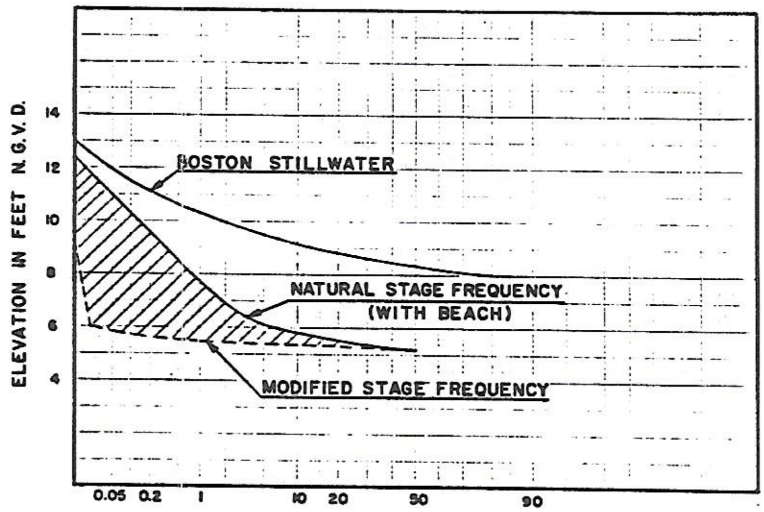
CREEK DRAINAGE AREA
NE NO. 2A- REVERE
STATION-OCEAN AVE. AREA



WONDERLAND PARK AREA
ZONE NO. 2B - REVERE



ND CREEK DRAINAGE
LAND & VICINITY AREA
NE NO. 4B-REVERE



PINES RIVER DRAINAGE
OAK ISLAND & VICINITY
ZONE NO. 4C - REVERE

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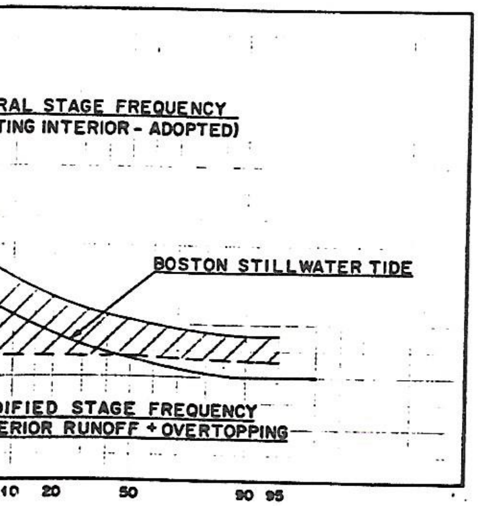
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION STUDY

