Horsley Witten Group
Sustainable Environmental Solutions
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MEMORANDUM

To: Kristopher Houle, DER

From: Neal Price

Date: June 30, 2021

Re: Ipswich Mills Dam Removal – Summary of Spring 2021 Project Work: EBSCO Geophysical Study, EBSCO-Area Hydrologic Evaluation, and In-River Sediment Properties

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The Horsley Witten Group, Inc. (HW) is pleased to submit to the Massachusetts Division of Ecological Restoration (DER) the following memorandum summarizing recent work completed in the spring of 2021 on the Ipswich Mills Dam Removal Feasibility Project (the Project). These work items included:

- An evaluation of the groundwater and river elevations in the vicinity of the EBSCO facility and the Ipswich Mills dam.
- A characterization of in-river sediment physical properties (grain size and thickness) at key locations along the length of the river from the Choate Bridge upstream to the Railroad Bridge (Figure 1).

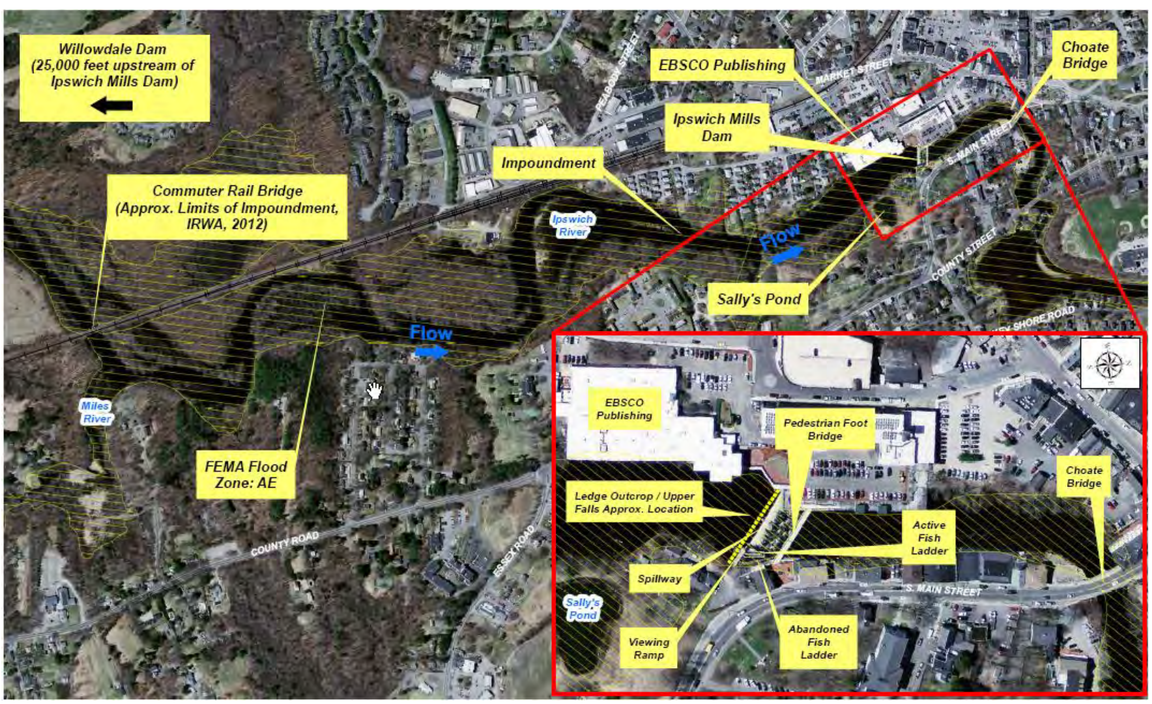


Figure 1. Key Project Area Features

- Follow-up geophysical exploration of the subsurface conditions beneath the interior of the EBSCO former mill building (as shown in the inset of Figure 1), located immediately adjacent to the river's left bank just upstream from the dam. Prior geophysical exploration of the interior and the exterior of the EBSCO facility was conducted in 2020.
- An update of the structural implications for the EBSCO building from potential dam removal based on the 2020 and 2021 geophysical studies.

All elevations discussed in this memorandum and supporting attachments are reported relative to the NAVD88 datum, in feet.

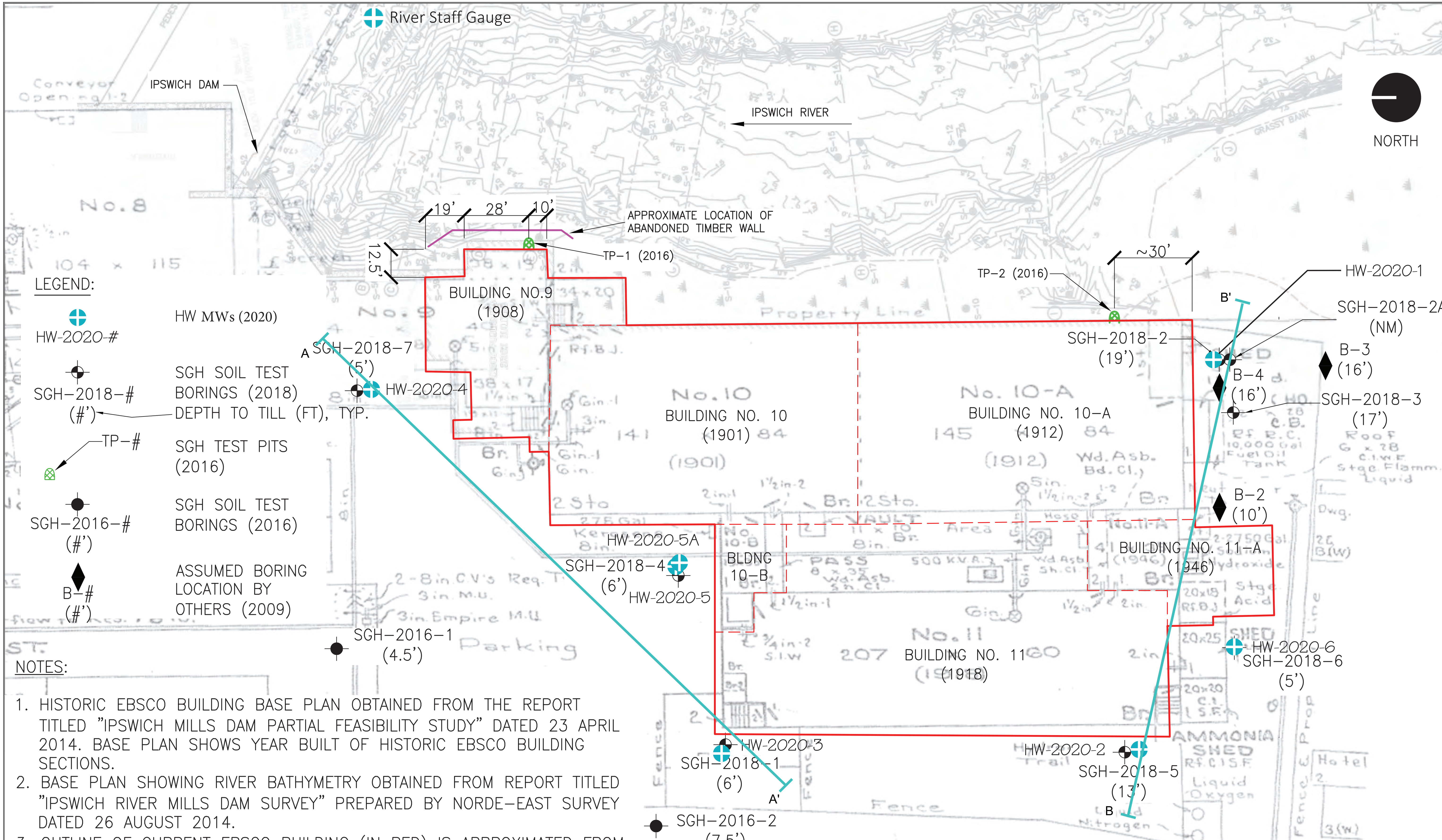
Hydrologic Evaluations

Time variable river level and groundwater levels (using the six monitoring wells installed on the EBSCO property in 2020) (Table 1 and Figure 2) were monitored from September 1, 2020 to June 23, 2021. Manual water levels were measured in each of the six wells and at a river staff gauge immediately upstream of the Ipswich Mills Dam a total of 15 times over this time period by DER and Ipswich River Watershed Association (IRWA) staff. Manual water level data collected over this time period are summarized in Table 2. In addition, continuous water level data loggers were installed in monitoring well HW-2020-1 and the river for the time period between September 1 and December 30, 2020. Unfortunately, the river data logger malfunctioned after deployment and no usable data were able to be retrieved from it.

Table 1. Subsurface Borings Key Information Summary from 2020

	.	Till		Bedrock		Groundwater#		Soft Sediments Interval	
HW Well	**Approx. Ground Elev.**	BGS	~Elev.	BGS	~Elev.	BGS	~Elev.	BGS	~Elev.
2020-1	13	19*	-6	27	-14	5	8	3 to 19*	10 to -6
2020-2	16	13*	3	24	-8	8	5	NA	NA
2020-3	17	6*	11	16	1	8	9	NA	NA
2020-4	12	5*	7	8	4	6	6	NA	NA
2020-5	16	6	10	14	2	9	7	NA	NA
2020-6+	13	5*	8	NA	NA	4	9	2.5 to 5*	10.5 to 8

BGS = Feet Below Ground Surface. All depths approximate to nearest foot.
Elev. = Approximate elevation in feet (NAVD88 datum) based on MassGIS LIDAR topography
NA = Not Applicable. Soft sediment or Bedrock not encountered.
+HW-2020-6 not drilled to bedrock refusal due to time constraints.
* Depth to Till or Soft sediment from SGH 2016 boring log (if that begins above first 2020 sample interval).
# Depth to groundwater following drilling approximate due to drilling influence on water levels.



