Marshfield Section 103 Reconnaissance Study
Coastal Engineering Analysis

New England District
Water Management Section

January 05, 2006

1.0 Purpose/Scope of Work

The intent of this work was to determine if there was a flooding potential related to wave overtopping of the existing seawall in Marshfield, MA and to determine if potential alternatives exist that could be constructed with a positive benefit cost ratio. The study level was conducted at a reconnaice level. The wave overtopping calculations, flood depths, flood frequency analysis was done using simplified assumptions and basic calculations. The only alternative investigated was to raise the sea wall. If this study/project is moved forward into a feasibility level study a much more thorough investigation into the flooding conditions will need to be completed along with full alternatives analysis.

2.0 Problem/Project Description

The study area is located in the town of Marshfield, MA, which is located between Boston and Cape Cod on Cape Cod Bay (Figure 1). The project is comprised of two sites within Marshfield and they are shown in Figure 2. The areas are the Fieldston/Rexhame area (referred to as Fieldston in this report) and the Brant Rock area.

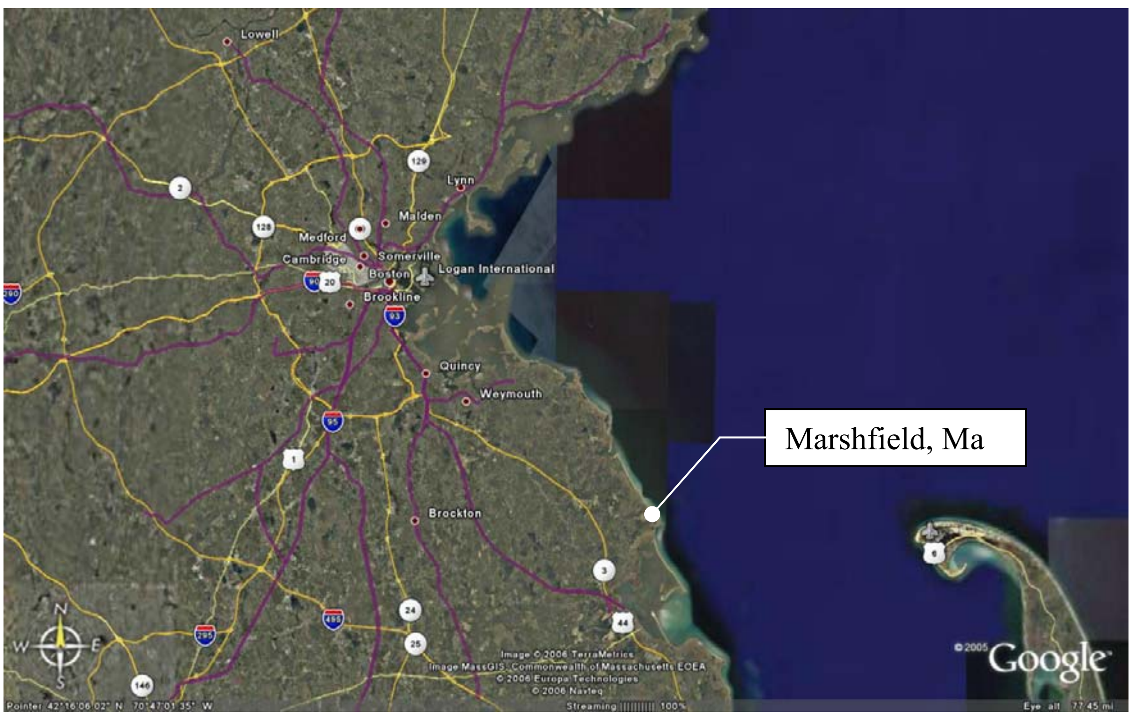

Figure 1. Marshfield location Map

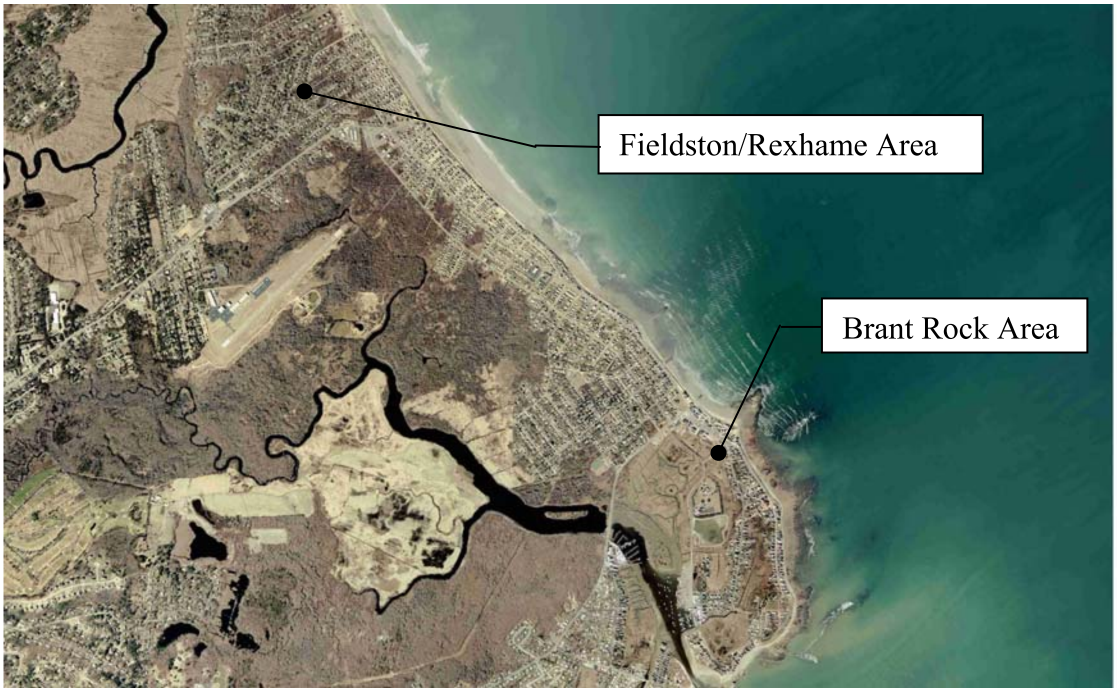

Figure 2. Fieldston and Brant Rock locations

Both areas are susceptible to fairly frequent coastal flooding due to each areas low elevation, their proximity to the ocean/Green Harbor, and local drainage issues. Typically the flooding events are extra tropical winter storms, know better as Nor'easters. In Figures 3 and 4 the contoured elevations are shown for each area and in Table 1 the ocean tidal regime has been provided to demonstrate the elevation issues of each site. The contour data was provided by the Town of Marshfield's GIS department and was provided as 1 foot contour data. The Boston tidal benchmark data was provided due to the proximity of Marshfield to Boston and the minor differences noted on the NOAA tidal benchmark/prediction web pages between the two areas. As seen in Figure 3, Fieldston has elevations that are lower than Mean High Water (MHW) and the Brant Rock area has elevations that are lower than the annual high tide/return period water level. Both areas are protected by a seawall with an approximate crest elevation of 12 feet NAVD88 and higher land between the low areas and the ocean.