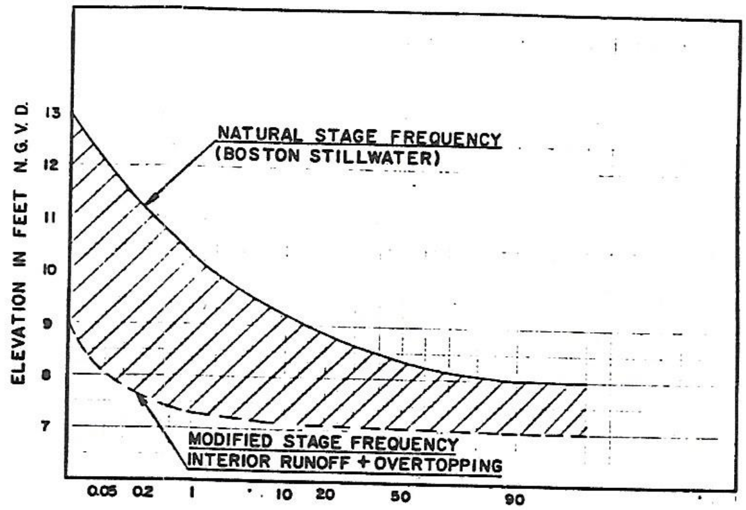
NATURAL AND MODIFIED
STAGE FREQUENCY CURVES

HYDRO. ENG. BR. JUN. 1993

PLATE 5

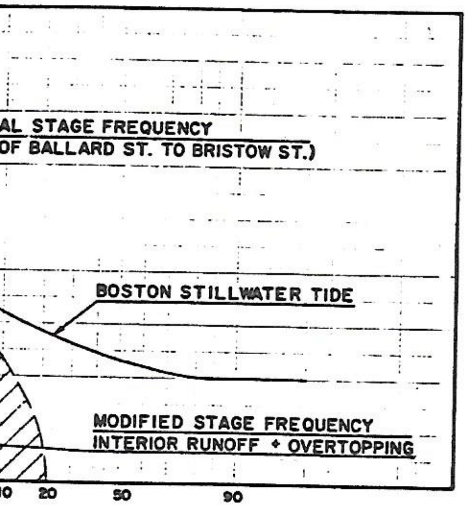


ERAL ELECTRIC AREA ZONE NO.2

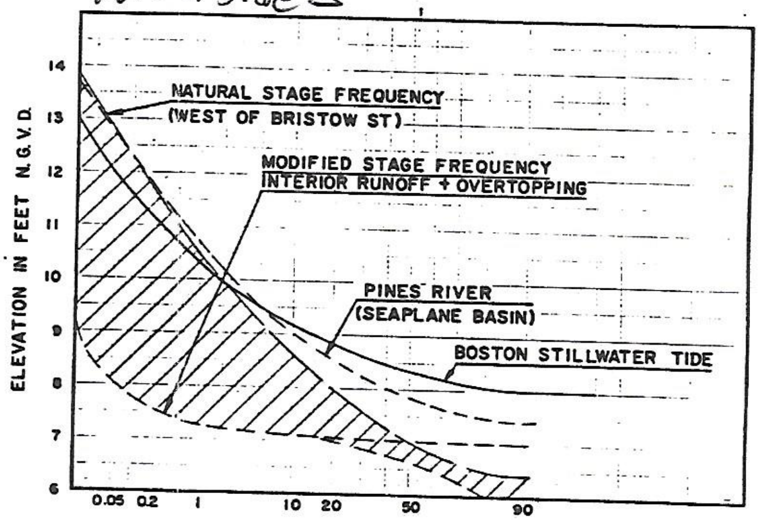


APPLICABLE FOR ZONES
NO. 3 LYNN
NOS. 5A, 5C, 5D REVERE

Riverside



SAUGUS ZONE NO. 2



EAST SAUGUS ZONE NO.3

Does not reflect
sea level rise
since '90 ±

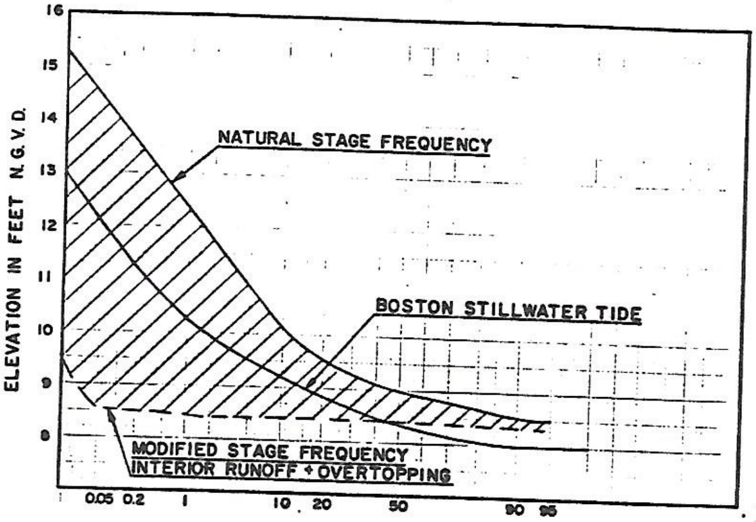
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION STUDY