NOTES:

1. HISTORIC EBSCO BUILDING BASE PLAN OBTAINED FROM THE REPORT TITLED "IPSWICH MILLS DAM PARTIAL FEASIBILITY STUDY" DATED 23 APRIL 2014. BASE PLAN SHOWS YEAR BUILT OF HISTORIC EBSCO BUILDING SECTIONS.
2. BASE PLAN SHOWING RIVER BATHYMETRY OBTAINED FROM REPORT TITLED "IPSWICH RIVER MILLS DAM SURVEY" PREPARED BY NORDE-EAST SURVEY DATED 26 AUGUST 2014.
3. OUTLINE OF CURRENT EBSCO BUILDING (IN RED) IS APPROXIMATED FROM MASS GIS ONLINE VIEWER AND LIMITED FIELD MEASUREMENTS NEAR TEST PITS.
4. DASHED LINES INDICATING EXTENTS OF HISTORIC EBSCO BUILDING SECTIONS ARE APPROXIMATE.
5. NM = NOT MEASURED

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Project:	FEASIBILITY STUDY FOR IPSWICH MILLS DAM REMOVAL IPSWICH, MA
Title:	EBSCO FACILITY SUBSURFACE INVESTIGATION LOCATION PLAN
Drawn:	
Checked:	
Approved:	
Project No.:	
Drawing No.:	FIG. 2
Scale:	1"=40'-0"
Date:	06/30/2021

Table 2. Manual Groundwater and Ipswich Mills Dam River Level Data (in NAVD88 Feet)

Date	Time	Staff Gauge	Time	HW-2020-1	Time	HW-2020-2	Time	HW-2020-3	Time	HW-2020-4	Time	HW-2020-5	Time	HW-2020-6
9/1/2020	14:00	8.87	10:56	8.79	10:46	9.10	10:41	9.07	10:25	7.31	10:36	8.23	10:50	8.96
9/17/2020	10:00	8.95	10:35	8.87	10:28	8.97	10:21	8.97	10:14	7.34	10:17	8.19	10:32	8.93
10/6/2020	10:23	8.83	10:12	8.77		Not acc	10:07	8.71	10:02	7.28	10:05	8.19	10:10	8.73
10/7/2020					10:31	8.75								
10/22/2020	9:49	9.25	9:41	9.25	9:32	9.30	9:27	8.76	9:15	7.41	9:21	8.28	9:36	9.32
11/4/2020	2:30	9.41	2:23	9.42	2:18	9.29	2:15	9.49	2:10	7.98	2:12	8.29	2:21	9.63
11/17/2020	2:25	9.26	2:19	9.37	2:13	9.25	2:10	9.21	2:05	8.60	2:07	8.15	2:16	9.28
12/2/2020	9:51	9.72	9:43	9.85	9:25	9.79	9:36	9.72	9:29	9.62	9:33	8.38	9:40	9.89
12/16/2020	9:42	9.65	9:37	9.71	9:30	9.68	9:25	9.55	9:15	8.41	9:20	8.30	9:34	9.75
12/30/2020	11:25	9.99	10:53	10.14	10:46	10.06	11:00	9.98	11:10	7.79	11:08	8.30	10:51	10.05
1/27/2021	10:01	9.47	9:46	9.50	9:15	9.59	9:34	9.42	9:22	7.49	9:27	8.22	9:41	9.57
2/24/2021	9:46	9.68	9:22	9.74	9:11	9.86	9:06	9.77	9:00	10.28			9:18	9.85
3/24/2021	11:22	9.65	10:15	9.67	9:43	9.70	10:52	9.56	9:28	7.79	9:35	8.27	10:45	9.68
4/21/2021	9:55	10.11	9:44	10.11	9:35	9.98	9:31	9.89	9:21	8.24	9:27	8.34	9:39	10.01
5/19/2021	9:53	9.65	9:46	9.64	9:43	9.76	9:35	9.56	9:39	7.55	9:50	8.24	9:50	9.70
6/23/2021	9:48	9.41	9:41	9.41	9:33	9.59	9:30	9.48	9:19	9.41	9:25	8.31	9:38	9.54

To compensate for the missing river level data logger data, the 15 available manual river elevation data points were correlated against the simultaneous river level data from the USGS Ipswich River Near Ipswich, MA Gauge (USGS Streamflow Station 01102000) located approximately 4 miles upstream from the Ipswich Mills Dam and 200 feet downstream from the Willowdale Dam. This station is referred to as the USGS Willowdale Gauge and river levels for the gauge are reported as heights above the local gauge datum. That local gauge datum is 19.83 feet above the NAVD88 datum. The correlation between the USGS Willowdale Gauge and the Ipswich Mills Dam river levels produced an excellent linear correlation relationship with an R^2 correlation coefficient of 0.97. That relationship is included here on Figure 3A. The linear equation predicting the river level at the Ipswich Mills Dam (Y) based on the recorded river level at the USGS Willowdale Gauge (X) is:

$$\boxed{Y = 0.5242X + 7.6068}$$

Two additional correlations were made to investigate the relationships of data from well HW-2020-1 to longer-term USGS data. One correlation to well HW-2020-1 data was made for the simultaneous USGS Willowdale Gauge river level data and a second was made to the nearest in time available groundwater level data from USGS Index Well MA-TQW-1 in Topsfield. In one regard, a groundwater-to-groundwater comparison would be more appropriate for well HW-2020-1 than would river data from the Willowdale Gauge. However, the Topsfield Index Well is located approximately 2 miles further away from the Ipswich Mills dam than is the Willowdale Gauge, and water levels from the Topsfield well are only recorded monthly, as opposed to the continuous data available from the Willowdale Gauge.

The well HW-2020-1 to USGS Willowdale Gauge correlation also produced an excellent linear correlation relationship with an R^2 correlation coefficient of 0.98. That relationship is included here on Figure 3B. The linear equation predicting the groundwater level at well HW-2020-1 (Y) based on the recorded river level at the USGS Willowdale Gauge (X) is:

$$\boxed{Y = 0.5943X + 7.3817}$$

The well HW-2020-1 to USGS Index Well MA-TQW-1 correlation produced a significantly poorer linear correlation relationship with an R^2 correlation coefficient of 0.66. That relationship is included here on Figure 3C. The linear equation predicting the groundwater level at well HW-2020-1 (Y) based on the recorded river level at the USGS Index Well MA-TQW-1 (X) is:

$$\boxed{Y = 6.6801X + 56.906}$$

In some ways it is not surprising that the correlation of water levels from well HW-2020-1 is better for the USGS Willowdale Gauge than it is for the USGS Topsfield Index Well. The availability of only monthly data from the Index well resulted in some of the "simultaneous" pairs of nearest in time data compared being a week or more off. The Index Well is also further away from HW-2020-1 and the close proximity of HW-2020-1 to the impounded Ipswich River (within approximately 50 feet) creates a very close connection between river levels and groundwater levels there.

Based on these correlation results, long term data and statistics from the USGS Willowdale Gauge can be used for planning level evaluations of river and groundwater levels near the Ipswich Mills Dam. While the accuracy of these correlation equations for USGS Willowdale Gauge river levels outside of the range of data analyzed here (approximately 2-4 feet gauge height) will not be as robust as those shown here for data within that range, the relationships are still useful tools for planning level evaluations.

For example, using these correlation equations, the long-term historic record of water levels from the USGS Willowdale Gauge can be used to roughly estimate the similar historic range of river and groundwater levels for the area near the Ipswich Mills Dam. Unfortunately, while a long record of flow data from the USGS Willowdale Gauge is available online, river stage data is only available from October 1, 2007 through current date. Similarly, statistics of stage (e.g., average, minimum, maximum, and others) are not available. Figure 4 depicts the available record of river stage at the Willowdale gauge from October 1, 2007 to current time. Visual examination suggests that the approximate average stage over this time period (in round numbers) was 3.5 feet, an approximate representative high stage (not the maximum but a high value commonly experienced) was 4.5 feet, and a similarly representative low stage value was 2.5 feet. Those values, along with the corresponding estimated values for the Ipswich Mills Dam and HW-2020-1 (based on the above correlation equations) are shown in Table 3.

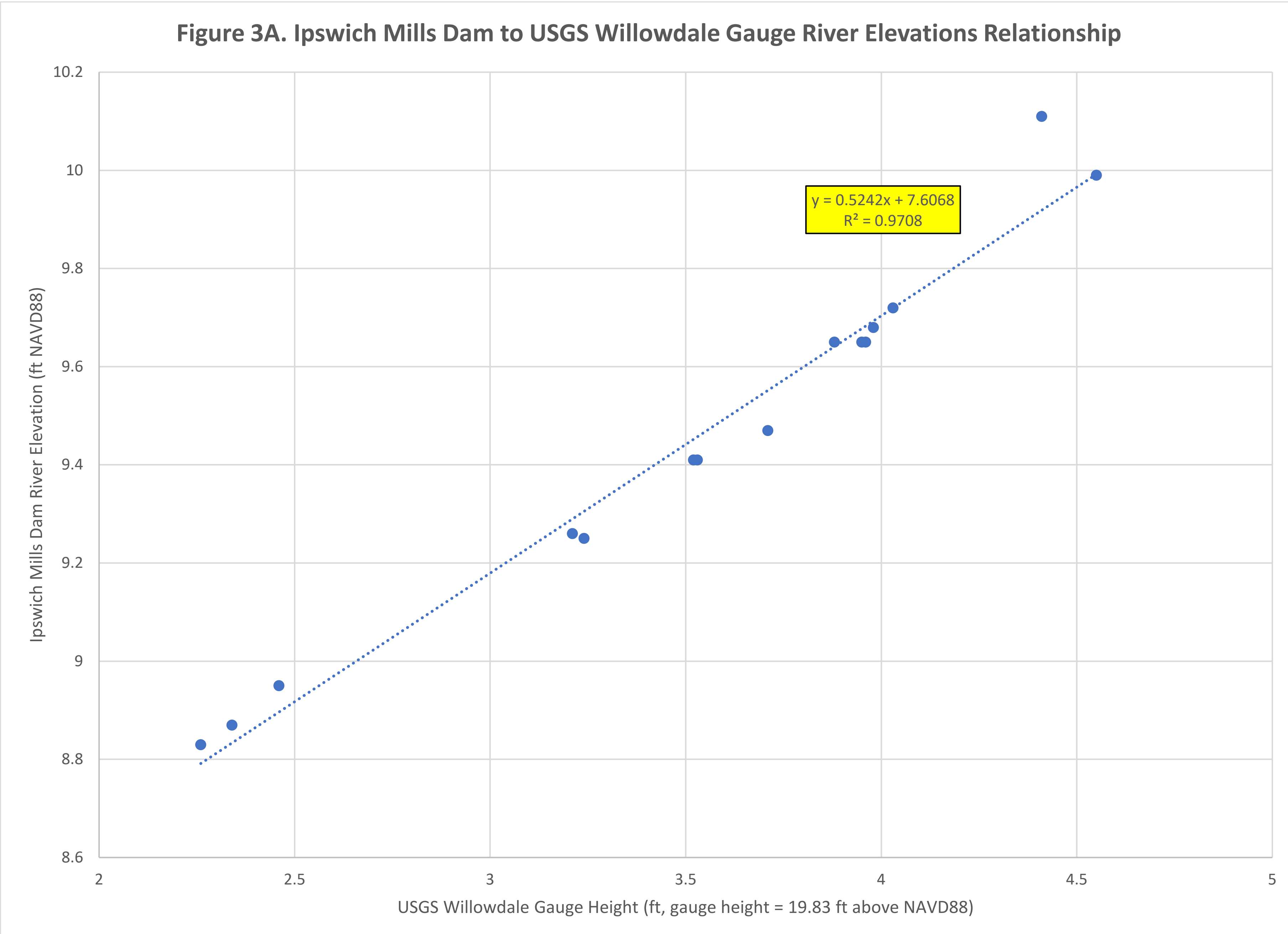


Figure 3A. Ipswich Mills Dam to USGS Willowdale Gauge River Elevations Relationship
10.2
10
9.8
9.6
9.4
9.2
9
8.8
8.6
Ipswich Mills Dam River Elevation (ft NAVD88)
y = 0.5242x + 7.6068
R² = 0.9708
2
2.5
3
3.5
4
4.5
5
USGS Willowdale Gauge Height (ft, gauge height = 19.83 ft above NAVD88)

