February 21, 2020

Maurice Goulet
Superintendent of Public Works
55 North Meadows Road
Medfield, MA 02052
mgoulet@medfield.net

Re: **Danielson Mill Dam (MA03351)**
Hazard Potential Classification Review
Medfield, Massachusetts
Pare Project Number: 19126.00

Dear Mr. Goulet:

The following report presents the Hydrologic and Hydraulic (H&H) analysis and hazard classification review completed by Pare Corporation (Pare) for the Danielson Mill Dam in Medfield, Massachusetts. In accordance with our scope of work, hydrologic and hydraulic evaluations and dam break analyses were completed to determine potential impacts to infrastructure in the downstream area associated with a failure of the Danielson Mill Dam.

EXECUTIVE SUMMARY

In accordance with 302 CMR 10.06 (6) Hazard Reconsideration, an owner may at any time request the Commissioner to reconsider the hazard determination. Given the apparent limited incremental impacts associated with a failure of the dam due to potential attenuation by the Homestead Drive roadway embankment, Pare reviewed the potential impacts associated with failure of the dam

Based on the model results, Danielson Mill Dam appears to be appropriately classified as a Significant Hazard potential dam. This finding is primarily supported by the model results indicating that a failure of the dam at during the 25- and 50-year storms is likely to result in overtopping of heavily traveled roads including Spring Street (Route 27) and South Street. In both of these cases, the non-failure scenario does not overtop the roads, as such, the incremental damage associated with overtopping of the roadways during a failure event justifies the current hazard potential classification of "Significant".

BACKGROUND

Danielson Mill Dam is currently classified as a **Small** size**, Class II (Significant)** hazard potential structure. In accordance with current state dam safety regulations, the spillway design flood (SDF) for the site is the 100-year storm event. Recent reports have suggested that the impacts associated with failure of the dam may be limited due to potential attenuation by the Homestead Drive roadway embankment downstream of the dam.

Given the current condition of the dam and the uncertain extents of the impacts resulting from dam failure, the owner has requested that Pare review the hazard potential classification of the dam through the completion of a detailed H&H and dam break analyses. The H&H analyses assessed the dam's ability to accommodate storm events up to the current SDF event (100-year storm) and the incremental damage of dam failure during the 25-, 50- and 100-year events. The following sections discuss the development of the hydrologic and hydraulic model(s) that were used for the hydrologic and hydraulic analysis.

All elevations reported within this analysis approximately reference NAVD88 based upon LiDAR data available from MassGIS.

HYDROLOGIC & HYDRAULIC ANALYSIS

Rainfall: Rainfall depth values for storms ranging between the 1-year and 500-year frequencies were obtained from the National Oceanic and Atmospheric Administration (NOAA) Atlas 14 rainfall data, included within the table below, and imported directly into the HydroCAD Version 10.00 software. Local hydrologic data was incorporated into the study by generating synthetic rainfall distribution curves for the site specific location using local precipitation frequency and rainfall distributions downloaded directly from NOAA.

Table 1: 24-hour Mean Precipitation Frequency Estimates.

Return Interval	Rainfall Depth (ft.)
1-year	2.78
2-year	3.42
5-year	4.45
10-year	5.31
25-year	6.49
50-year	7.36
100-year (SDF)	8.31
200-year	9.47
500-year	11.30
1,000-year	12.80

Rainfall-Runoff Model (HydroCAD): A rainfall-runoff model was developed in HydroCAD (Version 10.00-17) using TR-55 Curve Number (CN) methodologies, the Soil Conservation Service (SCS) TR-20 runoff method, and the Natural Resources Conservation Service (NRCS) Unit Hydrograph (UH) and time of concentration methodologies.

The NRCS UH method utilizes a best-approximate hydrograph based on the gamma equation (peak rate factor) that is scaled by the time lag to produce the unit hydrograph for the simulation. The lag is defined as the length of time between the centroid precipitation mass and the peak flow of the resulting hydrograph. The Watershed Lag Method was chosen to estimate the lag times for the watershed(s). The lag method uses the longest flow length, average watershed slope, and the maximum potential retention to determine the lag time. The time of concentration for the watershed was developed using the using the lag-CN method for inflow into the pond, which is a function of the land slope, curve number, and hydraulic length of the watershed, and the shallow wave equation for flow across the pond.

The CN represents the portion of the precipitation depth that will lead to runoff, and is a function of soil type (Hydrologic Soil Groups) and ground cover within the drainage area. A high CN (e.g., 98 for pavement/water surface) indicates a material that will have low water retention and a high runoff value. A low CN (e.g., 30 for a wooded area with permeable soils) indicates high infiltration and low runoff volume. The composite curve number (CN) used in the model of each watershed is an average CN value weighted by the percent area of each contributing land-use and soil type in the watershed.

To develop the CN, ArcGIS software was used to join GIS based NRCS Soil Survey data layers and MassGIS based Land Use data layers into a single data layer with discrete areas of unique soil groups and land use data. A CN Look-Up table (specific for Massachusetts) was developed and used HEC-geoHMS to automate CN values for each discrete area. The composite CN utilized for each a single watershed or sub-watersheds is an area-weighted average of the CN values contained within the limits of the drainage area.