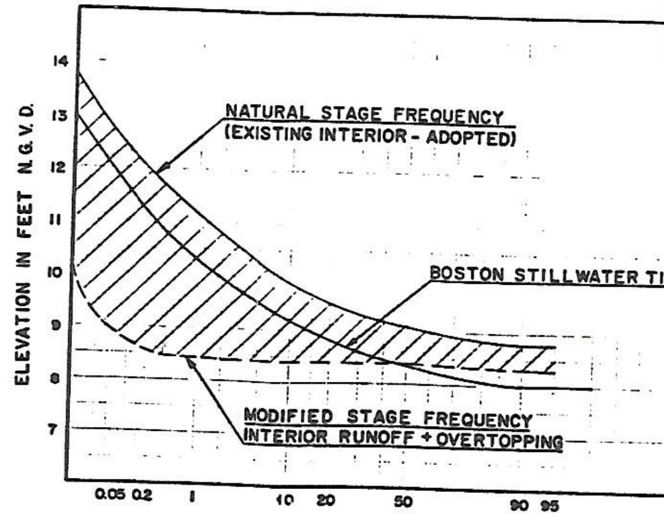
NATURAL AND MODIFIED
STAGE FREQUENCY CURVES

HYDRO. ENG. BR. FEB. 1993

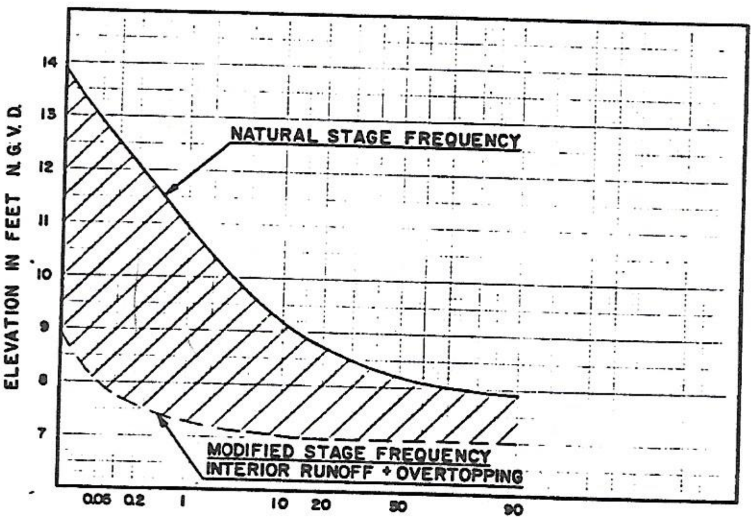
PLATE 6



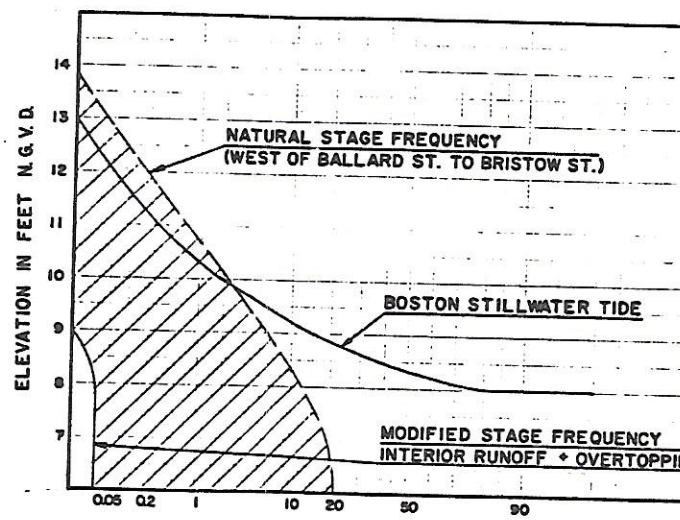
LYNN HARBOR COASTAL AREA ZONE NO. 1