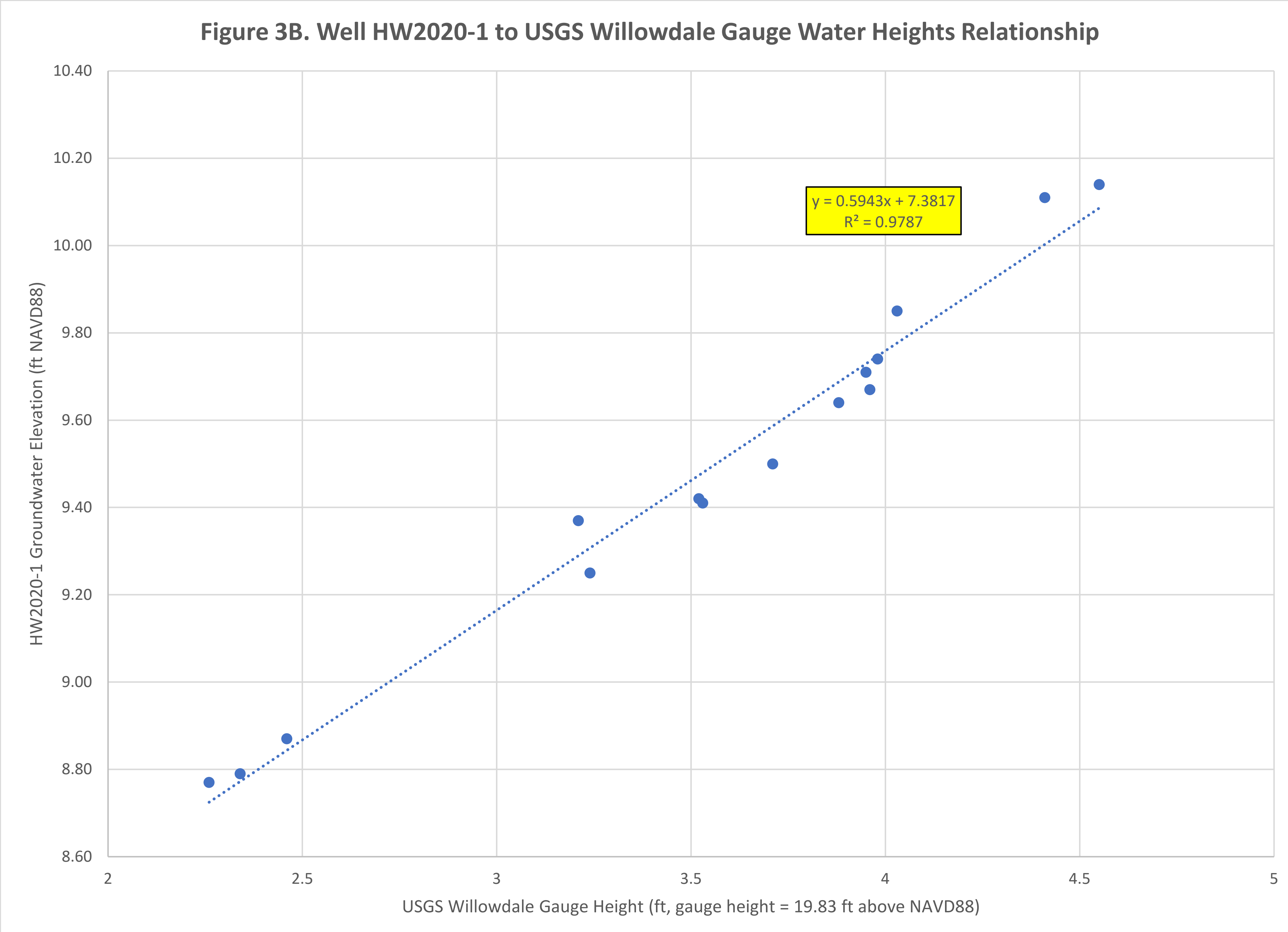


Figure 3B. Well HW2020-1 to USGS Willowdale Gauge Water Heights Relationship
y = 0.5943x + 7.3817
R² = 0.9787
HW2020-1 Groundwater Elevation (ft NAVD88)
USGS Willowdale Gauge Height (ft, gauge height = 19.83 ft above NAVD88)
10.40
10.20
10.00
9.80
9.60
9.40
9.20
9.00
8.80
8.60
2
2.5
3
3.5
4
4.5
5

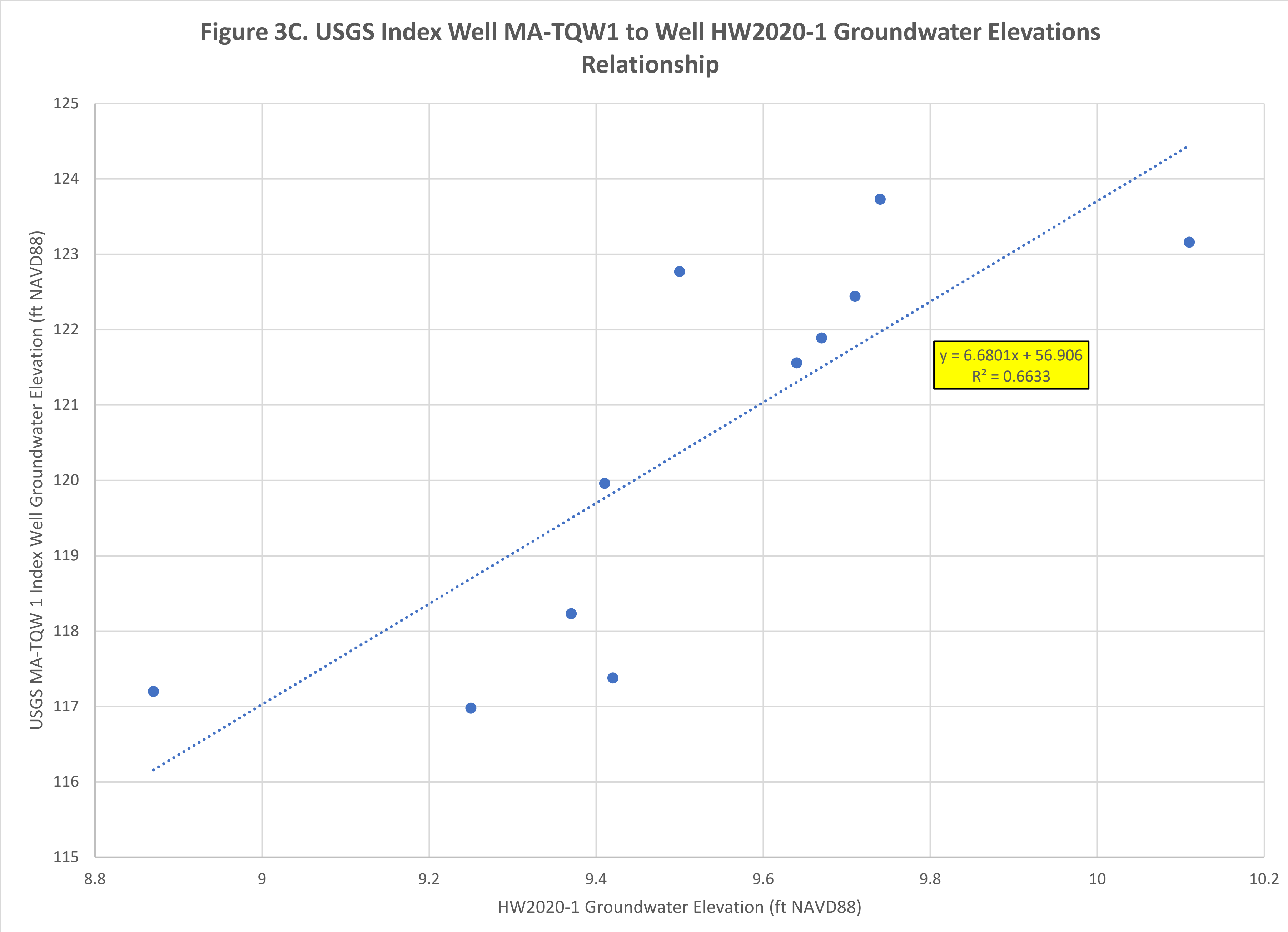


Figure 3C. USGS Index Well MA-TQW1 to Well HW2020-1 Groundwater Elevations Relationship
USGS MA-TQW 1 Index Well Groundwater Elevation (ft NAVD88)
125
124
123
122
121
120
119
118
117
116
115
8.8
9
9.2
9.4
9.6
9.8
10
10.2
HW2020-1 Groundwater Elevation (ft NAVD88)
y = 6.6801x + 56.906
R² = 0.6633

Figure 4. Ipswich River Stage (feet) at USGS Willowdale Gauge October 2007 – June 2021

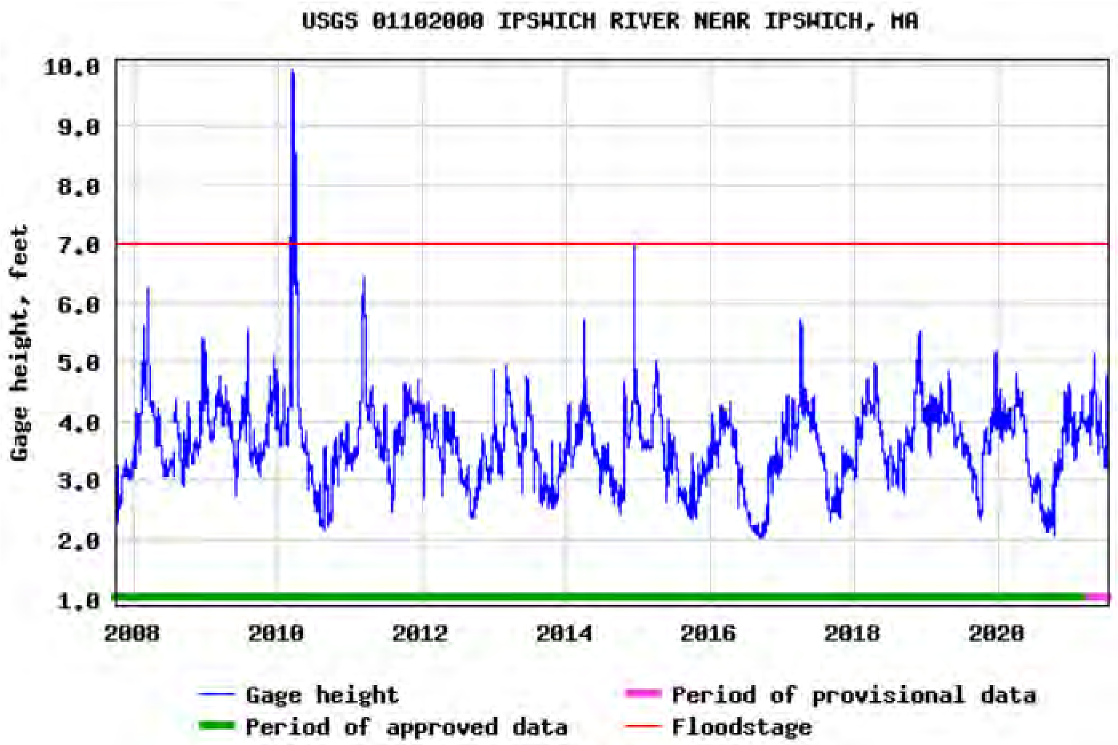


Table 3. Historic Water Level Statistics USGS Willowdale Gauge & Estimated for Ipswich Mills Dam and Well HW-2020-1

	USGS Willowdale Gauge Stage (ft)	Est. Ipswich Mills Dam River El. (ft NAVD88)	Est. Well HW-2020-1 El. (ft NAVD88)
Representative High-Water Value	4.5	10.0	10.1
Average Water Level Value	3.5	9.4	9.5
Representative Low-Water Value	2.5	8.9	8.9

A graph depicting all of the Ipswich Mills Dam river water level and HW-2020-1 groundwater level data from September 1, 2020 through June 23, 2021 is included here as Figure 5. Two groundwater level contour maps for the area around the EBSCO facility are presented here as Figures 6 and 7. Figure 6 depicts relatively low water level conditions from October 6, 2020, and Figure 7 depicts relatively high-water level conditions from April 21, 2021.