Drainage Area / Watershed: The drainage area for the watershed was delineated utilizing Massachusetts Streamstats and the HEC-geoHMS (add-on for ArcGIS) using GIS based terrain data generated with available MassGIS LiDAR data. Given the small size of the drainage area for the dam and no apparent other control structures within the drainage area, the drainage area was modeled as a single watershed. The following table presents the area, CN, and time of concentration for the watershed.

Table 2: Danielson Mill Dam Watershed- Hydrologic Parameters

Watershed Name	Area (acres / mi^2)	Weighted Curve Number	Average Land Slope (ft./ft.) (150k DEM)	Longest Flow Path (ft.)	Time of Concentration (minutes)	NEH Gamma Value
DA	308 / 0.5	58	0.025	4300	118	300

A baseflow of 1 cfs was utilized for this drainage area as determine by the recommended ratio of 1.5 cubic feet per second (cfs) of flow for every square mile of drainage area.

Stage-Storage Relationship: LiDAR was used to compute the above Lidar normal pool (El. 145 and above) stage-storage curve, while estimates of the pond bathymetry and assumed pond bathymetry were used within HydroCAD to develop the below normal pool stage-storage curve. The two curves were combined and input into HydroCAD as an impoundment. The following table summarizes the storage characteristics for the impoundment:

Table 3: Danielson Pond- Stage Storage Relationship

Elevation (feet, NGVD88)		Storage Volume (acre-feet)
Pond Bottom	140.9	0
	145	24
Normal Pool	145.6	31
	146	34
Top of Dam	147.2	44
	148.5	57
	149.0	62

Danielson Mill Dam: The dam was modeled with two outlets; the primary spillway system, and overtopping of the dam crest. The primary spillway system was modeled with stoplogs routed to the downstream area. The stoplogs were modeled as a sharp crested weir with a length of 4 feet with a control elevation of El. 145.60 and two end contractions which assumes no operations of stop logs occurred prior to storms. This elevation was based on a relative elevation survey completed by Pare. Overtopping of the dam crest was modeled as an asymmetrical weir at El 147.2. Flow over the crest was modeled as free discharge to the downstream channel. Through sensitivity analysis using HydroCAD, it was determined that tailwater effects from the 4-foot culvert at Homestead Drive immediately downstream of the dam did not impact the outflows from the dam.

H&H Model Results: The following table summarizes the results of the model:

Table 4: H&H Model Results

	25-yr Storm	50-yr Storm	100-yr Storm
Peak Inflow (cfs)	125	165	210
Peak Spillway Outflow (cfs)	32	34	36
Peak Elevation (ft)	147.4	147.5	147.6
Peak Overtopping Flow (cfs)	73	117	167
Peak Overtopping Depth (ft)	0.2	0.3	0.4

Assumes starting water surface elevation at Normal Pool El. 145.6 and no operations completed

DAM BREAK MODELING

A dam break model was prepared using a HEC-RAS 5.07 2D dam break model. The 2D program solves the depth-averaged dynamic wave equation with an implicit finite volume numerical method to provide a two-dimensional representation of the breach flood wave in the downstream area.

Terrain Development: The computational mesh for the evaluation will be based on a terrain model developed from available MassGIS statewide elevation data (LiDAR) and simplifying assumptions regarding variations in Mannings roughness (composite Mannings n value), cell sizes, and a reduction in manual two-dimensional break lines.

The computational mesh for the evaluation was based on LiDAR terrain data available from MassGIS. A variable manning's N value was developed utilizing available land use data (NLCD 2011) refined in areas of interest and/or where needed (i.e. stream channels) along with published correlations between land use and manning's N values (HEC-RAS and NRCS Kansas). The computational mesh cell size was typically 50-feet within the inundation areas with break lines and refinement regions used to refine the mesh in areas where appropriate (i.e. tops of embankments, etc.).

Pertinent dam data described in the H&H analysis was incorporated into the dam break model. Field data collection and engineering judgment based on available aerial imagery and LiDAR terrain data was used to model pertinent information at each of the downstream river crossings extending from the dam (Danielson Mill Dam) to the downstream limits of the model at the confluence with the Stop River.

Each stream crossing in the downstream area was modeled as a weir roadway embankment with the level of the roadway determined from the LiDAR terrain with a culvert outlet.

- The Homestead Drive roadway embankment that is located approximately 120 feet downstream of the dam was included within the 2D model in order to determine the effects of any developing tailwater as a result of the roadway embankment.
- The Spring street culvert was modeled as a 4-foot concrete culvert at El. 133.9 as determined during the relative elevation survey and field data collection.

A normal depth boundary condition was assigned at the downstream limits of the model downstream of Spring street, prior to the confluence with the Stop River.

Dam Break Model: For the purposes of this evaluation, two dam break models scenarios were completed including a sunny day (fair weather) condition and a wet weather condition.