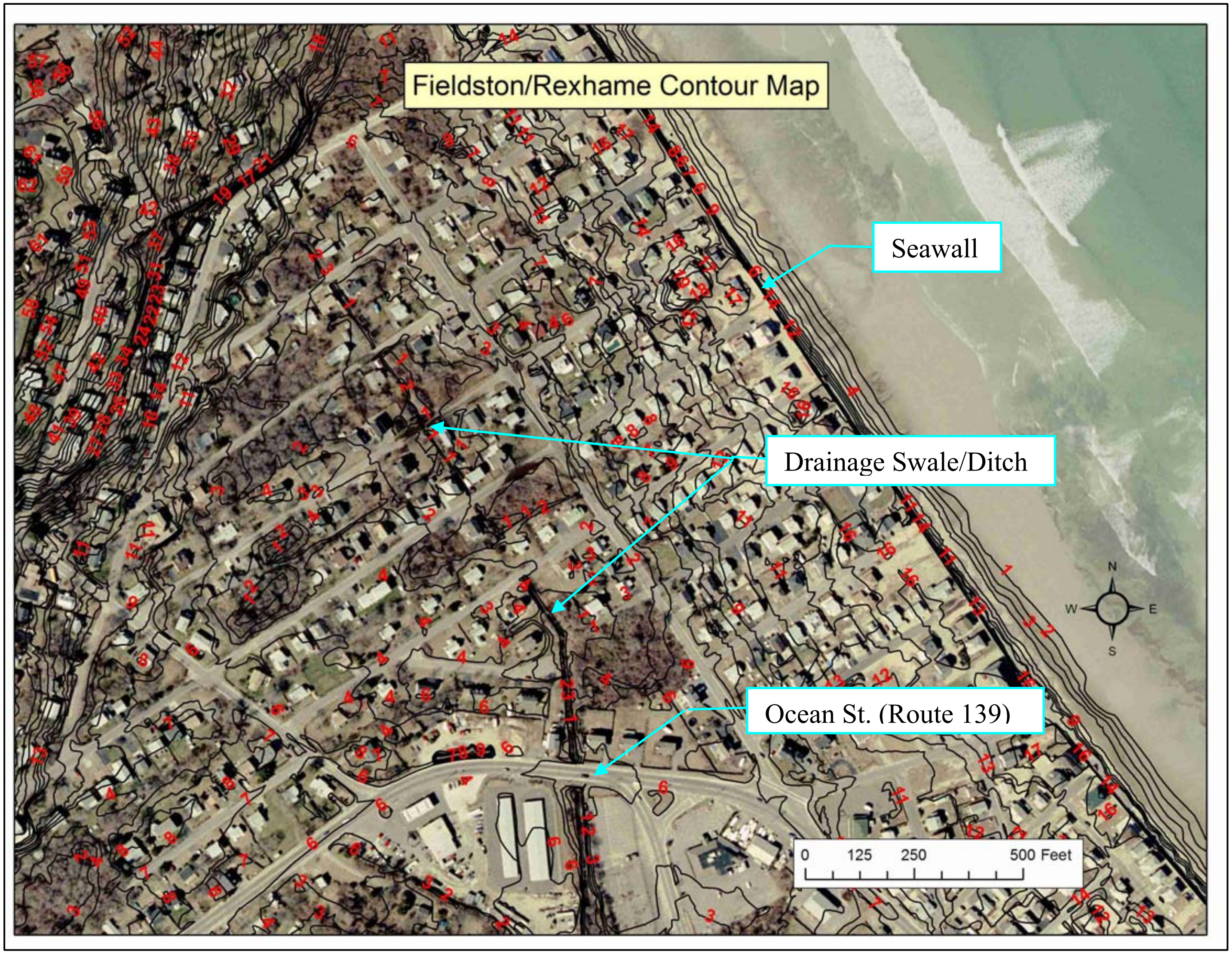

Figure 3. Fieldston area contour map

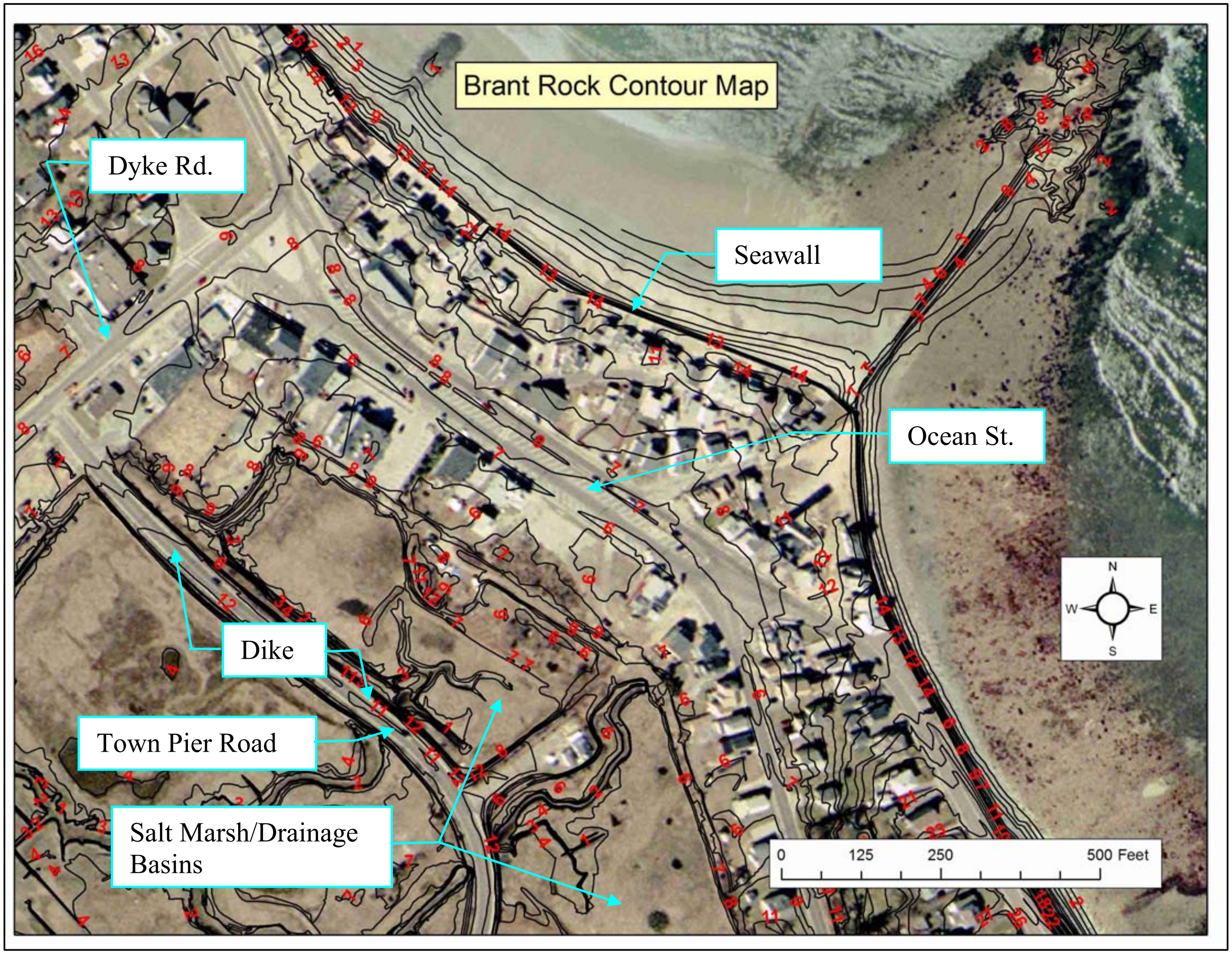

Figure 4. Brant Rock area contour map

Table 1. Boston, MA Bench Mark Data

	MLLW	MTL	NGVD29	NAVD88
Datum	**feet**	**feet**	feet	**feet**
100-Year Return Period Water Level (adjusted to 2006)[1]			10.47	9.66
50-Year Return Period Water Level (adjusted to 2006)[1]			10.17	9.36
25-Year Return Period Water Level (adjusted to 2006)[1]				8.90
15-Year Return Period Water Level (adjusted to 2006)[1]				8.60
10-Year Return Period Water Level (adjusted to 2006)[1]			9.17	8.36
5-Year Return Period Water Level (adjusted to 2006)[1]				8.05
2-Year Return Period Water Level (adjusted to 2006)[1]				7.56
1-Year Return Period Water Level (adjusted to 2006)[1]			7.57	6.76
Max. Annual Predicted Tide (2005 to 2023)[2]	12.50	7.41	7.80	6.99
Max. Annual Predicted Tide (average)[3]	12.23	7.14	7.53	6.72
MEAN HIGHER HIGH WATER (MHHW)	10.27	5.18	5.57	4.77
MEAN HIGH WATER (MHW)	9.83	4.74	5.13	4.32
NORTH AMERICAN VERTICAL DATUM-1988 (NAVD)	5.51	0.42	0.81	0.00
Mean Sea Level (MSL)	5.20	0.11	0.50	-0.31
MEAN TIDE LEVEL (MTL)	5.09	0.00	0.39	-0.42
NGVD29	4.70	-0.39	0.00	-0.81
Mean Low Water (MLW)	0.45	-4.64	-4.25	-5.06
Mean Lower Low Water (MLLW)	0.00	-5.09	-4.70	-5.51

LENGTH OF SERIES: 19 Years

TIME PERIOD: January 1983 - December 2001

TIDAL EPOCH: 1983-2001

[1]The elevations were adjusted using the sea level rise rate provided by NOAA for Boston Harbor. The elevations were corrected from 1988 (study completion date) to 2006 by applying the 0.87 feet/century rise rate over the 19 year time period or a correction of 0.17 feet.

[2]The elevation was determined using Tides and Currents Pro software to find the maximum annual predicted tide and then taking the maximum from that list (19 years of tidal predictions used)

[3]The elevation was determined using Tides and Currents Pro software to find the maximum annual predicted tide and then the average was taken (19 years of tidal predictions used)

3.0 Site Specific Flooding

Fieldston is believed to be prone to two types of flooding resulting from the low elevation of the area and poor local drainage. Fresh water (rain and snow) that falls in the immediate area is trapped in the "basin" formed by the bowl shaped topography and Ocean St., which acts a weir or small dam. As shown in Figures 3 and 11, there is an existing drainage swale/ditch that runs generally from north to south and exits the area under Ocean Blvd to the south. However, as seen in the field, and highlighted by the report titled "Rexhame Area Drainage Improvements" by Greene and Associates dated February 2005, the ditch is over grown, caved in, and poorly graded. The report by Greene and Associates found that the existing capacity is very small and that events smaller than a 2 year fresh water event would overwhelm the capacity of the ditch. The analysis developed several alternatives and concluded that upgrading the ditch and Ocean Street culverts could improve conditions to the point where the area would be "protected" from a 25 year fresh water event. However, the analysis did not take into account water from wave overtopping the seawall and flowing down into the basin. The volume of wave overtopping for several return period storms will be analyzed in this report.

The Brant Rock area is prone to flooding as well, and it has been reported that the largest flooding issue is from waves overtopping the seawall and the ocean water collecting in the low areas along Ocean Street. The topography around this area is complex and to completely understand the flooding issue in this area would take a significant study. As shown in Figures 4 and 12, the eastern border of this area is fronted by a seawall, to the south is an elevated headland type feature, to the west is a basin/salt marsh area that is separated from Green Harbor by a dike and tide gates, and to the north there is a hill with a significant elevation increase. As the seawall is overtopped the water initially collects in the salt marsh/basin area to the west. However, once these basins are full, the water overflows and floods the buildings and streets of the Brant Rock area.

4.0 Coastal Flooding Analysis

As discussed in the previous sections, the Fieldston and Brant Rock areas are prone to both freshwater and coastal flooding. The purpose of this analysis was to determine at a basic level, the level of coastal flooding at each site. Since the Corp's Section 14 Authority is for storm damage reduction and not for local drainage issues, only the flooding from wave overtopping in each area was looked at in this report. Given the complexities of the drainage for each area a simplistic approach to the flooding potential was taken since this was only a reconnaissance level effort. The supporting calculations/information along with a list of the key assumptions has been provided in the next section.

4.1 Site Condition Calculations and Analysis Assumptions

1. The first assumption was that all of the volume of water that entered each area during a storm stayed in that area until it overflowed the topography into Green Harbor. For Fieldston the maximum flood elevation was controlled by the elevation of Ocean Street at the southern end of the area. The elevation of the road was 4.5 feet NAVD88 and as such once the water elevation reached that elevation it was assumed the water would overflow the road and enter Bass Creek/Green Harbor. For the Brant Rock area the controlling elevation was 8 feet NAVD88 and was located at the northwest corner of the flood area (Dyke Road and Town Pier Road). Once the flood water reached this elevation it was assumed the water would overflow this point and enter Green Harbor. Once again it must be realized that this area has a very complex flood profile due to the low elevations of the land, tidal gates on Green Harbor, tidal gates on the salt marshes at Brant Rock, and the dynamics of Green Harbor.

2. Further clarifying assumption #1 was that the water entering each area did not leave the area via storm water culverts or channels. As discussed and shown in Figure 3, the Fieldston area has a drainage ditch. However, due to the poor condition of this channel (as discussed in Section 2.0), and the fact it is overwhelmed from just the fresh water run off, the assumption of no "ocean" water draining was considered reasonable. For the Brant Rock area the salt marsh/basin area is cut off from Green Harbor by tidal control gates that are closed during storms to prevent waters from flowing into Brant

Rock from Green Harbor. As such no drainage is available except for the overflow at the corner of Dyke Road and Town Pier Road road. Once again for this level of study and the water flow situation the assumption of no water flowing out of this area was assumed to be reasonable.

3. Infiltration of the overtopped water was also assumed to be negligible and was not considered for this level of study. In the Fieldston area this is certainly believed to be a valid assumption due to the high water table and the local flooding from the freshwater runoff. At Brant Rock the elevation is slightly higher so there may be more potential for infiltration, but due to the high water table it is likely low and too insignificant for this study.

4. The storm events of various return frequencies were "created" due to the lack of processed data at the site and because of the level of study. The 1, 2, 5, and 10 year storm events were generated using the elevations in the New England Tidal Profiles Study from 1988. The plot of water level vs. return frequency taken from this sheet is shown as Figure 5. The storms were generated by taking a spring tide tidal profile from a predicted tide for the area and adding a "surge" to the tidal profile so that it would equal the return frequency tidal elevation. Each storm was assumed to last over two high tide events. While it is realized that this is less than ideal, for this level of study and the funding available it would be difficult to perform a comprehensive return period analysis with the associated tidal profiles. The created storm tidal profiles are shown in Figure 6.