The following are the key hydrologic observations from the September 1, 2020 to June 23, 2021 water level monitoring, graphing, and assessment:

- The range of observed river elevations immediately above the Ipswich Mills Dam during the September 2020 through June 2021 monitoring period of this study was approximately 8.9 to 10.1 feet, similar to the estimated representative historical range shown in Table 3.
- The range of observed groundwater elevations at well HW-2020-1 (closest well to the southeast corner of the EBSCO building where compressible soils were encountered) during the September 2020 through June 2021 monitoring period of this study was approximately 8.6 to 10.5 feet, similar to though slightly greater than the estimated representative historical range shown in Table 3..
- Key correlated river elevation statistics immediately above the Ipswich Mills Dam (Table 3), estimated by correlation with the USGS Willowdale Gauge, are an approximate representative low water elevation of 8.9 feet, average of 9.4 feet, and high of 10.0 feet.
- Key correlated groundwater elevation statistics for well HW-2020-1, estimated by correlation with the USGS Willowdale Gauge are an approximate representative low water elevation of 8.9 feet, average of 9.5 feet, and high of 10.1 feet (Table 3).
- Water levels for the river and well HW-2020-1 are generally similar. Sometimes there is a slight gradient from groundwater towards the river, and sometimes the opposite, with no clear prevailing pattern evident. This phenomenon can frequently happen in close proximity to dams and impoundments where the more commonly observed in New England hydraulic gradient from the surrounding groundwater towards the river can sometimes be reversed when rising water levels behind the dam exceed the adjacent groundwater levels.
- Groundwater elevations in HW-2020-1 respond quickly to the many significant precipitation events experienced during the monitoring period (Figure 4). One possibility for this strong response is that, according to IRWA, a significant amount of roof, loading dock, and parking lot area runoff from the EBSCO Facility is conveyed to the unpaved area where HW-2020-1 is located.
- The two groundwater contour maps for relatively low and relatively high-water level conditions (Figures 5 and 6, respectively) show generally similar patterns of groundwater flow with only the values of each contour changing based on relatively high versus low water level conditions. The pattern of groundwater flow is generally south to north, parallel to the river along the EBSCO facility, instead of trending perpendicular to the river as is more typically seen in New England. The low water point on both contour maps is at well HW-2020-4 at the north end of the EBSCO Facility. This pattern of groundwater flow parallel to and around the impoundment is common in close proximity to dams and impoundments where the elevated river level behind a dam can direct

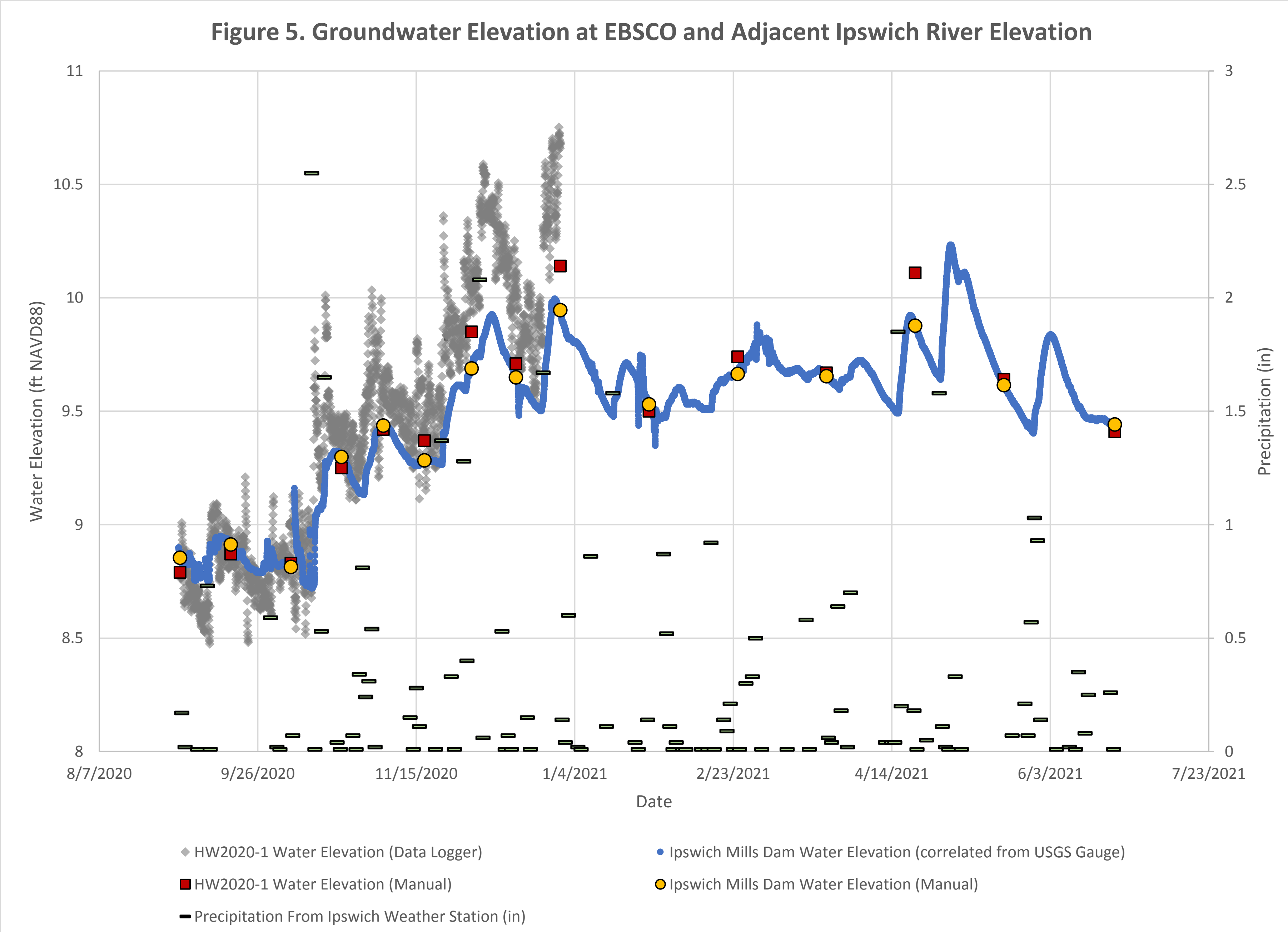
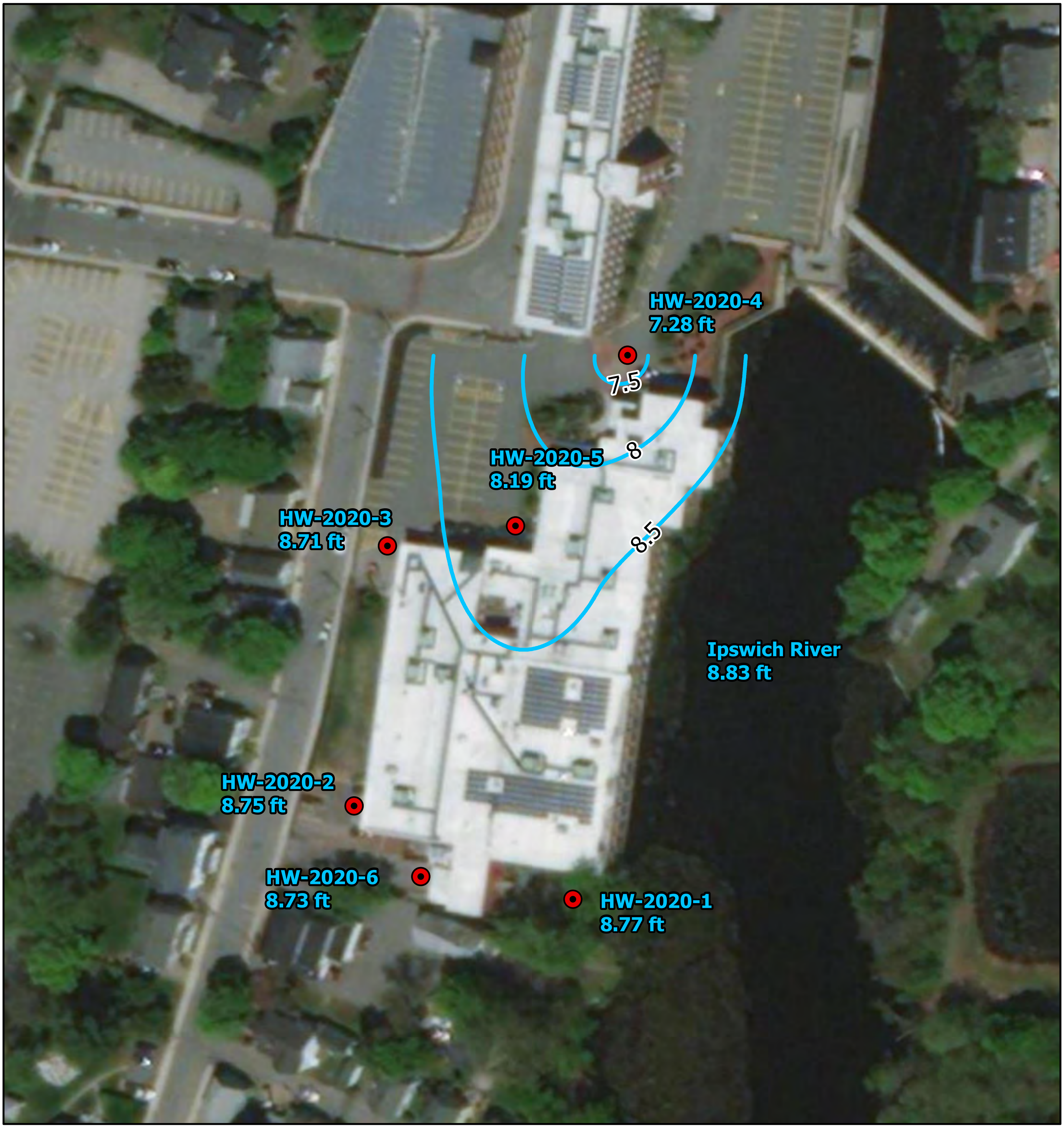


Figure 5. Groundwater Elevation at EBSCO and Adjacent Ipswich River Elevation
Water Elevation (ft NAVD88)
Precipitation (in)
11
10.5
10
9.5
9
8.5
8
3
2.5
2
1.5
1
0.5
0
8/7/2020
9/26/2020
11/15/2020
1/4/2021
2/23/2021
4/14/2021
6/3/2021
7/23/2021
Date
HW2020-1 Water Elevation (Data Logger)
Ipswich Mills Dam Water Elevation (correlated from USGS Gauge)
HW2020-1 Water Elevation (Manual)
Ipswich Mills Dam Water Elevation (Manual)
Precipitation From Ipswich Weather Station (in)



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Legend

- Monitoring Wells
- Water Table Contours (ft NAVD88)