- The sunny day flows were determined from the size of the contributing drainage areas of each inflow location and the published approximate correlation of 1.5 cfsm (CFS/SM) and were increased as needed for model minimums. Sunny Day Dam failure was initiated within the model once conditions within the downstream area stabilized from base flow conditions. A piping failure mode was chosen for the dam as the dam is not expected to overtop during sunny day conditions. Conservatively, and in accordance with Massachusetts Department of Conservation and Recreation Office of Dam Safety guidance, a clogged spillway was assumed with the level of the impoundment raising to the crest of the dam prior to initiation of dam failure.

- Wet weather failure during the 25-, 50-, and 100-year events was initiated once the impoundment reached its peak water surface elevation and an overtopping failure mode was chosen because the dam is expected to overtop during these events with no operations.

The dam breach hydrographs routed to the downstream area computational mesh were developed from empirical methods using the built-in dam breach plan and parameter calculation workflows within the HEC RAS software. The breach parameters for both scenarios are included within the table below.

Table 5- Breach Parameters

	Sunny Day Failure	Wet Weather Failure
Method	Frohlich 2008	Frohlich 2008
Failure Mode:	Piping	Overtopping
Breach Bottom Width (ft):	25	25
Breach Bottom Elevation (NAVD88)	140.4	140.4
Breach Side Slopes (H:V)	0.7H:1V	0.7H:1V
Breach Formation (hours)	0.63	0.63
Storage at Failure (Acre-Feet)	44	44
Water El. at Failure (NAVD88)	147.2	147.6

In order to complete a hazard class review, an incremental damage assessment was completed using a dam break model by assessing the flooding downstream during the 25-,50-, and 100-year (SDF) events under both dam failure and non-failure scenarios. A sunny-day dam failure model was also created to illustrate inundation from normal pool storage.

Inundation areas were developed for each of the models described above. Extents of the modeled inundation for the entire study limits are presented on Figure 3A, 3B, 3C, 3D. The following describes the impacts to the downstream areas within the study limits during various flow/failure conditions:

Table 6: Dam Break Model Results

		Modeled Event						
		Sunny Day Failure	25-yr Event		50-yr Event		100-yr Event	
			Non Failure	Failure	Non Failure	Failure	Non Failure	Failure
Danielson Mill Dam	Peak Flow (cfs)	270	105	525	150	530	203	960
	Peak Overtopping Depth (ft)	NA	0.2	0.2	0.3	0.3	0.4	0.4
Homestead Drive	Peak Flow (cfs)	145	105	180	145	180	180	180
	Peak Overtopping Depth (ft)	NA	NA	NA	NA	NA	NA	NA
Spring Street	Peak Flow (cfs)	105	87	140	120	145	145	150
	Peak Overtopping Depth (ft)	NA	NA	**1.0**	NA	**1.3**	1.3	1.3

As indicated in the table, Homestead Drive functions to significantly attenuate breach flows due to dam failure with the capacity of the culvert beneath Homestead Drive reducing flow to reaches further downstream from the peak breach discharge to 180 cfs. However, while the culvert at Spring Street can accommodate flows in excess of the 50-year storm non-failure event, the Spring Street culvert cannot accommodate the 180 cfs discharges through the Homestead Drive culvert, resulting in overtopping of Spring Street during failure events that would not occur if the dam did not fail.

FINDINGS

Based on the model results, Danielson Mill Dam appears to be appropriately classified as a Significant Hazard potential dam. This finding is primarily supported by the model results indicating that, while Spring Street would not overtop during storm events in excess of the 50-year non-failure event, Spring Street would be overtopped during the same events should the dam fail.