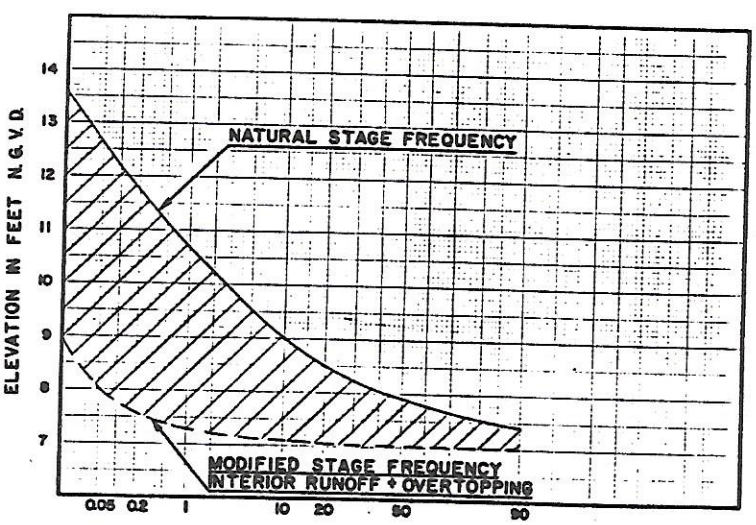
LYNN GENERAL ELECTRIC AREA ZONE N



EAST SAUGUS ZONE NO. 1

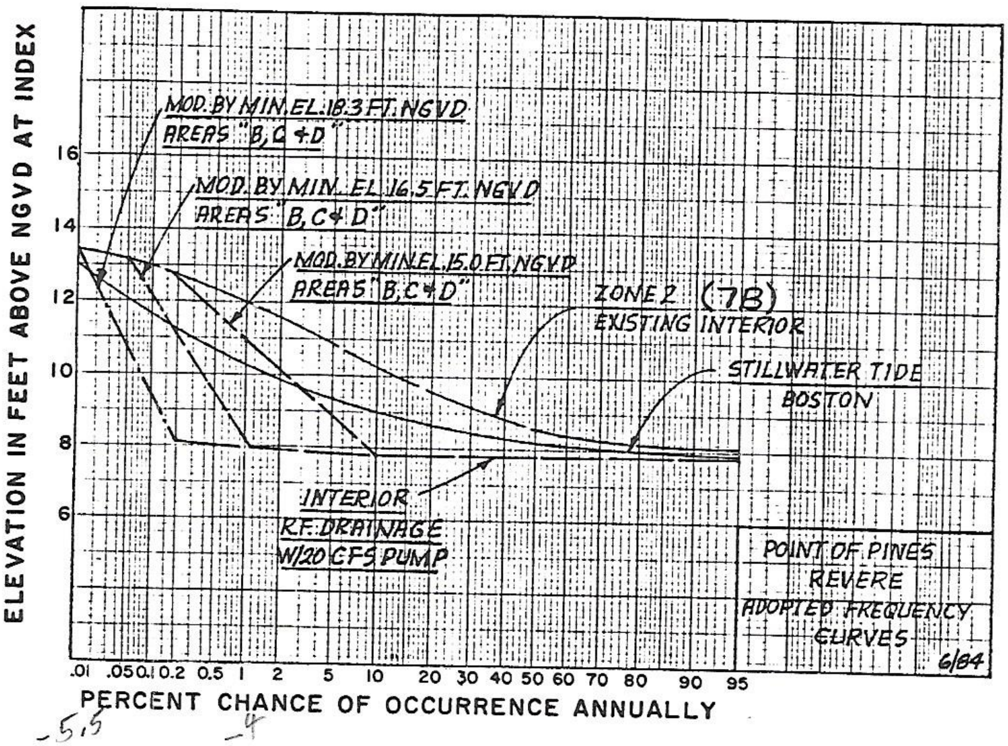


EAST SAUGUS ZONE NO. 2

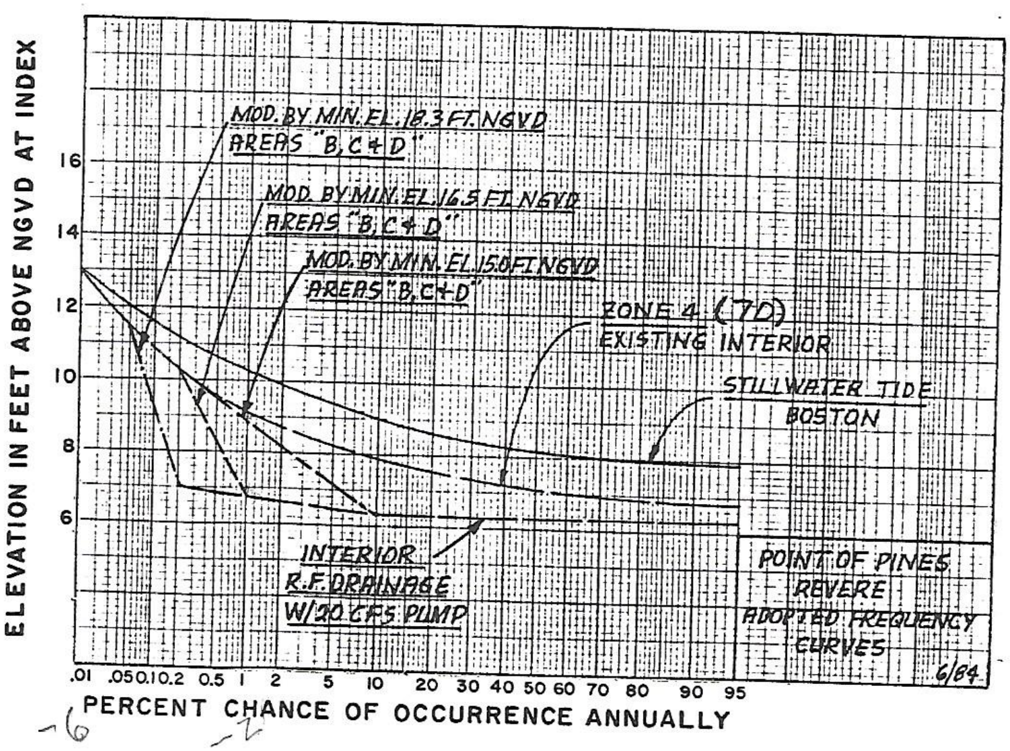


REVERE ZONE NO. 6

Does not reflect
sea level rise
since developed
1984-1990



#1 The min. EL's of 15.0, 16.5 & 18.3 reflects various top elev. of stone revetments. If a Total Dune system is selected, the Modified levels would be much lower, since no overtopping would occur even for the SPN design storm