N

0 50 100 200
Feet

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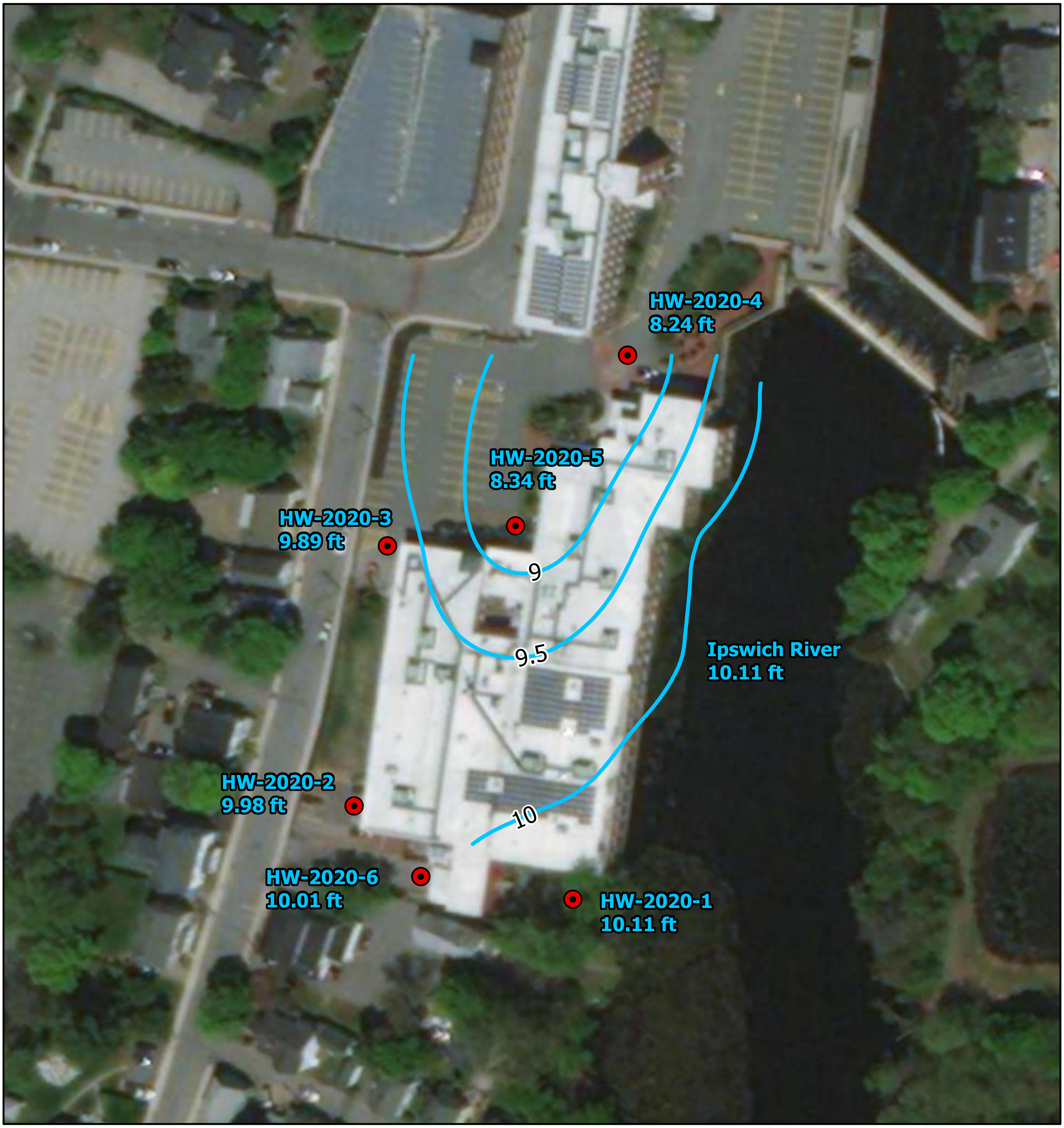
Water Table Contour Map
Low Water Conditions
October 6th, 2020
Ipswich River, MA

Date: 6/30/2021

Figure 6



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Legend

Monitoring Wells
Water Table Contours (ft NAVD88)

N

0 50 100 200
Feet

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Water Table Contour Map
High Water Conditions
April 21st, 2021
Ipswich River, MA

Date: 6/30/2021

Figure 7

groundwater flow to circumvent the dam and flow towards the lower elevation river downstream of the dam.

Sediment Characterization

With assistance from the Ipswich River Watershed Association (IRWA), HW conducted an assessment of sediment physical characteristics along the Ipswich River from the Choate Bridge upstream to the Railroad Bridge Crossing (Figures 8A and 8B). This sediment assessment included the collection of 15 samples for grain size analyses and probing to estimate the approximate thickness of soft sediments at 5 key transects (Figures 8A and 8B). Sediment samples were sent for grain size analysis to ESS Laboratories from Cranston, Rhode Island. Grain size laboratory analyses are summarized in Table 4 and included here at Attachment A. Sediment probing results are summarized in Table 5 and cross-sectional depictions of soft sediment thickness are included here at Attachment B.

Table 4. Summary of River Sediment Grain Size Analyses

Sample ID	Location	Latitude	Longitude	% Gravel	% Sand	% Fines
SS1	Choate Bridge	42.6794N	70.8369W	77.0	22.7	0.3
SS2	Retaining wall below dam	42.6792N	70.8375W	639	35.2	0.9
SS3	Retaining wall below dam	42.6789N	70.8375W	84.2	15.2	0.6
SS4	Retaining wall below dam	42.6782N	70.8376W	88.8	10.2	1.0
SS5	Fish Ladder outside	42.6777N	70.8377W	77.3	21.9	0.8
SS6	Pedestrian Bridge	42.6779N	70.8379W	67.9	31.2	0.9
SS7	Fish Ladder inside	42.6776N	70.8376W	20.1	40.7	39.2
SS8	EBSCO foundation	42.6768N	70.8385W	33.6	44.3	22.1
SS9	Saltonstall Brook	42.6743N	70.8384W	0.0	52.2	47.8
SS10	Kimball Brook	42.6741N	70.8393W	0.0	58.6	41.4
SS11	RR embankment N	42.6721N	70.8428W	45.7	16.8	37.5
SS12	Shady Brook	42.6682N	70.8444W	0.0	86.8	13.2
SS13	RR embankment S	42.6676N	70.8451W	68.2	31.1	0.7
SS14	Miles River	42.6638N	70.8462W	0.0	72.6	27.4
SS15	RR Bridge	42.6640N	70.8470W	94.7	4.9	0.4

Table 5. Summary of River Sediment Probing Results

Location	Assoc. Grain Size Sample #	Transect Length (ft)	Max Thickness Soft Sed. (ft)	Avg. Thickness Soft Sed (ft)
XS1 - Saltonstall Brook	SS9	18	6	2.6
XS2 – Kimball Brook	SS10	14	2.5	1.0
XS3 – Shady Brook	SS12	11	1	0.25
XS4 – Miles River	SS14	28	3.5	0.75
XS5 – Railroad Bridge Downstream	SS15	95	0	0

Geophysical Investigation

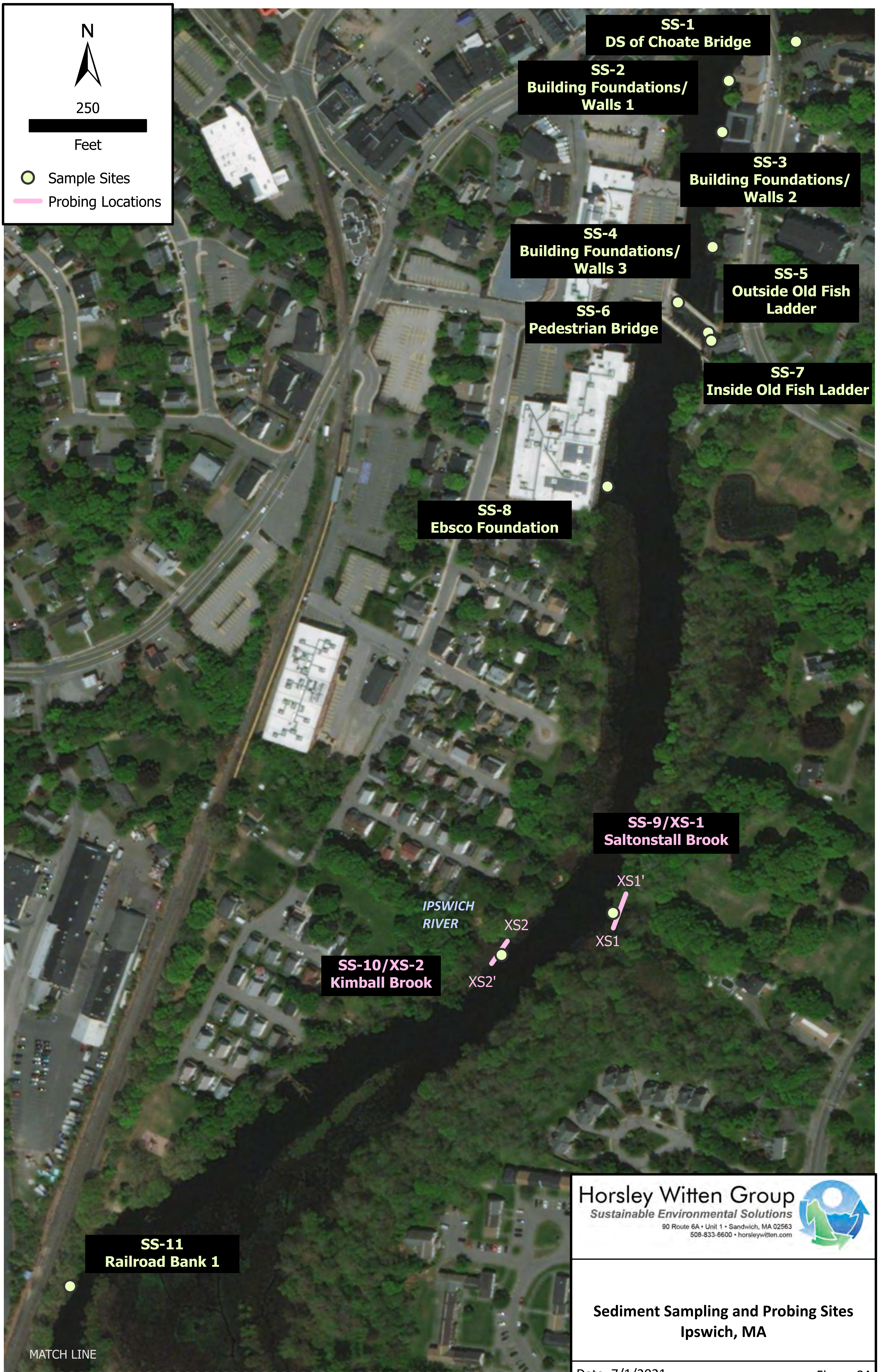
As a follow-up to previous Interior and exterior geophysical investigations conducted in 2020 by Hager Geoscience, Inc. (HGI) from Woburn, MA, additional interior EBSCO building geophysical investigations were completed from June 7 to 10, 2021 by Radar Systems International, Inc. (RSI) from Waltham, MA. RSI conducted its work as a sub-contractor to Simpson Gumpertz and Heger, Inc. (SGH), the structural engineering firm that has been working on the Ipswich Mills dam feasibility project since 2016 as a sub-contractor to HW. A full report of the RSI Investigation is included with the SGH report, included here as Attachment C.

The 2021 RSI geophysical investigation was undertaken to complement the 2020 HGI study by using different geophysical techniques and expanding the area of interior floor slab covered. RSI used two different techniques from those used by HGI in 2020: higher frequencies of ground penetrating radar (GPR) data acquisition, and Impact Echo sonic scanning of the concrete slab and underlying structures. Interior building work was spatially limited to the eastern / southeastern portion of the building with a focus on areas closest to the river wall where the likelihood for soft sediments and wooden support pilings is greatest. Within that general interior work area, work was further limited to open corridors and limited open spaces that had sufficient room for the geophysical equipment to be operated.