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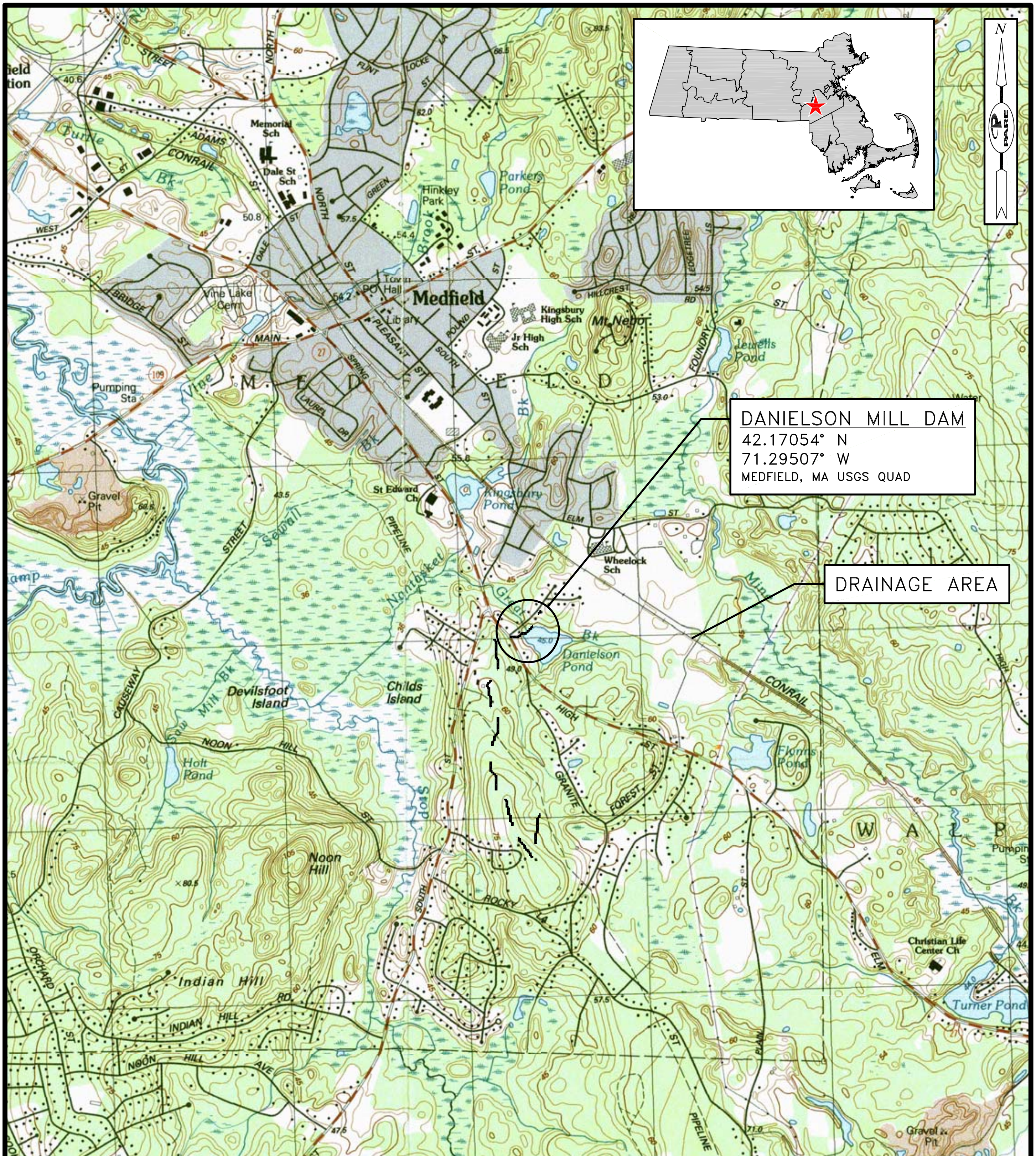
We trust this summary of the hazard classification review completed as part of this study will meet your needs at the current time. Should you have any questions or require any additional information please do not hesitate to contact me at 508.543.1755 or by email at aorsi@parecorp.com

Sincerely,
PARE CORPORATION

Allen R. Orsi, P.E.
Vice President

Andrew Dasilva, E.I.T.
Engineer II

Attachments:
Figure 1: Locus Plan
Figure 2: Aerial Plan
Figure 3A: Sunny Day Inundation Map
Figure 3B: 25-Year Inundation Map
Figure 3C: 50-Year Inundation Map
Figure 3D: 100-Year Inundation Map



USGS TOPOGRAPHIC MAP FROM OFFICE OF GEOGRAPHIC AND ENVIRONMENTAL INFORMATION (MASSGIS), COMMONWEALTH OF MASSACHUSETTS EXECUTIVE OFFICE OF ENVIRONMENTAL AFFAIRS