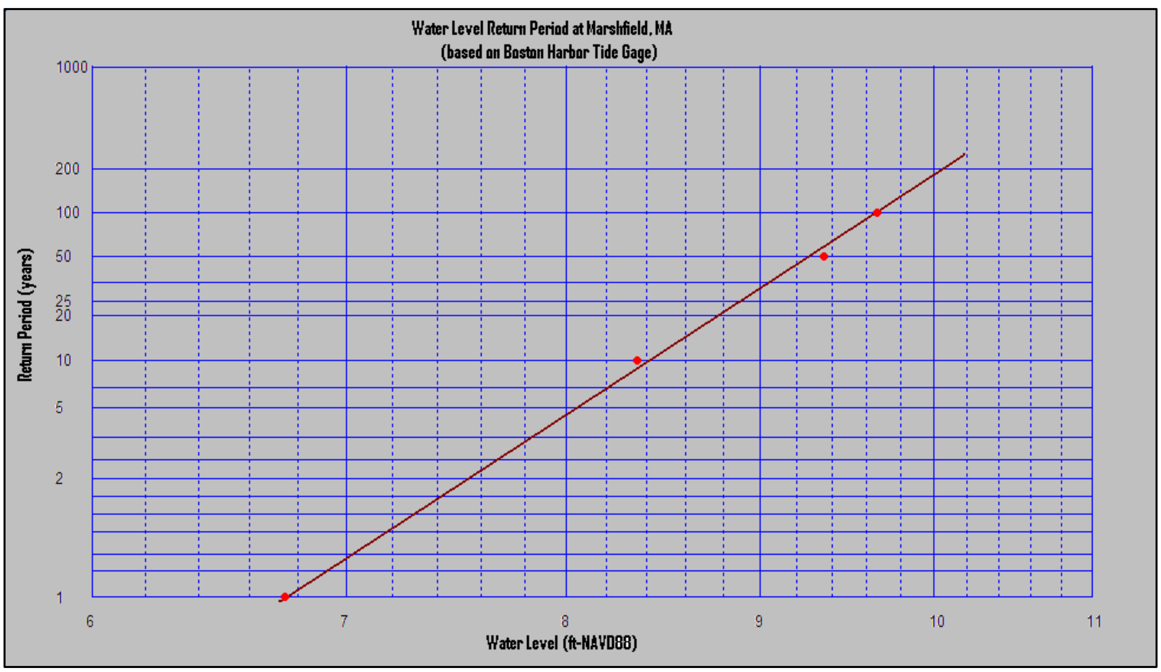

Figure 5. Storm tide return periods

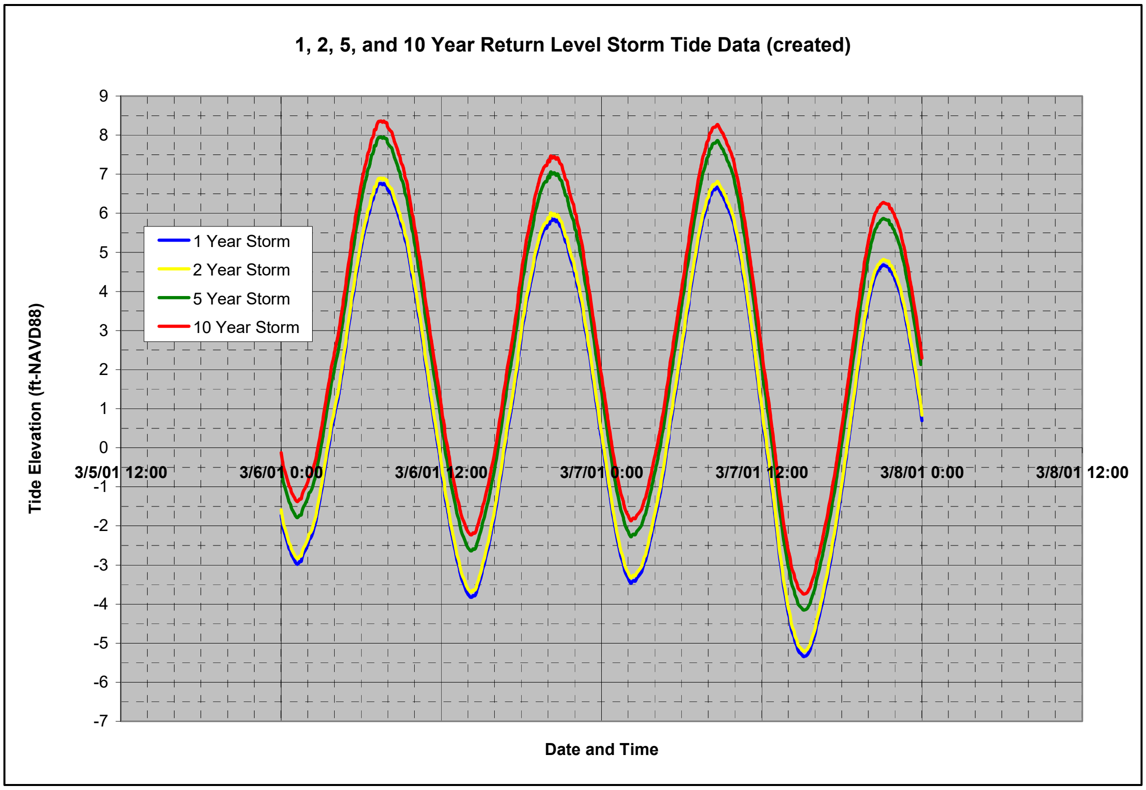

Figure 6. Created return period tide hydrographs

5. The storm waves were calculated using the water depth in front of the seawall since depth limited conditions apply at this site. This means the water depth in front of the seawall directly controls the size of the waves in front of the seawall. Significant wave height, or H_s, was calculated by multiplying the factor depth limiting factor of 0.60 times the water depth. This factor was taken from the CEM and can be found in Sections (II-4-2-a-4) and (II-2-2-b-3). Water depth was determined by using the NOAA LIDAR data from 2000 (Figures 7) and the storm water elevation. Based on the NOAA LIDAR data it was assumed that the beach elevation in front of the seawall at both locations was 4 feet NAVD88.

6. In order to calculate the overtopping volumes the formulation provided in the Wave Overtopping of Seawalls Design and Assessment Manual HR Wallingford Ltd February 1999 R&D Technical Report W178 was used. Based on the cross section of the Seawall (Figure 8) this formula was thought to be the most applicable. The equation and the necessary inputs have been provided below as Table 2. To calculate the overtopping volume throughout the storms a Microsoft Excel spread sheet was developed. A portion of the spread sheet for the 5 year event at Fieldston has been included as Table 3. As shown in the spreadsheet the tidal elevation is provided every six minutes. The water depth changes with each water surface elevation change, and therefore the depth limited wave height is changed as well. This information was fed into the overtopping formula which results in the overtopping volume at that point in the storm. The overtopping rate was initially reported as cubic feet per linear foot of wall length per second. This was converted to cubic feet per second per 1,000’ of seawall (Figure 9 and 10) and then

totaled to reach a total overtopping volume from the storm. The same procedure was followed for both Fieldston and Brant Rock, with the only difference being the length of seawall that is overtopped (1,000 feet and 1,700 feet, respectively). The total overtopping volume for each storm can be seen in Tables 4 and 6 in the next section.

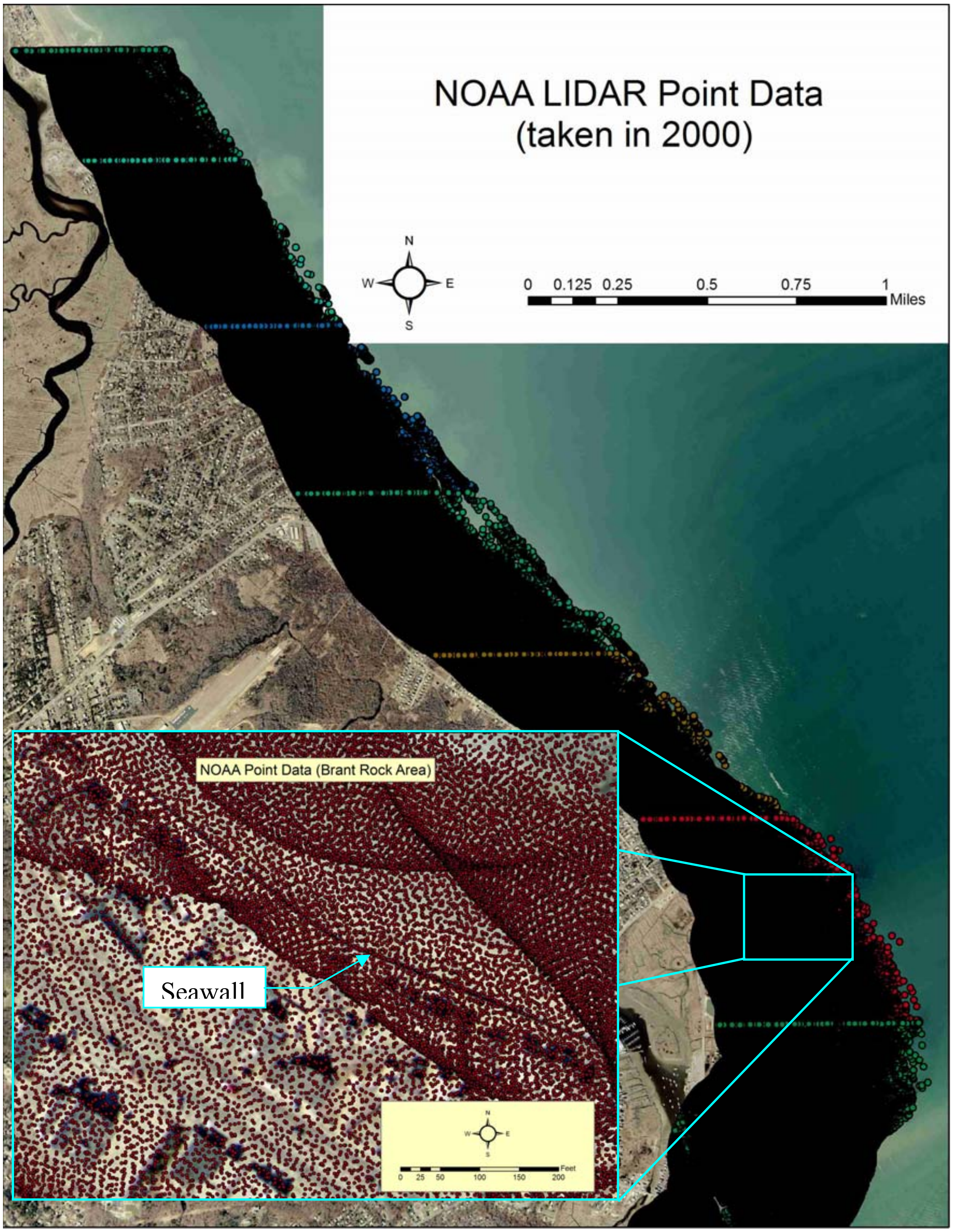

Figure7. NOAA LIDAR point data from 2000

Figure 8. Representative seawall cross sectional picture

Table 2. Seawall overtopping formulation (HR Wallingford Technical Report W178)