One original goal for this project was to conduct direct, physical, sub-surface corings through the EBSCO floor slab as a means to “ground truth” the 2020 and 2021 geophysical information. Unfortunately, EBSCO would not allow any intrusive sub-surface work, such as corings, at the time of this study.

Key findings from the RSI 2021 geophysical investigations are as follows:

- The foundation wall on the south elevation wall (including the southeast corner) is approximately 24 to 28 inches thick at the top (RSI Report Attachment C - Appendix C Fig.8) and between 17 and 21 feet deep (RSI Report Appendix C Fig.9). Based on these results, SGH estimates that the bottom of wall is located between approximately elevation -4.5 feet and -8.5 feet.



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Sediment Sampling and Probing Sites
Ipswich, MA

Date: 7/1/2021

Figure 8A

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Document Path: H:\Projects\2016\16041 DER_Ipswich Dam Removal Feasibility Study\16041E Hydrologic and Structural Assessment\GIS\Sediment Sample Sites\Sediment Sample Sites.aprx

- The impact echo results around column “B5” (RSI Report Attachment C - Appendix C Fig. 2) show that the slab-on-grade is nominally approximately 5.5 to 6 inches thick. The high frequency GPR shows that the slab is slightly thicker but is generally consistent with the impact echo results. RSI was unable to detect the slab thickness in several locations due to interference of the impact echo signal, likely from deteriorated concrete or concrete patch material, and buried conduit. RSI also interprets the presence of a 6-inch-thick layer of granular material underlying the slab-on-grade.

- The impact echo results around column “B5” indicate thicker concrete at column lines B and 5, approximately 28 to 35 inches thick, which is consistent with a thickened slab or grade beam foundations aligning with the column lines. RSI indicates that the width of the thickened concrete or grade beams is the same or slightly less than the 21 X 21-inch column pedestal visible above the slab. The column pedestal is approximately 24 to 34 inches deep, similar in width and depth to the grade beams. It is possible that the grade beams and pedestals are integral to each other.

- RSI were able to detect pile or support pier foundations located below the column pedestal. The depth to the bottom of the pile(s) or support pier is 10.7 to 10.8 feet from the top of the pedestal (RSI Report Attachment C - Appendix C Fig. 5). RSI could not determine the composition of the piles or support piers (i.e., wood versus concrete). RSI reports that the grade beams are 24 to 39 inches thick. Based on these results, SGH estimates that the pile or support pier top elevation(s) are located at approximately 8.8 to 9.7 feet and the pile or support pier bottom elevation(s) are located at approximately elevation 0.9 feet.

- RSI prepared depth slice images which are plan views of their results plotted at various depths for both the high frequency GPR survey (ranging from approximately 1 to 17 inches below the top of the slab) (RSI Report Attachment C - Figs. 4A to 4M), and the low frequency GPR survey (ranging from approximately 0.3 to 5.5 feet below the top of the slab) (RSI Report Appendix C Figs. 7A to 7M). RSI performed a visual inspection of the GPR data and summarized the results in RSI Report Appendix C Fig. 6. The GPR results do not indicate the presence of piles or support piers below the slab except for underlying the grade beams. RSI states that the GPR results indicate horizontally oriented targets at depths of 15 to 24 inches below the slab and interprets these targets to potentially be buried conduits. RSI was able to detect several locations with reflections that could indicate voiding or lower density soils, higher density materials such as boulders or structures, or increased moisture.

EBSCO Structural Assessment

SGH updated its prior assessment of potential structural implications for the EBSCO facility from potential dam removal based on the 2020 HGI geophysical investigation, the follow-up 2021 RSI geophysical investigation, and the 2020/2021 water level monitoring program. The SGH report discusses the overall structural implications for the EBSCO facility from potential dam removal, as well as potential options for the mitigation of undesirable impacts. The SGH report is included herein as Attachment C. The key findings from the SGH report are as follows:

- <u>Existing Foundations:</u>

  - It is very likely that the exterior walls of the EBSCO Facility (Building Nos. 9, Building No. 10, Building No. 10A, Building No. 10B, Building No. 11, and Building No. 11B) are founded on spread footings bearing on competent soils such as the Glacial Till stratum, Clayey Silt stratum, or rock.

  - Is likely that the interior columns of Building No. 9, Building No. 10B, and the northern portion of Building No. 11 are founded on shallow spread footings bearing on the Glacial Till stratum or rock.

  - The interior columns in Buildings No. 10 and No. 10A are likely supported on concrete grade beams underlain by piles or support piers of unknown material type (i.e., wood versus concrete) bearing on the glacial till stratum. The thickness of concrete elements measured in Building 10A suggests either a locally thickened slab or a pile supported foundation element such as a grade beam supporting the interior columns. The depth and thickness of the suspected compressible soils in this area of the building are such that the support beneath the grade beams could be either concrete support piers installed by locally excavating through the compressible soils to bear on the Glacial Till stratum below or timber piles driven through the soft compressible soils.

  - Building off of the 2021 geophysical study, as well as all prior studies, SGH estimates that the interior columns within Buildings 10 and 10A are likely supported on pile foundations of uncertain material makeup (e.g., concrete or wood), and that those pile tops have frequently been exposed above the water table over the building's lifespan. With the top of support piers and/or piles at approximate elevations from 8.8 to 9.7 feet, and prevailing groundwater elevations in 2020 and 2021 ranging between approximate elevations from 8.75 to 10.5 feet (Hydrologic Evaluation section above), the piles or support piers have been exposed by between 0 and 1 feet between September 2020 to late June 2021. As discussed above, the 2020 to 2021 monitoring period exhibited a similar range of overall water levels to those estimated to be representative or typical of longer-range conditions, not including less frequent, exceptionally low or high-water levels. During periods of lower river levels (e.g., the approximately 20-year interval from the 1960s into the 1980s when IRWA reports that the dam gates were either leaking badly or left open during repair periods, or significant droughts), both the vertical length and time duration of support pile exposure would be greater.

    Assuming no bedrock control of river elevation at the dam (bedrock control currently uncertain based on 2020 HGI geophysical study) and a resultant lowest possible future river elevation of approximately 2 feet (approximate riverbed elevation downstream of dam), those support piles could be exposed by up to approximately 7.7 feet under a dam-out scenario. This is a conservative, maximum exposure estimate based on the assumption of river levels falling to the grade of the existing river bed downstream of the dam (an essentially dry river), and the groundwater levels beneath the EBSCO mimicking that same river level decline. In reality, even if there is no bedrock river grade control at the dam, groundwater levels would tend to be more reflective of average tidal water level conditions in the river, not low tide, drought, dry riverbed conditions.

- o The floor of buildings 10 and 10A is slab on grade construction. This means that it supports only the floor itself and any machinery, furniture, etc. sitting on the floor, but not the buildings themself. As described in the above bullets, the buildings are likely supported by the exterior foundation walls and the interior grid of support piers or piles, grade beams, and columns.

- o At this time, it is uncertain to what extent, if any, compressible soils underlie the EBSCO Facility. SGH did not encounter soft compressible soils in soil test borings located on the west and north sides of the building away from the river but did encounter a small area of compressible soils outside of the southeastern corner of the building. GPR surveys performed by HGI in 2020 inside the EBSCO Facility indicate depths to the top of the glacial till stratum consistent with prior soil test borings.

- o SGH observed no signs of settlement of the EBSCO facility interior structural framing, with the exception of minor cracking near the southeast corner. It is possible that the lack of signs of significant settlement of the columns, if they are supported on timber piles, is due to the tops of the timber piles remaining saturated at the low bound of the groundwater levels. The presence of organic soils around the tops of the timber piles may help maintain soil conditions sufficiently saturated such as to not allow for significant deterioration within the durations of the periods of low groundwater levels. Note that the IRWA description of an approximately 20 years of low water behind the dam during the 1960s through 1980s does not support that potential situation. The lack of column settlement could also indicate that the piles (or piers) are concrete and, therefore, not susceptible to drying and fungal attack.

- o SGH observed sloped areas of the slab inside the EBSCO Facility, which suggests that some settlement of the slab has taken place in the past (assuming it was level when installed). Settlement of the slab is most apparent at the southeast corner of the EBSCO Facility, along column line B, between Column Lines 1 and 10. Surface patching material was observed on the slab below the carpet finishes at the one location where finishes were removed around column "B5." The location of this settlement correlates with the soft soils encountered in soil test borings drilled on the exterior and the 2020 HGI GPR results showing that the depth to the Glacial Till stratum is greatest in this area (up to 16 ft below the slab). The observed settlement also correlates well with RSI's findings that there are no piles supporting the slab.

- <u>Effects of Lowering Groundwater:</u> Lowered groundwater levels could result in settlement of pavement, slabs-on-grade, and structures on spread footings or buried utilities supported by soft compressible soils. The accumulation of evidence from recent geophysical studies, exterior borings, and interior visual observations suggests that the building structure itself is not supported by potentially compressible soils. SGH previously estimated a potential total settlement of the soft compressible soils of approximately between 0.9 to 1.5 inches. respectively due to a water level drawdown of between 1 and 5 feet, assuming a remaining service life of 50 years for the EBSCO facility, in those areas where compressible soils are present (SGH Report Attachment C - Appendix A).

- Next Steps: The accumulation of information from recent studies in and around the EBSCO Facility indicates that, if timber piles are present, they are likely limited to areas supporting the grid of grade beam structural supports beneath Buildings 10 and 10A. Geophysical evidence indicates that structural support piers or pilings likely do exist beneath these grade beams, but the material makeup of the support piers could not be determined. Therefore, the primary next step would be to conduct targeted interior corings and test pits to directly identify the material type and elevations of support piers. The Geophysical evidence indicates at least one known support pier location where such subsurface investigations could be targeted. If timber support piles are found to exist beneath the grade beam grid of building foundational elements, and if the project team anticipates that the post-dam removal groundwater levels cannot be maintained at or above the elevation of the top of those potential timber piles, the project team should reserve funds for settlement mitigation repairs. If the project team is not provided access to determine if timber piles and compressible soils are present within the EBSCO Facility, the following approach could be implemented to mitigate potential settlement of the slab-on-grade and interior columns of the EBSCO Facility, assuming an unknown but worst-case scenario that susceptible timber piles are present:

    - Develop and implement a precision movement monitoring program to monitor for the potential movement of structures during dam removal construction. Install the instrumentation prior to the start of construction, and also establish acceptable settlement limits with approval from EBSCO.

    - Reserve funds for settlement mitigation repairs to interior columns and slabs located within Buildings 10 and 10A. Additional subsurface investigation inside the building would be required to develop detailed repair design and confirm the presence of timber pile foundations.

    The SGH report provides more detail and planning level cost estimates for these potential mitigation items.

Conclusions

The 2021 RSI Geophysical Investigation indicates that the thickness of the column support pedestals and grade beam is between 2 to 2.9 feet, based on RSI's impact echo measurements. These results indicate that the likely support piers or pile tops (cutoffs) are immediately below the grade beams at approximately 8.8 to 9.7 feet elevation. For comparison, the dam spillway elevation is approximately 8.8 feet; and from September 2020 through June 2021 the range of river and groundwater levels at the southeast corner of the EBSCO facility were from 8.8 to 10.2 feet, and 8.5 to 10.5 feet, respectively. Approximate, representative, calculated (by correlation with the USGS Willowdale Gauge), longer-term water elevations are essentially similar.