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DANIELSON MILL DAM

MA03351

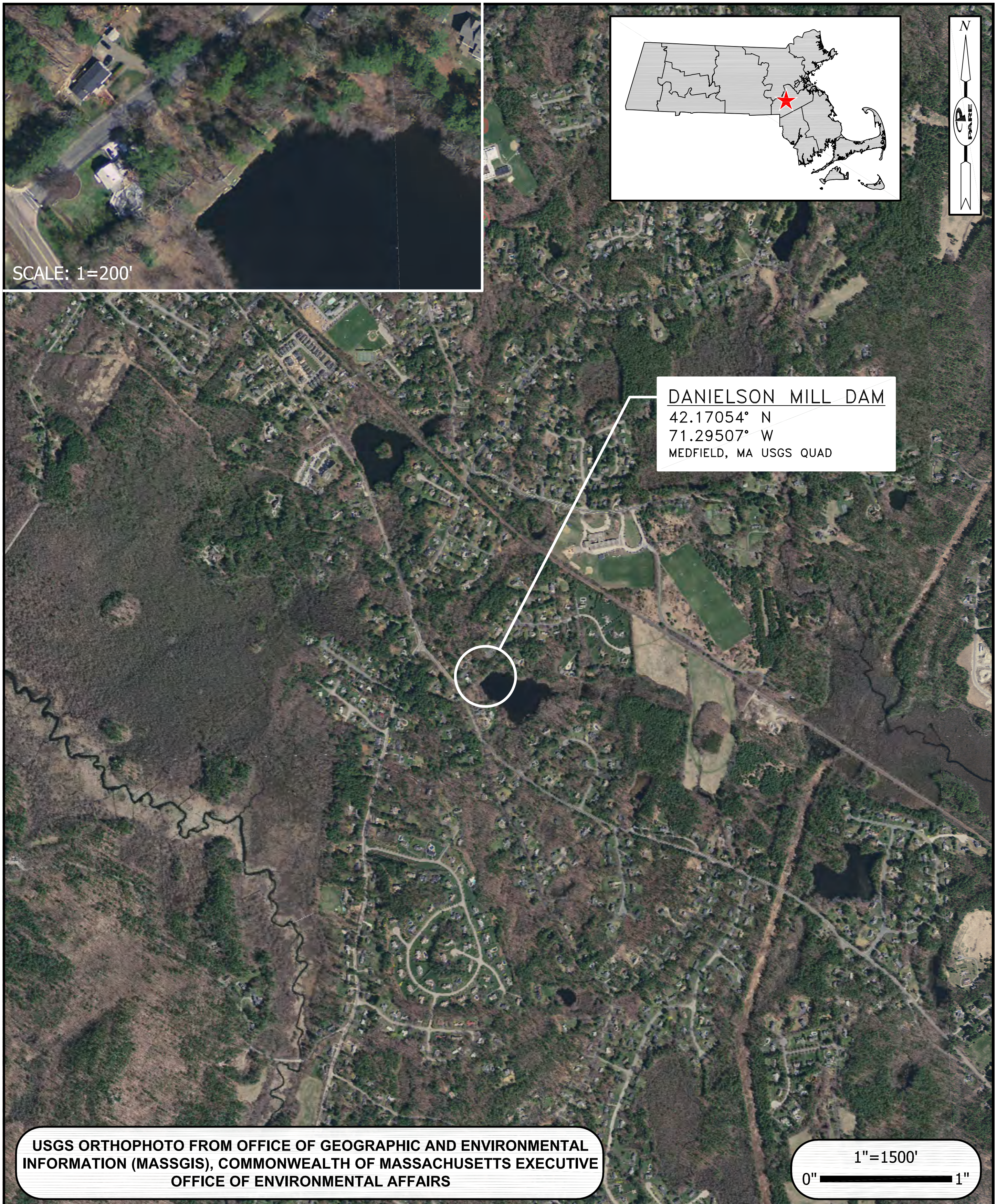
MEDFIELD, MASSACHUSETTS

OWNER : TOWN OF MEDFIELD

LOCUS PLAN

FEBRUARY 2020

FIGURE 1



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DANIELSON MILL DAM
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MEDFIELD, MASSACHUSETTS
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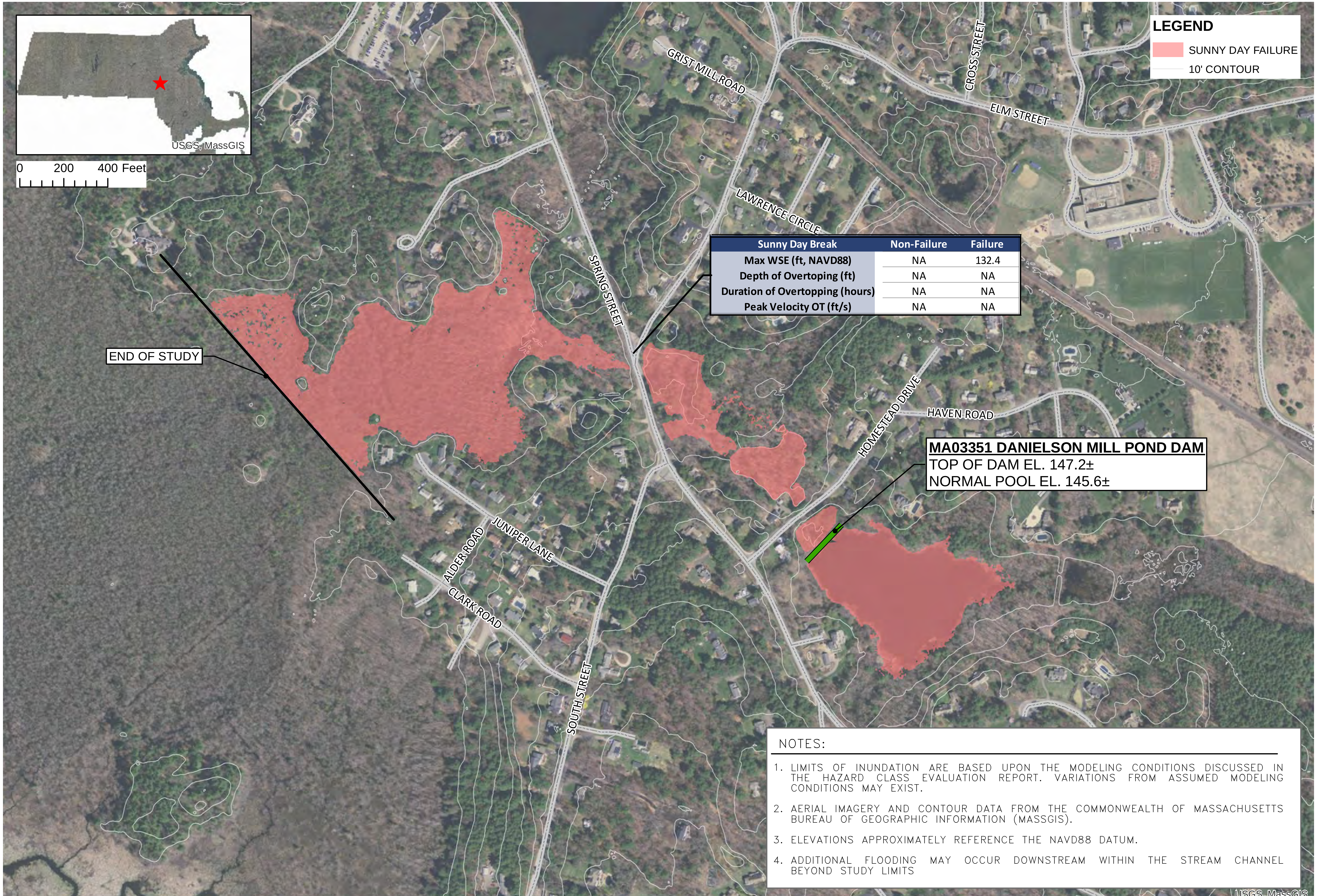
AERIAL PLAN