BOX 3.7
PLAIN VERTICAL WALLS
MEAN OVERTOPPING DISCHARGE

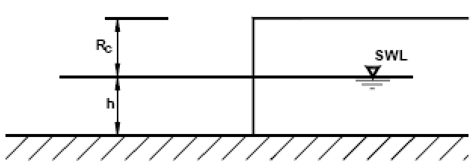

Figure 3.11 Plain vertical wall

NORMAL WAVE ATTACK

First calculate h_* to determine whether the waves are predominantly impacting or reflecting.
The parameter h_* is given by :-

$$h_* = (h/H_s)(2\pi h/(gT_m^2)) \qquad (22)$$

where
- h is the water depth at the toe of the structure (m)
- H_s is the significant wave height at the toe of the structure (m)
- g is acceleration due to gravity (m/s^2)
- T_m is the mean wave period at the toe of the structure (s)

Impact waves predominate when $h_* \leq 0.3$, in which case the following equation applies:-

$$Q_h = 0.000137\, R_h^{-3.24} \qquad (24)$$

where Q_h is the dimensionless discharge, given by :-

$$Q_h = \{Q/(gh^3)^{0.5}\} / h_*^2 \qquad (25)$$

and R_h is the dimensionless crest freeboard, given by :-

$$R_h = (R_c/H_s)h_* \qquad (26)$$

Equation 26 is valid for $0.05 < R_h < 1.00$

Table 3. Example of Excel spread sheet used to calculate overtopping rates and volumes

5 Year Storm Event - 2' seawall raise															
time	Water Elev.	Bottom Elev.	Elev. of Seawall	T_p	Depth (h)	Rc	H_s	T_m	h*	Rh	Qh	Q	**Q**	**Q (Rexhame)**	**Total Volume**
date and time	feet	feet	feet	sec.	meters	meters	meters	second				$M^3/s/m$	**$ft^3/s/ft$**	**$ft^3/s/1000'$**	**ft^3**
3/6/01 6:18	6.81	4	14	10	0.86	2.192	0.51	8.2	0.01359	0.05796	1.394	0.0006	0.01	6.88	2478.353469
3/6/01 6:24	6.93	4	14	10	0.89	2.155	0.54	8.2	0.01417	0.05699	1.47213	0.0008	0.01	8.42	3029.775869
3/6/01 6:30	7.11	4	14	10	0.95	2.100	0.57	8.2	0.01504	0.05554	1.60042	0.0010	0.01	11.27	4058.109995
3/6/01 6:36	7.19	4	14	10	0.97	2.076	0.58	8.2	0.01543	0.05489	1.66214	0.0012	0.01	12.80	4606.419435
3/6/01 6:42	7.35	4	14	10	1.02	2.027	0.61	8.2	0.0162	0.0536	1.79524	0.0015	0.02	16.40	5904.830364
3/6/01 6:48	7.49	4	14	10	1.06	1.984	0.64	8.2	0.01688	0.05248	1.92337	0.0019	0.02	20.28	7300.949141
3/6/01 6:54	7.55	4	14	10	1.08	1.966	0.65	8.2	0.01717	0.05199	1.98194	0.0021	0.02	22.18	7985.801153
3/6/01 7:00	7.73	4	14	10	1.14	1.911	0.68	8.2	0.01804	0.05054	2.17229	0.0027	0.03	28.91	10407.0301
3/6/01 7:06	7.73	4	14	10	1.14	1.911	0.68	8.2	0.01804	0.05054	2.17229	0.0027	0.03	28.91	10407.0301
3/6/01 7:12	7.87	4	14	10	1.18	1.868	0.71	8.2	0.01872	0.04941	2.33718	0.0033	0.04	35.38	12738.23251
3/6/01 7:18	7.94	4	14	10	1.20	1.847	0.72	8.2	0.01906	0.04885	2.42579	0.0036	0.04	39.10	14077.2624
3/6/01 7:24	7.94	4	14	10	1.20	1.847	0.72	8.2	0.01906	0.04885	2.42579	0.0036	0.04	39.10	14077.2624
3/6/01 7:30	7.95	4	14	10	1.20	1.844	0.72	8.2	0.0191	0.04877	2.4388	0.0037	0.04	39.66	14278.91317
3/6/01 7:36	7.92	4	14	10	1.19	1.853	0.72	8.2	0.01896	0.04901	2.40003	0.0035	0.04	38.01	13681.89866
3/6/01 7:42	7.95	4	14	10	1.20	1.844	0.72	8.2	0.0191	0.04877	2.4388	0.0037	0.04	39.66	14278.91317
3/6/01 7:48	7.91	4	14	10	1.19	1.856	0.72	8.2	0.01891	0.04909	2.38728	0.0035	0.04	37.47	13488.11793
3/6/01 7:54	7.91	4	14	10	1.19	1.856	0.72	8.2	0.01891	0.04909	2.38728	0.0035	0.04	37.47	13488.11793
3/6/01 8:00	7.79	4	14	10	1.16	1.893	0.69	8.2	0.01833	0.05006	2.24103	0.0029	0.03	31.54	11353.06651
3/6/01 8:06	7.74	4	14	10	1.14	1.908	0.68	8.2	0.01809	0.05046	2.18355	0.0027	0.03	29.33	10559.47922
3/6/01 8:12	7.7	4	14	10	1.13	1.920	0.68	8.2	0.01789	0.05078	2.13895	0.0026	0.03	27.67	9961.744203
3/6/01 8:18	7.66	4	14	10	1.12	1.932	0.67	8.2	0.0177	0.05111	2.09554	0.0024	0.03	26.10	9395.228243
3/6/01 8:24	7.54	4	14	10	1.08	1.969	0.65	8.2	0.01712	0.05207	1.97202	0.0020	0.02	21.85	7867.754328
3/6/01 8:30	7.41	4	14	10	1.04	2.009	0.62	8.2	0.01649	0.05312	1.84874	0.0017	0.02	17.97	6470.596447
3/6/01 8:36	7.34	4	14	10	1.02	2.030	0.61	8.2	0.01615	0.05368	1.78652	0.0015	0.02	16.15	5814.988539
3/6/01 8:42	7.24	4	14	10	0.99	2.060	0.59	8.2	0.01567	0.05449	1.7023	0.0013	0.01	13.84	4981.647258
3/6/01 8:48	7.11	4	14	10	0.95	2.100	0.57	8.2	0.01504	0.05554	1.60042	0.0010	0.01	11.27	4058.109995
3/6/01 8:54	7.04	4	14	10	0.93	2.121	0.56	8.2	0.0147	0.0561	1.54885	0.0009	0.01	10.07	3626.572283
3/6/01 9:00	6.88	4	14	10	0.88	2.170	0.53	8.2	0.01393	0.05739	1.4389	0.0007	0.01	7.75	2788.247102
3/6/01 9:06	6.81	4	14	10	0.86	2.192	0.51	8.2	0.01359	0.05796	1.394	0.0006	0.01	6.88	2478.353469

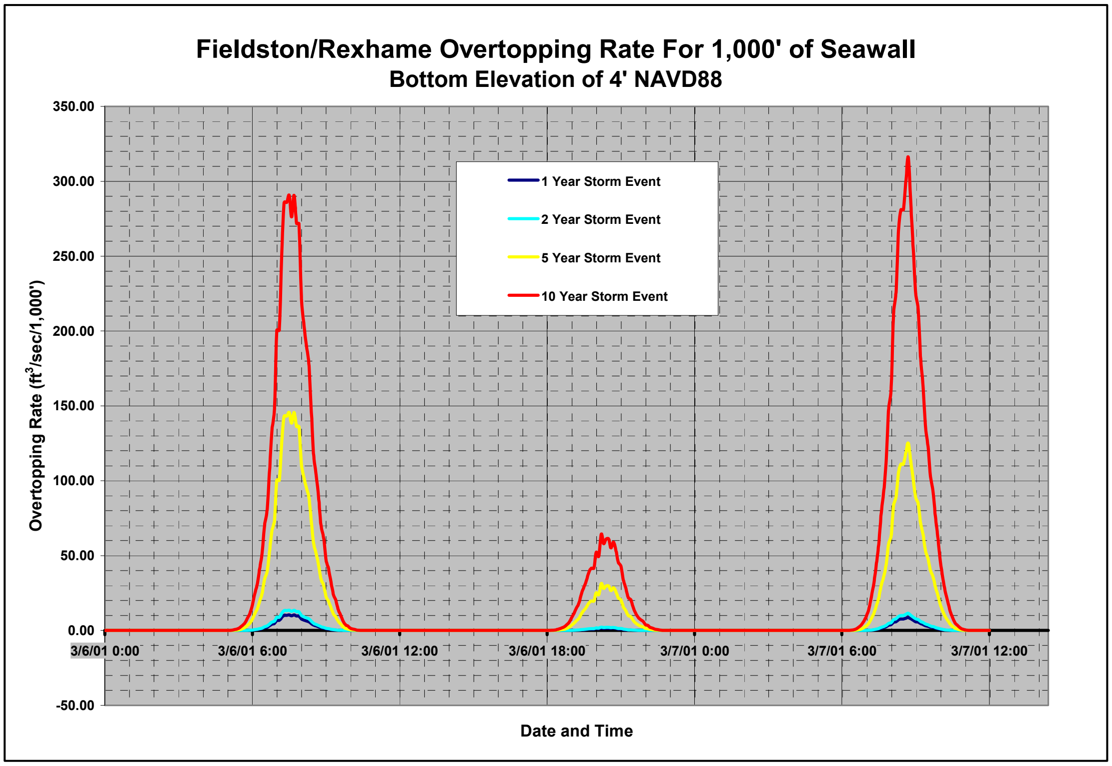

Figure 9. Overtopping rate for the Fieldston area seawall.