The exterior foundation walls extend to a depth of up to 21 feet below the top of the wall and are likely bearing on glacial till or rock. RSI was able to detect several locations with reflections that could indicate voiding or lower density soils, higher density materials such as boulders or structures, or increased moisture; however it was unable to identify timber piles beneath the slab, except for potentially beneath concrete grade beam structural elements, Building off of the 2021 geophysical study, as well as all prior studies, SGH estimates that the interior columns within Buildings 10 and 10A are likely supported on concrete grade beams with underlying pile or support piers foundations of unknown material makeup (i.e. wood versus concrete). The concrete floor slab between grade beam foundation elements is unlikely to be supported by timber piles.

The accumulation of information from this study and other recent studies in and around the EBSCO Facility has improved our understanding of how the buildings are structurally supported and the potential implications of lowered river levels on the buildings' structural stability, as follows:

- With the exception of Buildings 10 and 10A, the majority of the EBSCO Facility appears to be most likely supported by exterior foundation walls and interior columns founded on shallow spread footings bearing on the Glacial Till stratum or rock. These structural conditions are unlikely to be significantly negatively impacted by lowered river levels.
- The exterior foundation walls of Buildings 10 and 10A also appear most likely to be founded on shallow spread footings bearing on the Glacial Till stratum or rock. The Interior columns of Buildings 10 and 10A appear to be supported by a grid of grade beams which themselves are supported by piers or pilings of either wood or concrete construction.
- Whether the support piers or piles beneath the Buildings 10 and 10A grade beams grid are of concrete or wooden construction has not yet been determined. However, if the support piers or piles are wooden, the potential extent of potential timber pile presence appears likely to be limited to areas supporting that grid of grade beam structural supports. Timber piles, if present, would potentially be negatively impacted by lowered river levels.
- If the grade beam support piles are wooden, then their elevations are such that the pile tops have already been exposed by approximately one foot under the current river conditions, potentially by more during past lower water level conditions, and could be exposed by up to 7.7 feet in a conservative worst-case post-dam scenario that assumes river levels falling to the grade of the existing river bed downstream of the dam (an essentially dry river). In reality, groundwater levels would tend to be more reflective of average tidal water level conditions in the river, not low tide, drought, dry riverbed conditions.
- Lowered river levels could potentially lead to settlement of the floor slab for Buildings 10 and 10A by compaction of compressible soils. Such compaction would negatively impact the floor and any equipment or furniture on that floor, but would be unlikely to significantly impact the overall structure of the buildings themselves.

The following options should be considered to better understand the potential implications of lowered river water levels on the EBSCO Facility, and to mitigate against potential negative impacts. The primary and highest priority recommended activity is to conduct interior test pits at targeted locations within the Facility to definitively answer the remaining questions about whether or not timber piles exist, and the corresponding potential structural susceptibility of the Facility to lowered water levels. Approximate planning-level costs for these mitigation alternatives are presented in the SGH report (Attachment C). Other alternatives not discussed below may also exist.

- Intrusive Interior Studies: As recommended in the SGH 2018 report, test pits, borings, and/or concrete and timber piling coring and competency testing conducted through the slab at key locations in the building interior would allow for direct physical and/or visual confirmation or refutation of the geophysical findings with regards to soft sediment and timber piling characteristics. Such direct physical examination would allow the current condition of the pilings to be better understood, as well as the potential for additional risk, if any, that might be posed by lowered river water levels. Examples of such direct piling examinations include Pile Integrity Testing (PIT), which is most accurately performed by excavating through the concrete slab to the tops of the pilings, and parallel seismic testing, which involves generating a seismic pulse directly above the pilings (through the slab) and then recording the seismic responses in a parallel borehole drilled through the slab and the underlaying geologic materials.
- Pre and Post-Dam Removal Precision Movement Monitoring: As discussed in the SGH 2018 report, existing conditions building assessment and precision monitoring of building movement is one technique to assess potential settlement. However, due to accuracy of the measurement devices and time lags for settling, such monitoring may not be able to identify settlement issues in time to address them before damage occurs. In addition, given the apparent elevations of the tops of support piers or piles from the geophysical study, and if the piers/piles are of wooden construction, piling deterioration and settlement may already be occurring under existing conditions. Determining to what extent, if any, future building settlement may be a result of lowered water levels as opposed to ongoing existing conditions may be challenging. Should this alternative be advanced, a visual inspection in combination with the monitoring program to document existing conditions structural evaluation of the EBSCO building is recommended.
- Perform cut and post underpinning repairs of potential timber piles (if present): As discussed in the SGH 2018 report, this is a technique to remove the tops of deteriorating timber piles (if present) and replace them with concrete and steel extensions. This technique could be conducted as a repair for existing conditions deterioration (if present) and/or a mitigation to protect against lowered water levels (if applicable). This option would be significantly disruptive and expensive due to the size of the building and number of pilings observed beneath the building. This method is generally more suitable for situations where the total lengths of the potential timber piles to be repaired (based on exposure above anticipated future groundwater levels) are short relative to the total lengths of the piles (i.e., depth to supportive till material). As the ratio of repair length to total length increases, one essentially is replacing the piles and, at that point compaction grouting (described below) generally becomes a more suitable option.

- Perform compaction grouting ground improvement to provide remedial support of potential timber pile supported structures: This method involves installing low-slump grout in lifts below grade to displace the soil and create columns of grout. More specifically, compaction grouting is performed by inserting 2-inch diameter grout injection pipes through the target weak soil stratum, then pumping low-slump grout under pressure, which forms a bulb of grout and pushes the surrounding soil, thus densifying the soil. After achieving a target pressure or volume of grout, the grout injection pipe is raised to a higher elevation, and another bulb of grout is injected. The process is repeated, extending the grout vertically through the entire weak soil stratum. The spacing of the compaction grout locations is designed by an engineer and typically depends on the subsurface conditions, the use of the building space (loads), and the capacity of the existing or new slab. One advantage to this method is that no excavation is required. However, a specialty grouting contractor is required, and both compaction grouting and cut and post underpinning are expensive and disruptive to building operations.

Maintaining water levels high enough to cover the potential timber pilings in a potential dam removal scenario (either through manipulation of the riverbed elevation or through the installation of a groundwater recharge system around the eastern and southeastern portions of the building) has been discussed in prior SGH reports. However, given that the recent RSI geophysical study indicates that the tops of support piers or piles (potentially wooden though uncertain) are located up to approximately elevation 9.7 feet, and that the recent HGI geophysical study calls into questions the presence of a high bedrock ridge at the dam location that might control water elevations in a dam-out scenario, such a water level maintenance program would likely be both expensive and difficult to implement successfully. Further, if timber piles are present, and with tops located at this elevation, the pile tops have already been subject to aerobic deterioration due to their top elevations likely extending seasonally above existing groundwater levels, and for longer past periods of time (as reported by IRWA) when the dam has been open to allow lower water levels than are currently typical. Given these findings, it is unclear at this time if maintaining groundwater or river levels at the current dam crest elevation would be successful (or necessary) toward protecting the future structural integrity of the EBSCO building.

Attachments

Attachment A – 2021 Sediment Grain Size Laboratory Report

Attachment B – 2021 Soft Sediment Thickness Cross-Sections

Attachment C – 2021 SGH EBSCO Structural Assessment Report with 2021 RSI Geophysical Investigation included

ATTACHMENT A –
2021 Sediment Grain Size Laboratory Report

CERTIFICATE OF ANALYSIS

Neal Price
Horsley & Witten
90 Route 6A
Sandwich, MA 02563

RE: Ipswich Mills Dam (16041E)
ESS Laboratory Work Order Number: 21F0999

This signed Certificate of Analysis is our approved release of your analytical results. These results are only representative of sample aliquots received at the laboratory. ESS Laboratory expects its clients to follow all regulatory sampling guidelines. Beginning with this page, the entire report has been paginated. This report should not be copied except in full without the approval of the laboratory. Samples will be disposed of thirty days after the final report has been delivered. If you have any questions or concerns, please feel free to call our Customer Service Department.

REVIEWED
By ESS Laboratory at 2:45 pm, Jul 02, 2021

Laurel Stoddard
Laboratory Director

Analytical Summary
The project as described above has been analyzed in accordance with the ESS Quality Assurance Plan. This plan utilizes the following methodologies: US EPA SW-846, US EPA Methods for Chemical Analysis of Water and Wastes per 40 CFR Part 136, APHA Standard Methods for the Examination of Water and Wastewater, American Society for Testing and Materials (ASTM), and other recognized methodologies. The analyses with these noted observations are in conformance to the Quality Assurance Plan. In chromatographic analysis, manual integration is frequently used instead of automated integration because it produces more accurate results.

The test results present in this report are in compliance with TNI and relative state standards, and/or client Quality Assurance Project Plans (QAPP). The laboratory has reviewed the following: Sample Preservations, Hold Times, Initial Calibrations, Continuing Calibrations, Method Blanks, Blank Spikes, Blank Spike Duplicates, Duplicates, Matrix Spikes, Matrix Spike Duplicates, Surrogates and Internal Standards. Any results which were found to be outside of the recommended ranges stated in our SOPs will be noted in the Project Narrative.

Subcontracted Analvses

CTS - Cranston, RI	Grain Size Analysis

CERTIFICATE OF ANALYSIS

Client Name: Horsley & Witten
Client Project ID: Ipswich Mills Dam
ESS Laboratory Work Order: 21F0999

SAMPLE RECEIPT

The following samples were received on June 25, 2021 for the analyses specified on the enclosed Chain of Custody Record.

Lab Number	Sample Name	Matrix	Analysis
21F0999-01	SS1	Soil	SUB
21F0999-02	SS2	Soil	SUB
21F0999-03	SS3	Soil	SUB
21F0999-04	SS4	Soil	SUB
21F0999-05	SS5	Soil	SUB
21F0999-06	SS6	Soil	SUB
21F0999-07	SS7	Soil	SUB
21F0999-08	SS8	Soil	SUB
21F0999-09	SS9	Soil	SUB
21F0999-10	SS10	Soil	SUB
21F0999-11	SS11	Soil	SUB
21F0999-12	SS12	Soil	SUB
21F0999-13	SS13	Soil	SUB
21F0999-14	SS14	Soil	SUB
21F0999-15	SS15	Soil	SUB

CERTIFICATE OF ANALYSIS

Client Name: Horsley & Witten
Client Project ID: Ipswich Mills Dam
ESS Laboratory Work Order: 21F0999

PROJECT NARRATIVE

No unusual observations noted.

End of Project Narrative.

DATA USABILITY LINKS

To ensure you are viewing the most current version of the documents below, please clear your internet cookies for www.ESSLaboratory.com. Consult your IT Support personnel for information on how to clear your internet cookies.