FEBRUARY 2020 FIGURE 2



Sunny Day Break	Non-Failure	Failure
Max WSE (ft, NAVD88)	NA	132.4
Depth of Overtoping (ft)	NA	NA
Duration of Overtopping (hours)	NA	NA
Peak Velocity OT (ft/s)	NA	NA

NOTES:

1. LIMITS OF INUNDATION ARE BASED UPON THE MODELING CONDITIONS DISCUSSED IN THE HAZARD CLASS EVALUATION REPORT. VARIATIONS FROM ASSUMED MODELING CONDITIONS MAY EXIST.
2. AERIAL IMAGERY AND CONTOUR DATA FROM THE COMMONWEALTH OF MASSACHUSETTS BUREAU OF GEOGRAPHIC INFORMATION (MASSGIS).
3. ELEVATIONS APPROXIMATELY REFERENCE THE NAVD88 DATUM.
4. ADDITIONAL FLOODING MAY OCCUR DOWNSTREAM WITHIN THE STREAM CHANNEL BEYOND STUDY LIMITS

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DANIELSON MILL POND DAM

MA03351 MEDFIELD, MA

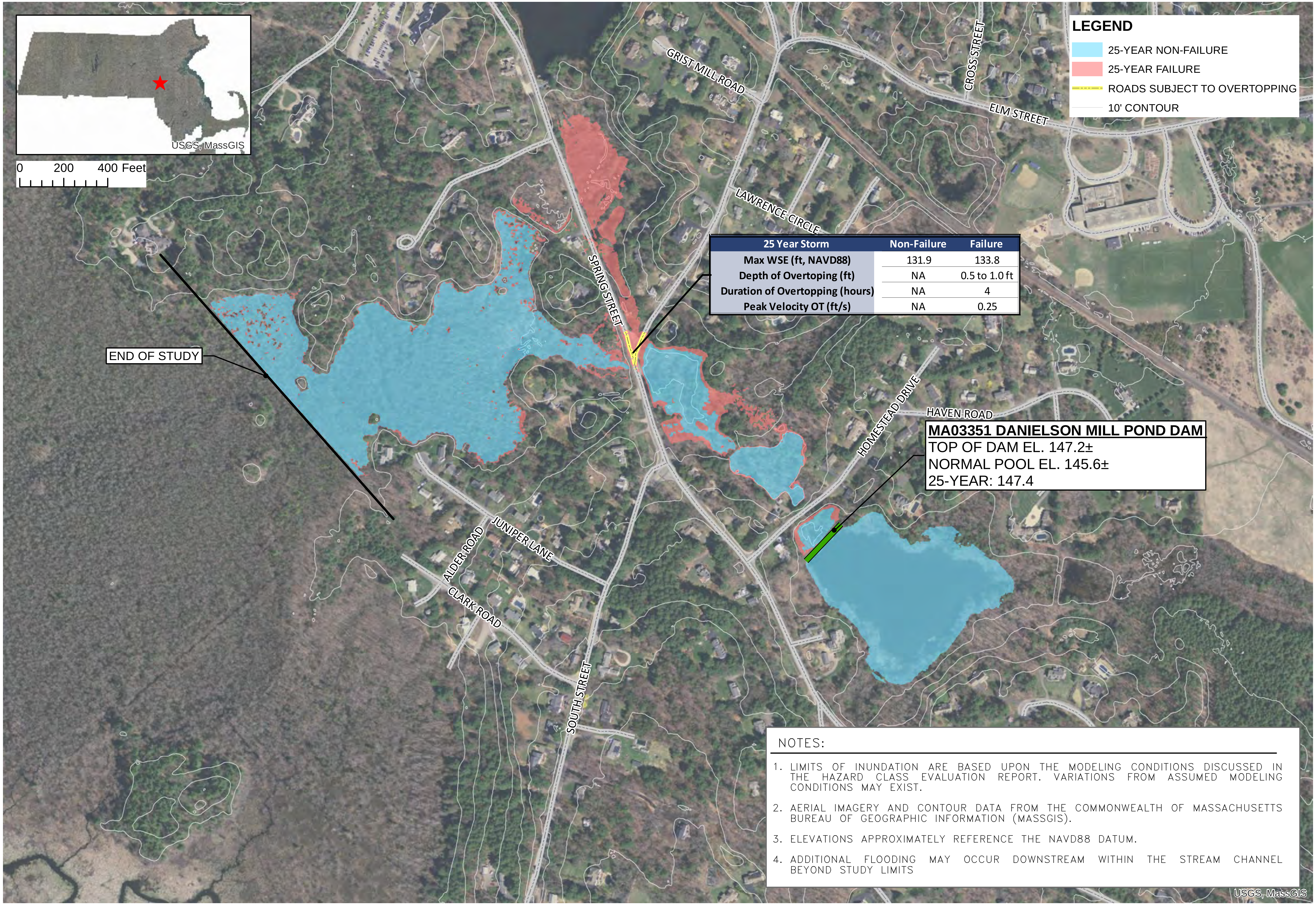
HAZARD CLASS EVALUATION

OWNER: TOWN OF MEDFIELD

PREPARED FOR: MASSACHUSETTS DEPARTMENT OF CONSERVATION AND RECREATION

PROJECT NO.: 19126.00
DATE: FEBRUARY 2020
SCALE: AS NOTED
DRAWN BY: SJP
APPROVED BY: AO

FIGURE 3A
SUNNY DAY
INUNDATION
MAP



25 Year Storm	Non-Failure	Failure
Max WSE (ft, NAVD88)	131.9	133.8
Depth of Overtoping (ft)	NA	0.5 to 1.0 ft
Duration of Overtopping (hours)	NA	4
Peak Velocity OT (ft/s)	NA	0.25

NOTES:

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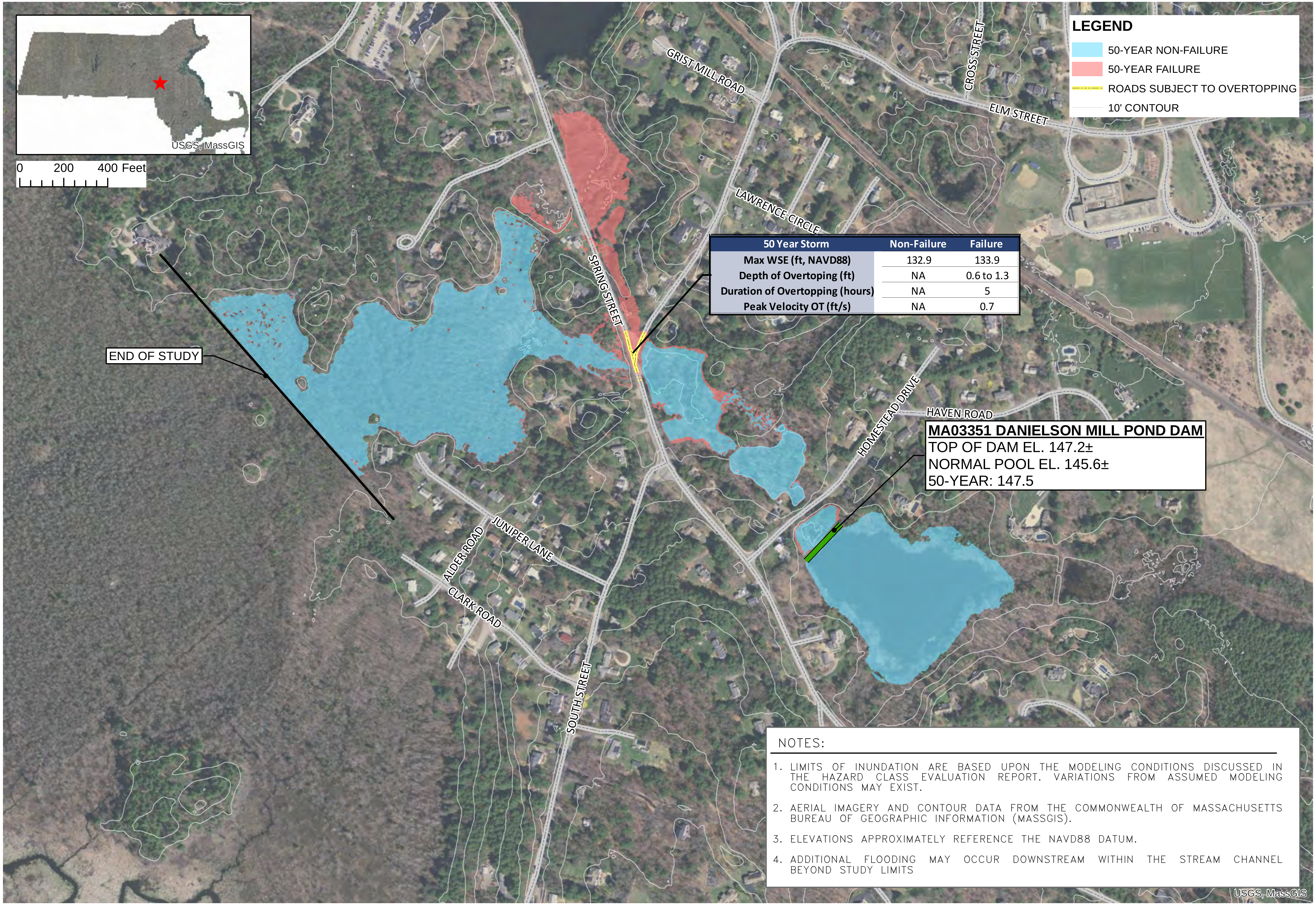
FIGURE 3B
25 YEAR
INUNDATION
MAP