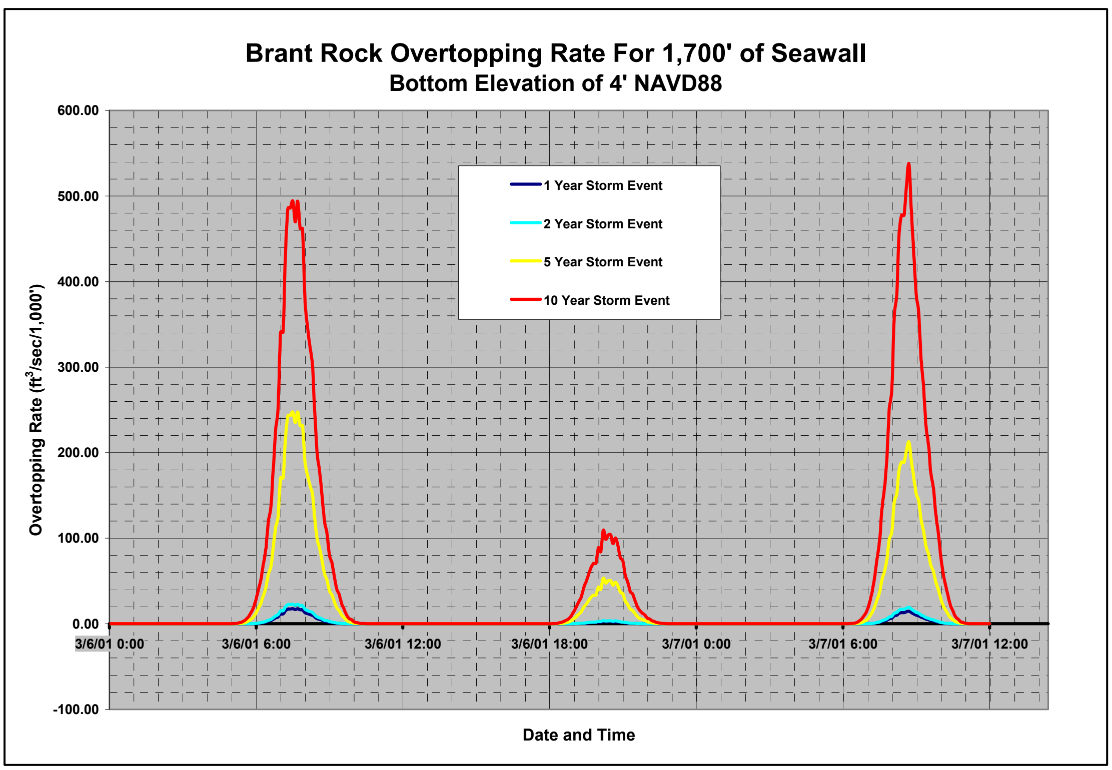

Figure 10. Overtopping rate for the Brant Rock area seawall.

4.2 Flooding Depth Determination

4.2.1 Existing Conditions

Once the total volume of overtopped water was determined for each storm the flooding depth for each area was mapped in ArcMap GIS. The maximum elevation of the water was determined by calculating the volume below each elevation for each area using digital terrain models developed from the Marshfield contour data and using the volume calculator in the ArcMap 3-D Analyst extension. The digital terrain models used for each area can be seen in Figures 11 and 12. With that information, the flood elevation that would be reached during each storm could be determined. Tables 4 through 7 show the "storage" volume vs. elevation for each area and show the total storm overtopping volumes. To clarify how this information was used an example has been provided.

> Looking at Table 5 it can be seen that the total overtopping volume for Fieldston during a 2 year storm has been calculated to be 141,210 cubic feet. Looking at Table 4 it can be seen that this volume will "fill up" the Fieldston area to an elevation between 1.5 feet and 2 feet since the storage volume is 163,141 cubic feet at elevation 2 and only 67,985 cubic feet at elevation 1.5. Since the 141,210 cubic feet is much closer to the volume associated with 2 feet it was assumed that during a 2 year storm the flood waters resulting from wave overtopping would reach elevation 2 feet NAVD88 at Fieldston. It can also be seen that for anything greater than a five year storm the overtopping volume overwhelms the area and passes over Ocean St. which is at elevation 4.5 feet NAVD88. Flooding depths for both areas are shown in Figures 13 through 17.

Tables 4 and 5. Fieldston storage volume vs. elevation and storm overtopping volume

Fieldston/Rexhame Area Storage Capcity and Storm overtopping Volumes

Volume vs. Elevation

Elevation ft-NAVD88	Volume ft^3
1	5,360
1.5	67,985
2	163,141
2.5	323,514
3	525,626
3.5	827,635
4	1,181,530
4.5	1,630,414

Overtopping Volume vs. Storm Event

Storm Return Period Year	Volume of Overtopping ft^3
1	105,998
2	141,210
5	1,727,800
10	3,870,644

Tables 6 and 7. Brant Rock storage volume vs. elevation and storm overtopping volume

Brant Rock Area Storage Capcity and Storm overtopping Volumes

Volume vs. Elevation

Elevation ft-NAVD88	Volume ft^3
4	27,850
4.5	49,178
5	80,069
5.5	126,720
6	180,489
6.5	269,970
7	382,133
7.5	535,913
8	710,897

Overtopping Volume vs. Storm Event

Storm Return Period Year	Volume of Overtopping ft^3
1	180,196
2	240,057
5	2,937,260

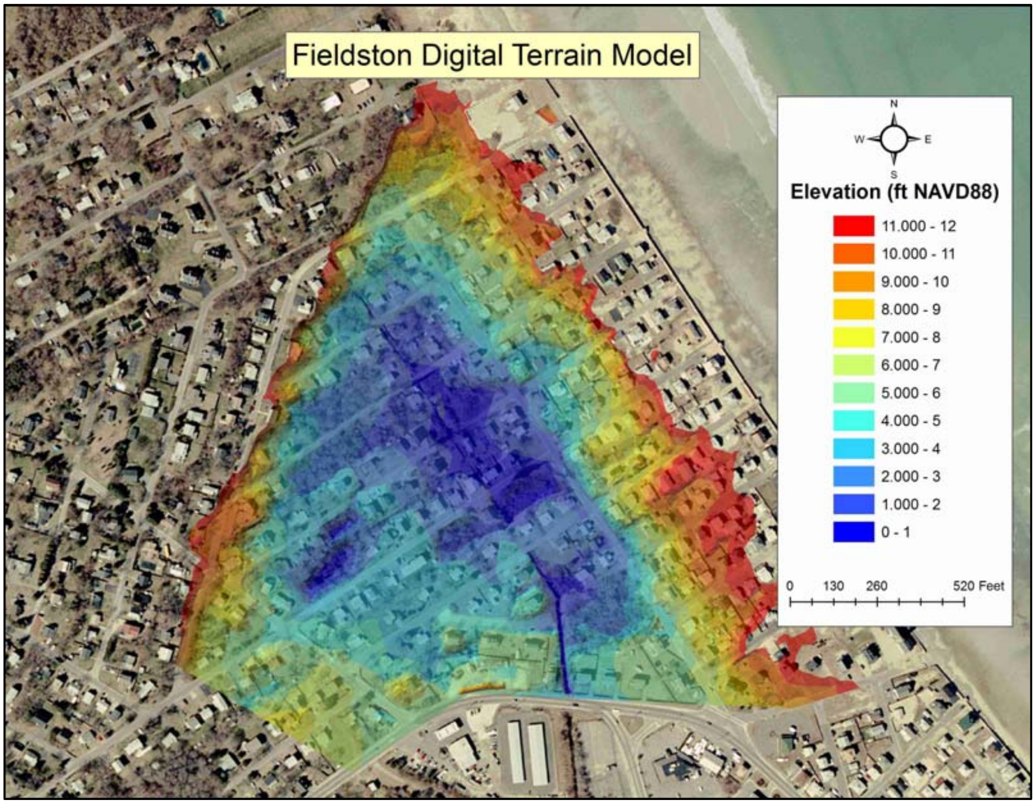

Figure 11. Fieldston digital terrain model.

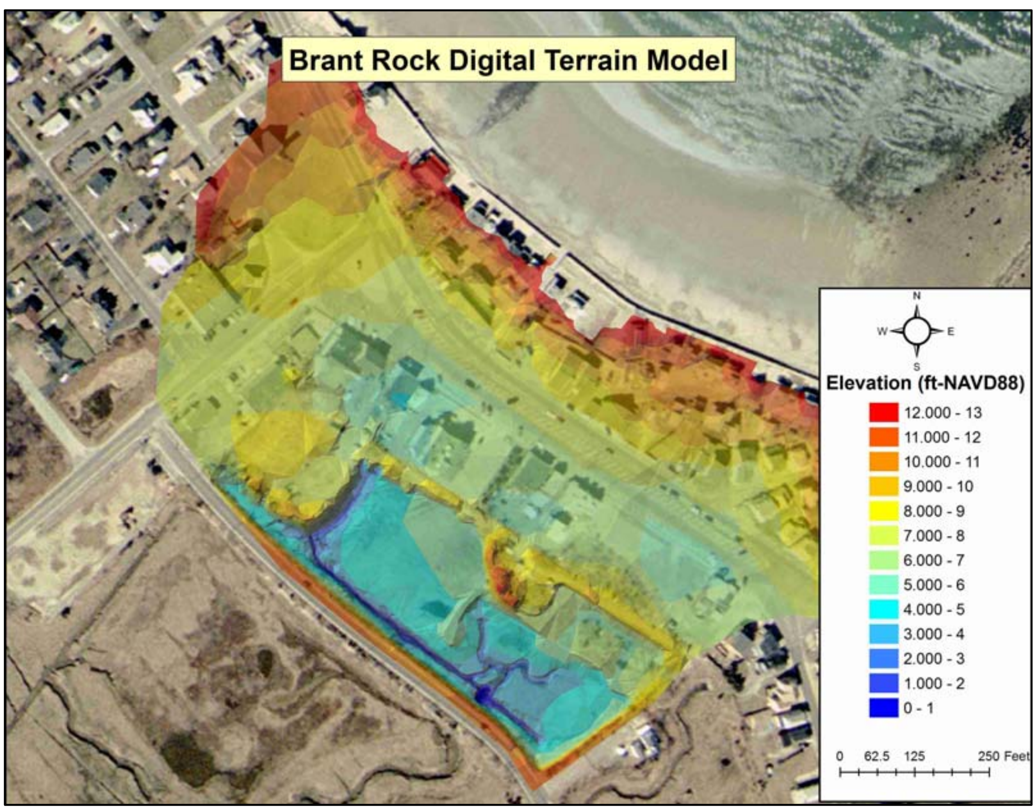

Figure 12. Brant Rock digital terrain model.