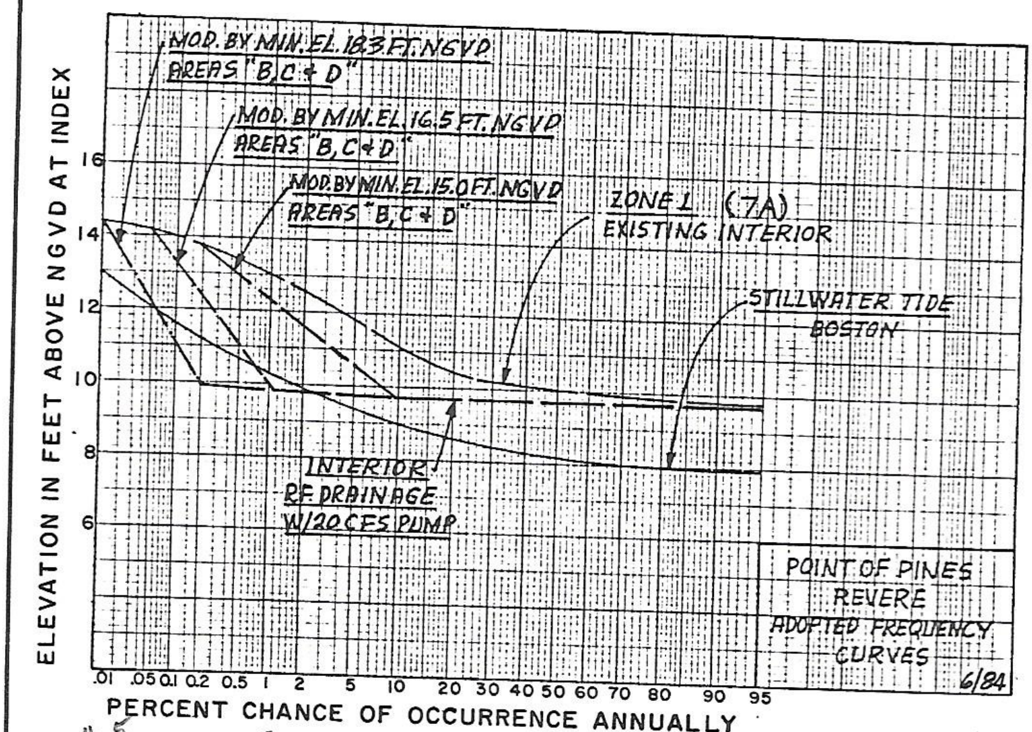


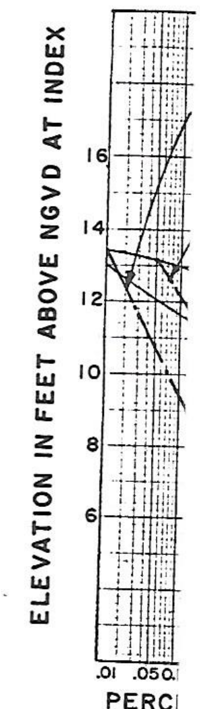
#2 Does not reflect sea level rise since developed 1984-1990 ±

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

COASTAL FLOOD PROTECTION
REVERE, MASSACHUSETTS
POINT OF PINES
(MODIFIED AND ADOPTED)
ELEVATION FREQUENCY CURVES

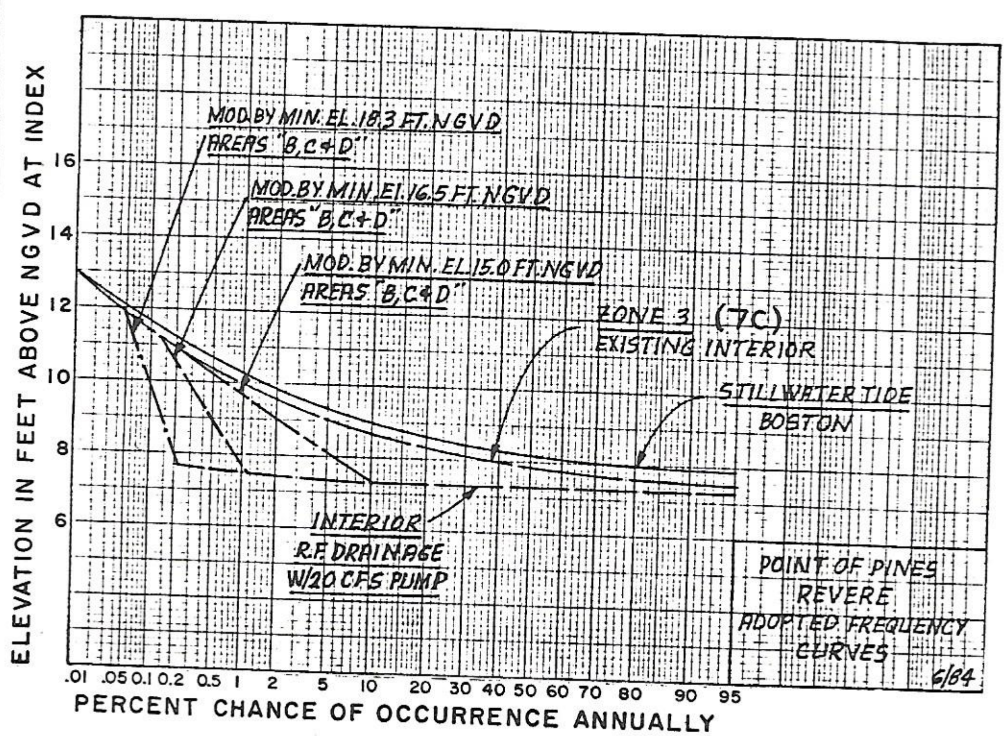
PLATE 7

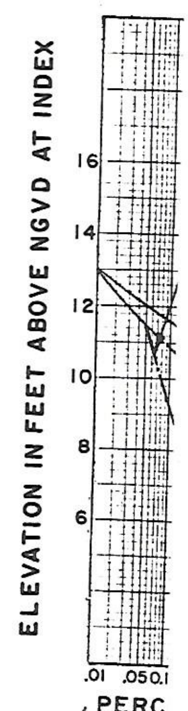




-4.5 -3 -5.5

see Notes #1 + #2





-5 -2.5 -6

Point of Pines
Revetments only