50 Year Storm	Non-Failure	Failure
Max WSE (ft, NAVD88)	132.9	133.9
Depth of Overtoping (ft)	NA	0.6 to 1.3
Duration of Overtopping (hours)	NA	5
Peak Velocity OT (ft/s)	NA	0.7

NOTES:

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OWNER: TOWN OF MEDFIELD

PREPARED FOR: MASSACHUSETTS DEPARTMENT OF CONSERVATION AND RECREATION

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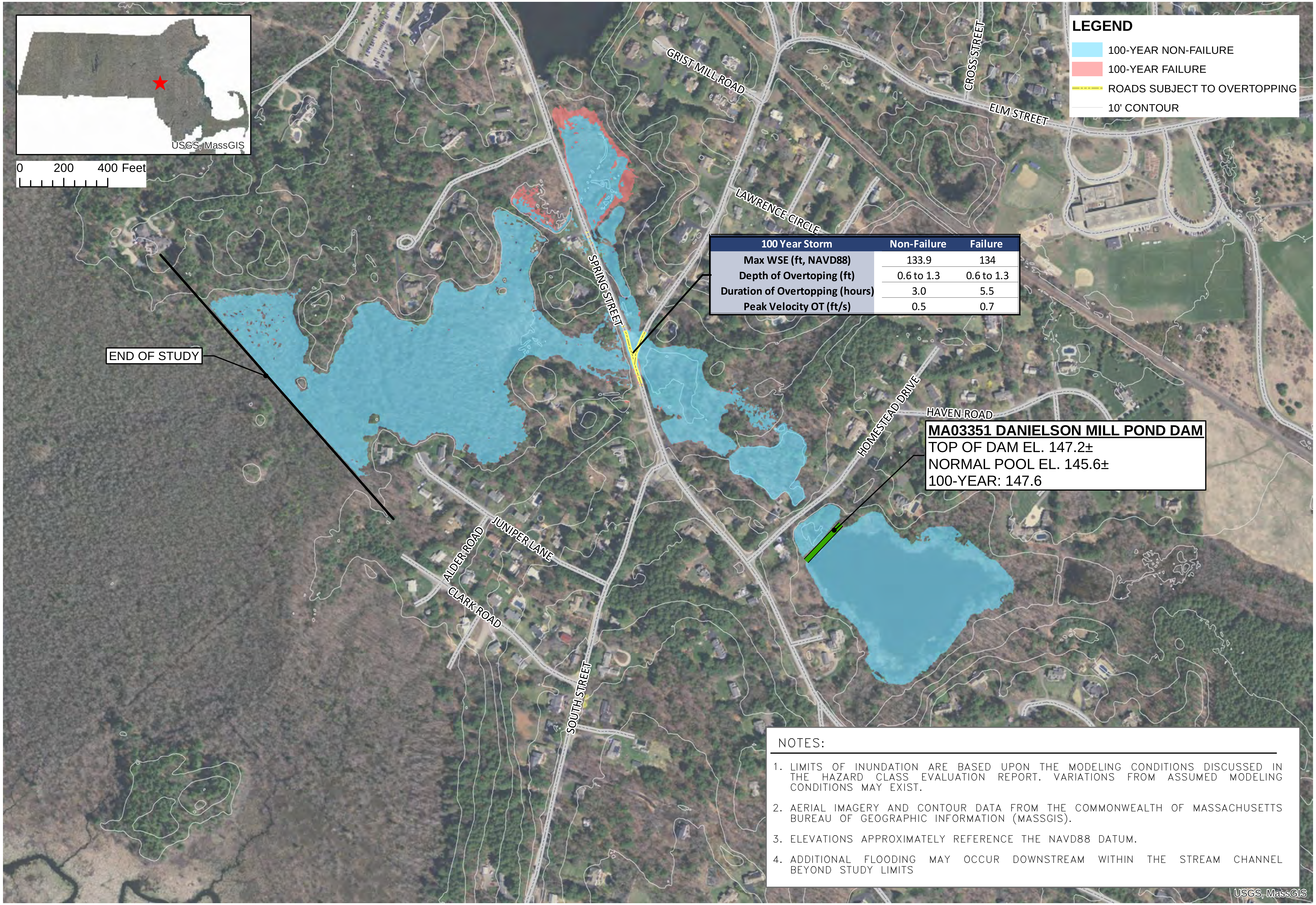
FIGURE 3C
50 YEAR INUNDATION MAP



100 Year Storm	Non-Failure	Failure
Max WSE (ft, NAVD88)	133.9	134
Depth of Overtoping (ft)	0.6 to 1.3	0.6 to 1.3
Duration of Overtopping (hours)	3.0	5.5
Peak Velocity OT (ft/s)	0.5	0.7

NOTES:

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FIGURE 3D
100 YEAR INUNDATION MAP