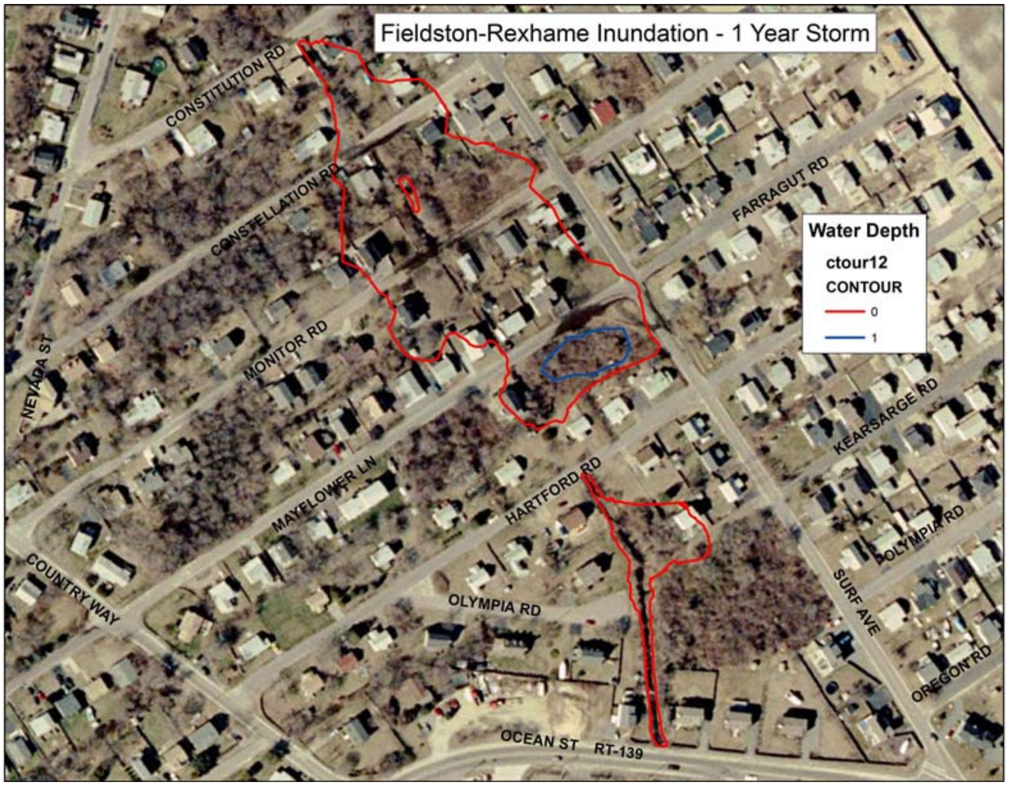

Figure 13. Fieldston Inundation for 1 year storm - existing conditions

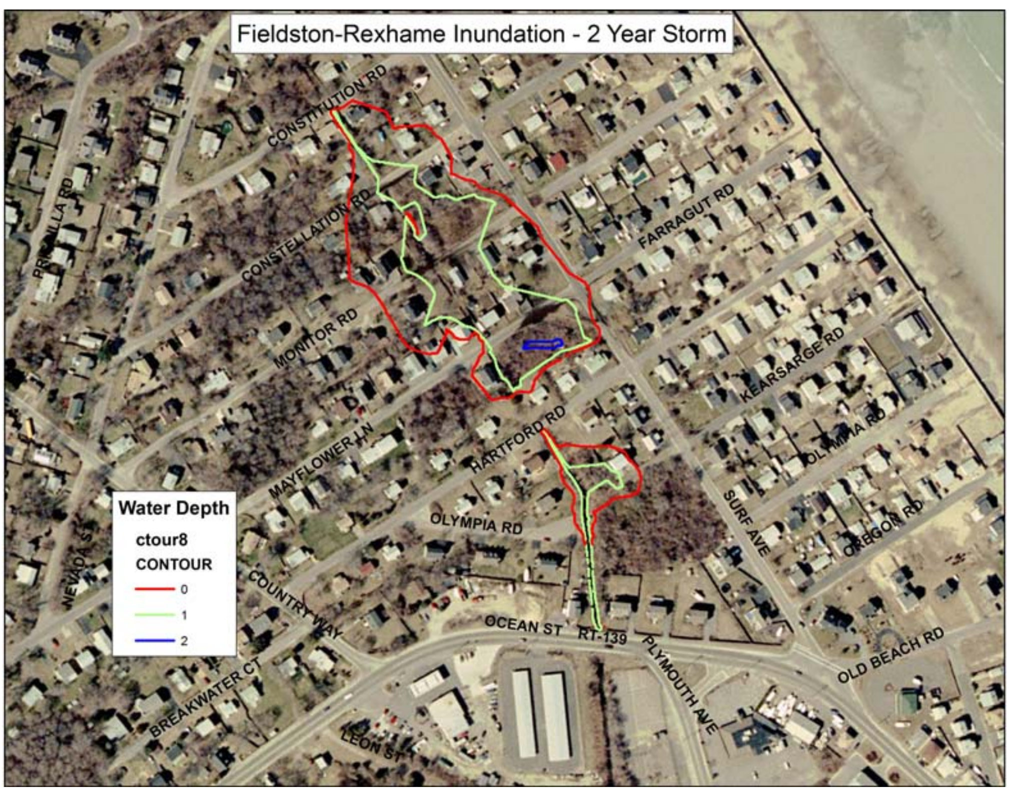

Figure 14. Fieldston Inundation for 2 year storm – existing conditions

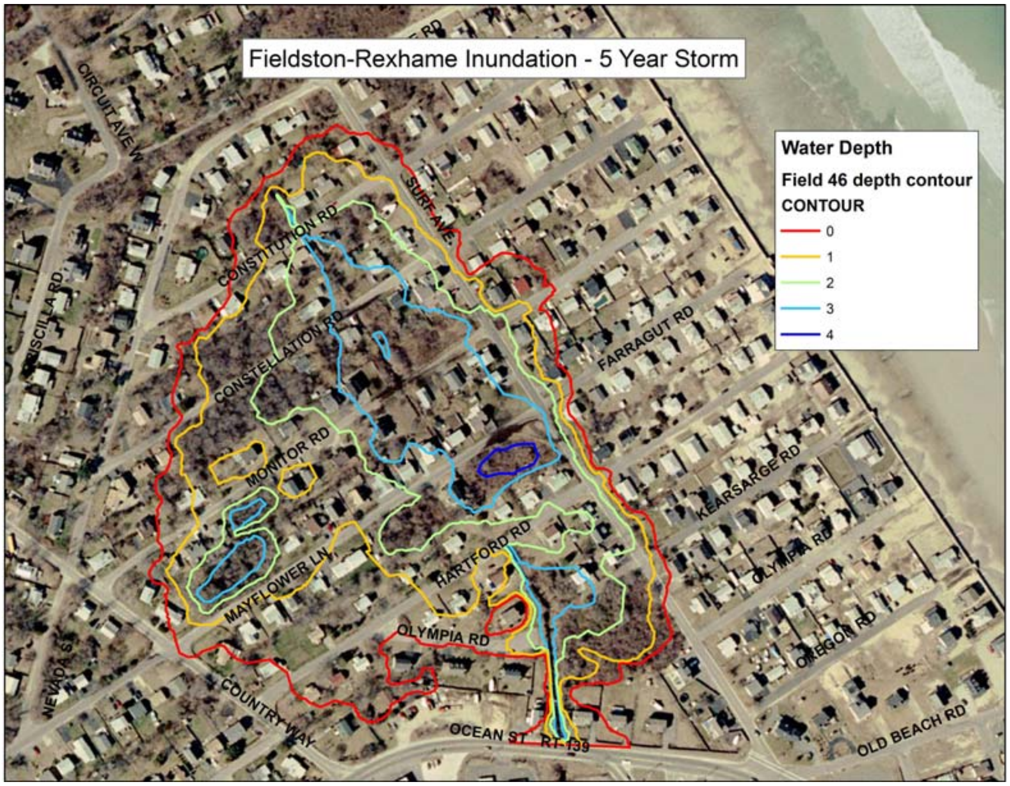

Figure 15. Fieldston Inundation for 5 year storm - existing conditions

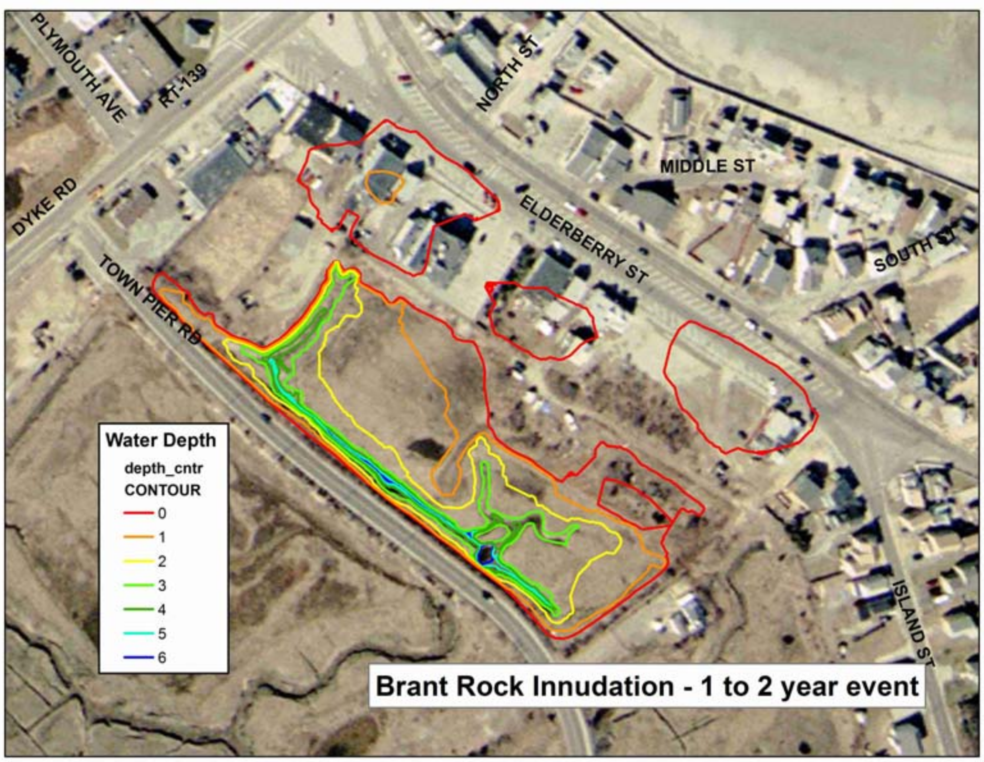

Figure 16. Brant Rock for 1 to 2 year storm - existing conditions

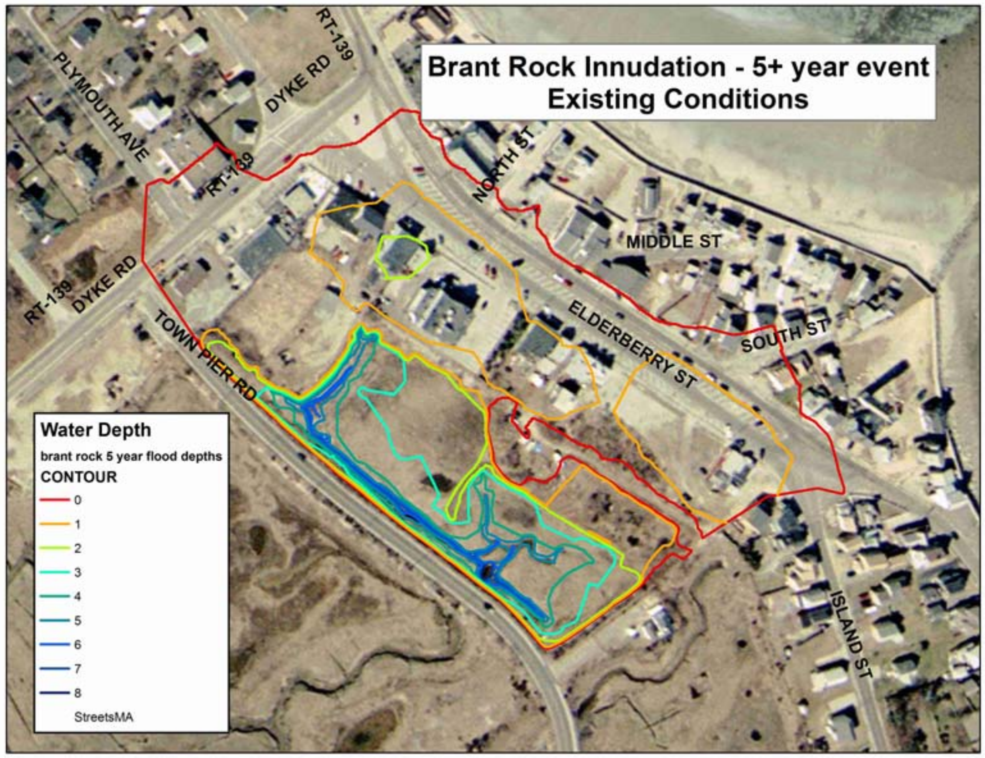

Figure 17. Brant Rock for 5+ year storm - existing conditions

4.2.2 Previous Studies and Observational Information

Without actual overtopping and flood depth data to verify the calculations past studies and personal observations from the Town staff and residents was checked to see if the calculated values were reasonable. Comparison to previous studies was difficult since they did not look at the specific issue of wave overtopping and subsequent flooding. The previous studies by the Corps and consultants for the Town looked at flooding due to Green Harbor, tide gate control, fresh water run off/drainage, and a combination of all three. This information is of interest and important, but the purpose of this study was to investigate the impact of wave/ocean flooding over the seawall.

Looking at the FEMA map for the area, it can be seen that the two areas of interest do have AO type flooding (Figure 18). As shown in flood zone legend in Figure 18 this is sheet flow type flooding, and so for this area it means wave overtopping of the seawall and the subsequent flow down to the lower elevations of each area.

At both sites significant overtopping has been witnessed fairly frequently (1 to 5 year events) resulting in fairly frequent flooding events. At the Brant Rock area it has been noted that the flooding is caused almost entirely by wave overtopping of the seawall and examples of the overtopping are shown in Figures 19 through 22. This is makes sense when looking at the surrounding topography and noting the relatively small area that drains into Brant Rock. For the Fieldston area the flooding is caused by both fresh water drainage and wave overtopping of the seawall. The wave overtopping has been witnessed by residents. It is also worth noting that the seawall elevation and exposure to the ocean is the same as Brant Rock so it would be expected that wave overtopping at Fieldston is significant.

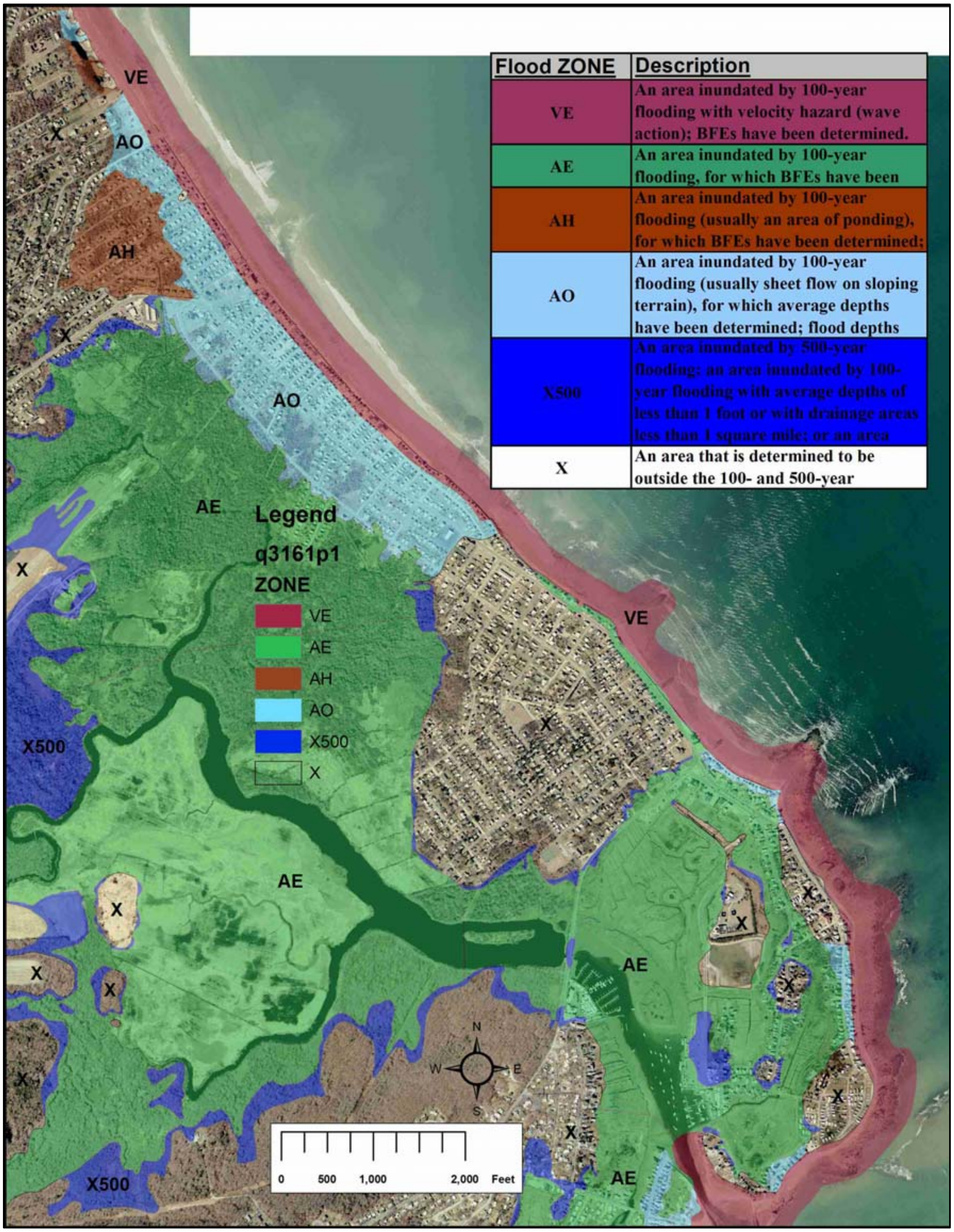

Figure 18. FEMA flood zone mapping

Figure 19. Brant Rock wave overtopping

Figure 20. Brant Rock wave overtopping

Figure 21. Brant Rock wave overtopping

Figure 22. Brant Rock wave overtopping

4.2.3 With-Project Condition

To determine if a cost effective alternative was available to help reduce the wave overtopping flooding problems, raising the seawall by two feet at each area was investigated. Since this was a reconnaissance level study, a complete set of alternatives was not needed, but instead only one that showed a positive benefit cost ratio. To analyze the raised seawall alternative the same exact procedure using the same storm water levels, wave heights, and overtopping analysis was used. The same method for determining the flood levels for each area were also used. The overtopping rates for the various return period storms have been provided as Figures 23 and 24 (same as Figures 10 and 11 for the existing conditions). The storage volume vs. elevation and total storm overtopping volumes for each area have been provided in Tables 8 through 11. The revised flooding depths for the with-project conditions for the Fieldston area are provided in Figures 25 through 27. It can be seen when comparing these figures to Figures 14 through 16 that the flooding is reduced and it takes an event larger than a 10 year event to overflow Ocean St. instead of a 5 year event for existing conditions. For the Brant Rock area based, on the overtopping calculations and the storage volumes provided in Tables 10 and 11, it can be seen that flooding is eliminated for the 1 and 2 year storms (as shown in Figure 16 there is flooding from the 1 to 2 year event for existing conditions). However, for with project conditions, the 5 year storm still fully inundates the Brant Rock area. New inundation figures were not developed for the Brant Rock with-project condition.

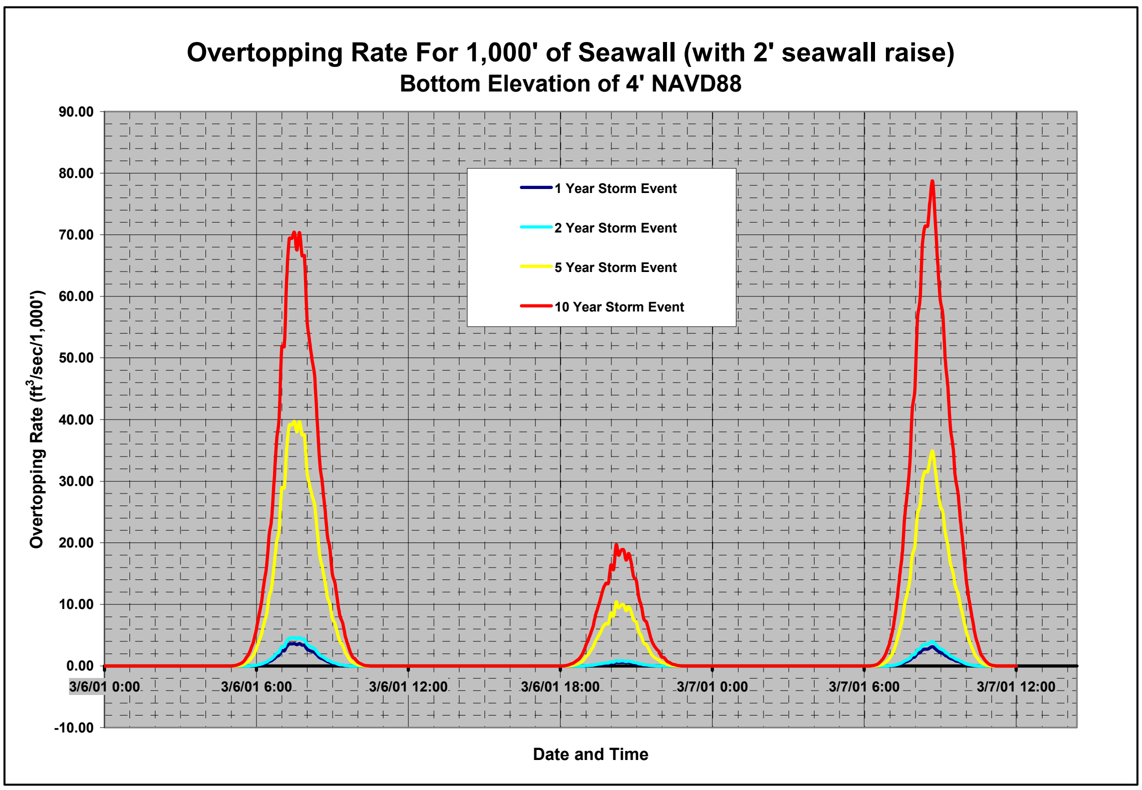

Figure 23. Overtopping rate for the Fieldston area seawall (with 2' seawall raise).

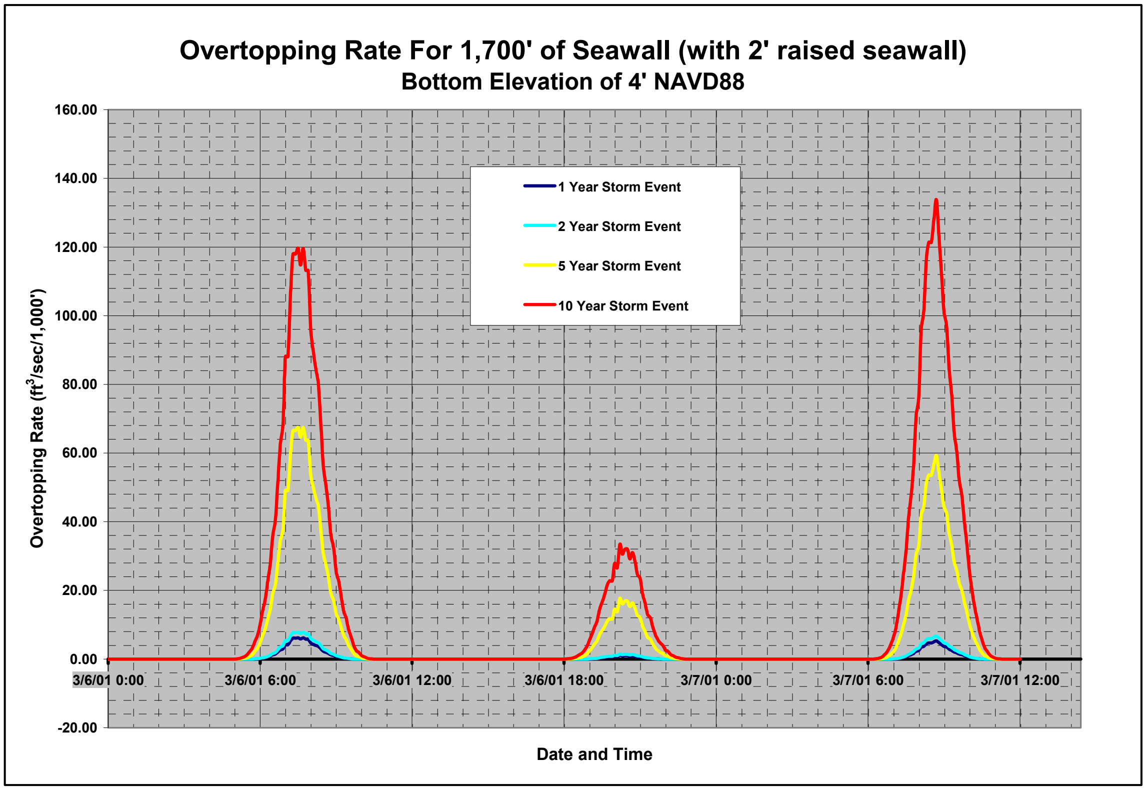

Figure 24. Overtopping rate for the Brant Rock area seawall (with 2' seawall raise).

Tables 8 and 9. Fieldston storage volume vs. elevation and storm overtopping volume (with 2' seawall raise)

<u>Fieldston/Rexhame Area Storage Capcity and Storm Overtopping Volumes (with 2' seawall raise)</u>

Volume vs. Elevation

Elevation ft-NAVD88	Volume ft^3
1	5,360
1.5	67,985
2	163,141
2.5	323,514
3	525,626
3.5	827,635
4	1,181,530
4.5	1,630,414

Overtopping Volume vs. Storm Event

Storm Return Period Year	Volume of Overtopping ft^3
1	39,276
2	51,296
5	521,819
10	1,062,559
15	2,143,161

Tables 10 and 11. Brant Rock storage volume vs. elevation and storm overtopping volume (with 2' seawall raise)

Brant Rock Area Storage Capcity and Storm Overtopping Volumes (with 2' seawall raise)

Volume vs. Elevation

Elevation ft-NAVD88	Volume ft^3
4	27,850
4.5	49,178
5	80,069
5.5	126,720
6	180,489
6.5	269,970
7	382,133
7.5	535,913
8	710,897

Overtopping Volume vs. Storm Event

Storm Return Period Year	Volume of Overtopping ft^3
1	66,770
2	87,203
5	887,093

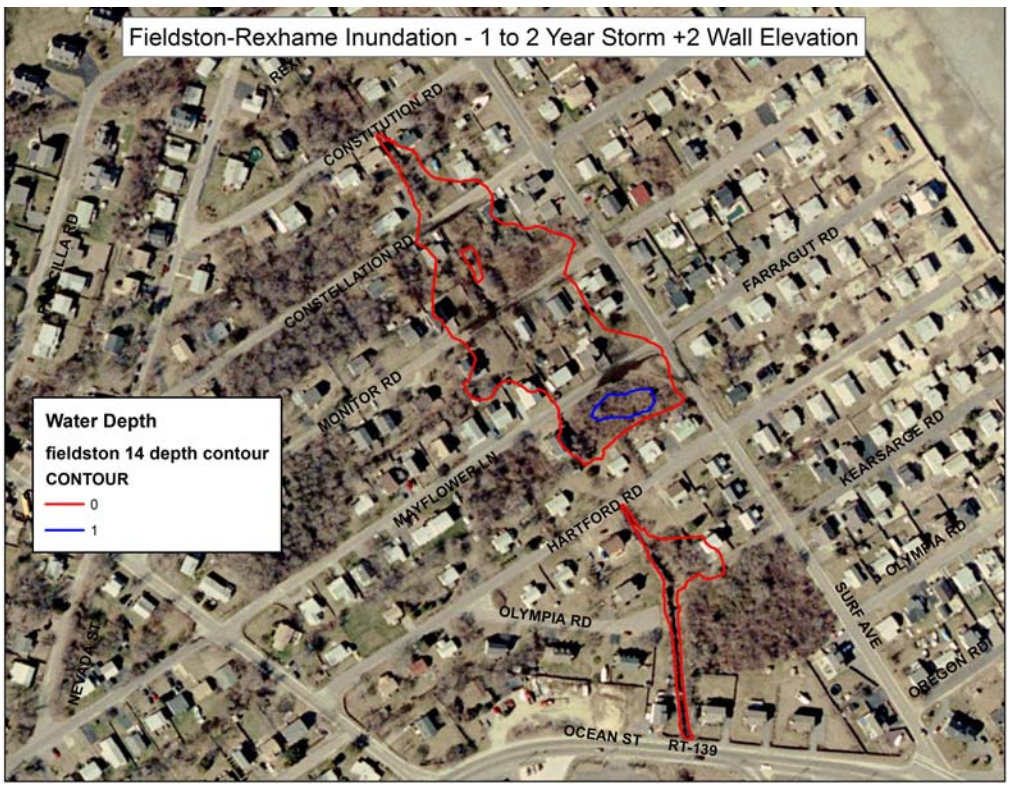

Figure 25. Fieldston Inundation for 1 and 2 Year Storm +2' on seawall

Figure 26. Fieldston Inundation for 5 Year Storm +2' on seawall

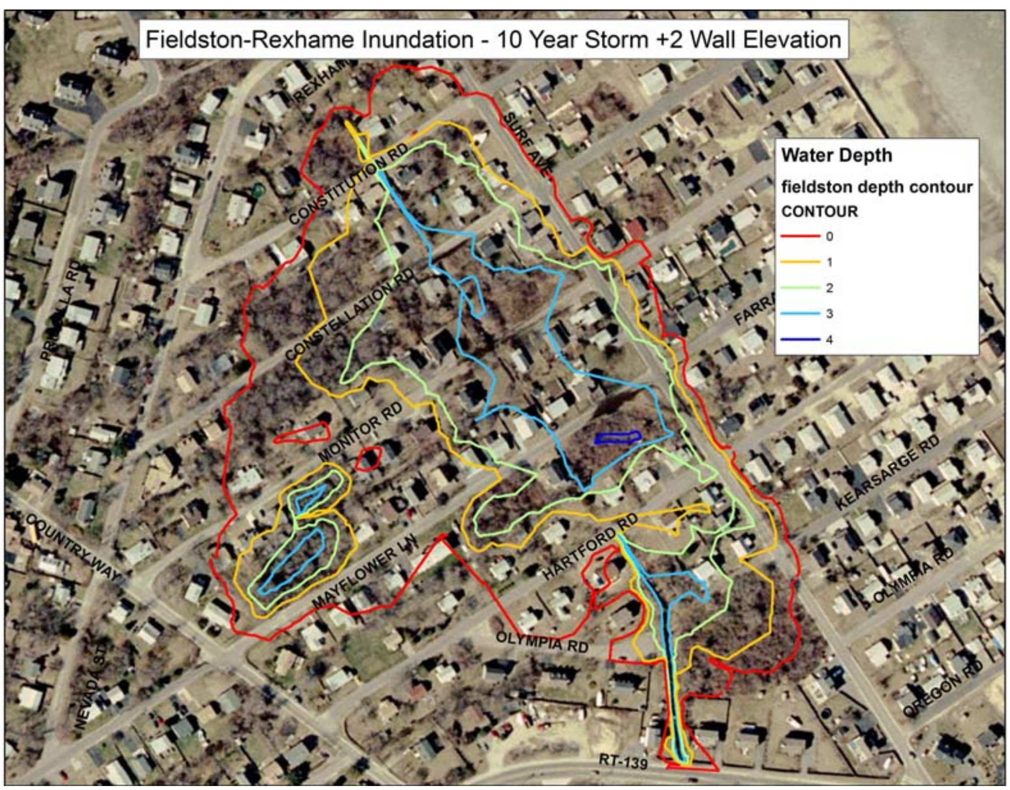

Figure 27. Fieldston Inundation for 10 Year Storm +2' on seawall

5.0 Conclusions and Summary

As part of an initial appraisal for this Continuing Authority Program (CAP) Section 14 – Storm Damage Reduction Project a wave overtopping/flooding depth analysis was conducted at two locations within Marshfield, MA. The locations were the Fieldston/Rexhame area and the Brant Rock area. Both locations were found to be impacted by lower level coastal storm events (1 to 10 year events) and that the wave overtopping volumes over the existing seawalls were significant enough to cause flooding. Both locations were relatively low in elevation and suffer from poor drainage, which allowed the wave overtopping to be trapped in these areas.

Since this investigation was only for an initial appraisal only one viable alternative was analyzed to determine the potential for flood impact reduction. The seawall was raised by two feet at both locations and the overtopping/flooding analysis was redone. It was found that for both areas there was significant reduction in flood depths for the 1 to 2 year events. Additionally it was found that at the Fieldson/Rexhame area there were reductions in flood depths up to the 10 year event. If this project does move forward into a feasibility level study several more alternatives will be considered along with a much more complete analysis of the nearby "system".