Definitions of Quality Control Parameters

Semivolatile Organics Internal Standard Information

Semivolatile Organics Surrogate Information

Volatile Organics Internal Standard Information

Volatile Organics Surrogate Information

EPH and VPH Alkane Lists

CERTIFICATE OF ANALYSIS

Client Name: Horsley & Witten
Client Project ID: Ipswich Mills Dam
ESS Laboratory Work Order: 21F0999

CURRENT SW-846 METHODOLOGY VERSIONS

Analytical Methods

1010A - Flashpoint
6010C - ICP
6020A - ICP MS
7010 - Graphite Furnace
7196A - Hexavalent Chromium
7470A - Aqueous Mercury
7471B - Solid Mercury
8011 - EDB/DBCP/TCP
8015C - GRO/DRO
8081B - Pesticides
8082A - PCB
8100M - TPH
8151A - Herbicides
8260B - VOA
8270D - SVOA
8270D SIM - SVOA Low Level
9014 - Cyanide
9038 - Sulfate
9040C - Aqueous pH
9045D - Solid pH (Corrosivity)
9050A - Specific Conductance
9056A - Anions (IC)
9060A - TOC
9095B - Paint Filter
MADEP 04-1.1 - EPH
MADEP 18-2.1 - VPH

Prep Methods

3005A - Aqueous ICP Digestion
3020A - Aqueous Graphite Furnace / ICP MS Digestion
3050B - Solid ICP / Graphite Furnace / ICP MS Digestion
3060A - Solid Hexavalent Chromium Digestion
3510C - Separatory Funnel Extraction
3520C - Liquid / Liquid Extraction
3540C - Manual Soxhlet Extraction
3541 - Automated Soxhlet Extraction
3546 - Microwave Extraction
3580A - Waste Dilution
5030B - Aqueous Purge and Trap
5030C - Aqueous Purge and Trap
5035A - Solid Purge and Trap

SW846 Reactivity Methods 7.3.3.2 (Reactive Cyanide) and 7.3.4.1 (Reactive Sulfide) have been withdrawn by EPA. These methods are reported per client request and are not NELAP accredited.

CERTIFICATE OF ANALYSIS

Client Name: Horsley & Witten
Client Project ID: Ipswich Mills Dam
ESS Laboratory Work Order: 21F0999

Subcontracted Analysis

Client Sample ID: SS1
Date Sampled: 06/23/21 10:02
ESS Laboratory Sample ID: 21F0999-01
Sample Matrix: Soil

Analyte	Results	Units	MRL	Method	DF	Analyst	Analyzed	I/V	F/V
Grain Size	**See Attached**								

Client Sample ID: SS2
Date Sampled: 06/23/21 10:13
ESS Laboratory Sample ID: 21F0999-02
Sample Matrix: Soil

Analyte	Results	Units	MRL	Method	DF	Analyst	Analyzed	I/V	F/V
Grain Size	**See Attached**								

Client Sample ID: SS3
Date Sampled: 06/23/21 10:26
ESS Laboratory Sample ID: 21F0999-03
Sample Matrix: Soil

Analyte	Results	Units	MRL	Method	DF	Analyst	Analyzed	I/V	F/V
Grain Size	**See Attached**								

Client Sample ID: SS4
Date Sampled: 06/23/21 10:49
ESS Laboratory Sample ID: 21F0999-04
Sample Matrix: Soil

Analyte	Results	Units	MRL	Method	DF	Analyst	Analyzed	I/V	F/V
Grain Size	**See Attached**								

CERTIFICATE OF ANALYSIS

Client Name: Horsley & Witten
Client Project ID: Ipswich Mills Dam

ESS Laboratory Work Order: 21F0999

Subcontracted Analysis

Client Sample ID: SS5
Date Sampled: 06/23/21 11:04

ESS Laboratory Sample ID: 21F0999-05
Sample Matrix: Soil

Analyte	Results	Units	MRL	Method	DF	Analyst	Analyzed	I/V	F/V
Grain Size	**See Attached**								

Client Sample ID: SS6
Date Sampled: 06/23/21 11:20

ESS Laboratory Sample ID: 21F0999-06
Sample Matrix: Soil

Analyte	Results	Units	MRL	Method	DF	Analyst	Analyzed	I/V	F/V
Grain Size	**See Attached**								

Client Sample ID: SS7
Date Sampled: 06/23/21 12:45

ESS Laboratory Sample ID: 21F0999-07
Sample Matrix: Soil

Analyte	Results	Units	MRL	Method	DF	Analyst	Analyzed	I/V	F/V
Grain Size	**See Attached**								

Client Sample ID: SS8
Date Sampled: 06/23/21 13:00

ESS Laboratory Sample ID: 21F0999-08
Sample Matrix: Soil

Analyte	Results	Units	MRL	Method	DF	Analyst	Analyzed	I/V	F/V
Grain Size	**See Attached**								

CERTIFICATE OF ANALYSIS

Client Name: Horsley & Witten
Client Project ID: Ipswich Mills Dam
ESS Laboratory Work Order: 21F0999

Subcontracted Analysis

Client Sample ID: SS9
Date Sampled: 06/24/21 09:02
ESS Laboratory Sample ID: 21F0999-09
Sample Matrix: Soil

Analyte	Results	Units	MRL	Method	DF	Analyst	Analyzed	I/V	F/V
Grain Size	**See Attached**								

Client Sample ID: SS10
Date Sampled: 06/24/21 09:12
ESS Laboratory Sample ID: 21F0999-10
Sample Matrix: Soil

Analyte	Results	Units	MRL	Method	DF	Analyst	Analyzed	I/V	F/V
Grain Size	**See Attached**								

Client Sample ID: SS11
Date Sampled: 06/24/21 09:25
ESS Laboratory Sample ID: 21F0999-11
Sample Matrix: Soil

Analyte	Results	Units	MRL	Method	DF	Analyst	Analyzed	I/V	F/V
Grain Size	**See Attached**								

Client Sample ID: SS12
Date Sampled: 06/24/21 09:35
ESS Laboratory Sample ID: 21F0999-12
Sample Matrix: Soil

Analyte	Results	Units	MRL	Method	DF	Analyst	Analyzed	I/V	F/V
Grain Size	**See Attached**								

CERTIFICATE OF ANALYSIS

Client Name: Horsley & Witten
Client Project ID: Ipswich Mills Dam
ESS Laboratory Work Order: 21F0999

Subcontracted Analysis

Client Sample ID: SS13
Date Sampled: 06/24/21 09:45
ESS Laboratory Sample ID: 21F0999-13
Sample Matrix: Soil

Analyte	Results	Units	MRL	Method	DF	Analyst	Analyzed	I/V	F/V
Grain Size	**See Attached**								

Client Sample ID: SS14
Date Sampled: 06/24/21 10:00
ESS Laboratory Sample ID: 21F0999-14
Sample Matrix: Soil

Analyte	Results	Units	MRL	Method	DF	Analyst	Analyzed	I/V	F/V
Grain Size	**See Attached**								

Client Sample ID: SS15
Date Sampled: 06/24/21 10:10
ESS Laboratory Sample ID: 21F0999-15
Sample Matrix: Soil

Analyte	Results	Units	MRL	Method	DF	Analyst	Analyzed	I/V	F/V
Grain Size	**See Attached**								

CERTIFICATE OF ANALYSIS

Client Name: Horsley & Witten
Client Project ID: Ipswich Mills Dam
ESS Laboratory Work Order: 21F0999

Notes and Definitions

Z-08	See Attached
ND	Analyte NOT DETECTED at or above the MRL (LOQ), LOD for DoD Reports, MDL for J-Flagged Analytes
dry	Sample results reported on a dry weight basis
RPD	Relative Percent Difference
MDL	Method Detection Limit
MRL	Method Reporting Limit
LOD	Limit of Detection
LOQ	Limit of Quantitation
DL	Detection Limit
I/V	Initial Volume
F/V	Final Volume
§	Subcontracted analysis; see attached report
1	Range result excludes concentrations of surrogates and/or internal standards eluting in that range.
2	Range result excludes concentrations of target analytes eluting in that range.
3	Range result excludes the concentration of the C9-C10 aromatic range.
Avg	Results reported as a mathematical average.
NR	No Recovery
[CALC]	Calculated Analyte
SUB	Subcontracted analysis; see attached report
RL	Reporting Limit
EDL	Estimated Detection Limit
MF	Membrane Filtration
MPN	Most Probably Number
TNTC	Too numerous to Count
CFU	Colony Forming Units

CERTIFICATE OF ANALYSIS

Client Name: Horsley & Witten
Client Project ID: Ipswich Mills Dam   ESS Laboratory Work Order: 21F0999

ESS LABORATORY CERTIFICATIONS AND ACCREDITATIONS

ENVIRONMENTAL

Rhode Island Potable and Non Potable Water: LAI00179
http://www.health.ri.gov/find/labs/analytical/ESS.pdf

Connecticut Potable and Non Potable Water, Solid and Hazardous Waste: PH-0750
http://www.ct.gov/dph/lib/dph/environmental_health/environmental_laboratories/pdf/OutofStateCommercialLaboratories.pdf

Maine Potable and Non Potable Water, and Solid and Hazardous Waste: RI00002
http://www.maine.gov/dhhs/mecdc/environmental-health/dwp/partners/labCert.shtml

Massachusetts Potable and Non Potable Water: M-RI002
http://public.dep.state.ma.us/Labcert/Labcert.aspx

New Hampshire (NELAP accredited) Potable and Non Potable Water, Solid and Hazardous Waste: 2424
http://des.nh.gov/organization/divisions/water/dwgb/nhelap/index.htm

New York (NELAP accredited) Non Potable Water, Solid and Hazardous Waste: 11313
http://www.wadsworth.org/labcert/elap/comm.html

New Jersey (NELAP accredited) Non Potable Water, Solid and Hazardous Waste: RI006
http://datamine2.state.nj.us/DEP_OPRA/OpraMain/pi_main?mode=pi_by_site&sort_order=PI_NAMEA&Select+a+Site:=58715

United States Department of Agriculture Soil Permit: P330-12-00139

Pennsylvania: 68-01752
http://www.dep.pa.gov/Business/OtherPrograms/Labs/Pages/Laboratory-Accreditation-Program.aspx

THIELSCH ENGINEERING	195 Frances Avenue Cranston RI, 02910 Phone: (401)-467-6454 Fax: (401)-467-2398 thielsch.com *Let's Build a Solid Foundation*	Client Information: Horsely Witten Group Sandwich, MA PM: Neal Price Assigned By: Neal Price Collected By: Client	Project Information: **Ipswich Mills Dam** **Ipswich, MA** ESS Project Number: 21F0999 Summary Page: 1 of 2 Report Date: 07.02.21

LABORATORY TESTING DATA SHEET, Report No.: 7421-F-235

Boring No	Sample No.	Depth (Ft)	Laboratory No.	Identification Tests								Proctor / CBR / Permeability Tests								Laboratory Log and Soil Description
				As Received Moisture Content %	LL %	PL %	Gravel %	Sand %	Fines %	Org. %	G_s	Dry unit wt. pcf	Test Moisture Content %	γ_d MAX (pcf) W_{opt} (%)	γ_d MAX (pcf) W_{opt} (%) (Corr.)	Target Test Setup as % of Proctor	CBR @ 0.1"	CBR @ 0.2"	Permeability cm/sec	
				D2216	D4318		D6913			D2974	D854			D1557						
	SS1		21F0999-01				77.0	22.7	0.3											Dark Brown well-graded gravel with sand
	SS2		21F0999-02				63.9	35.2	0.9											Dark Brown well-graded gravel with sand
	SS3		21F0999-03				84.2	15.2	0.6											Brown poorly graded gravel with sand
	SS4		21F0999-04				88.8	10.2	1.0											Brown well-graded gravel
	SS5		21F0999-05				77.3	21.9	0.8											Brown poorly graded gravel with sand
	SS6		21F0999-06				67.9	31.2	0.9											Brown well-graded gravel with sand
	SS7		21F0999-07				20.1	40.7	39.2											Brown silty sand with gravel
	SS8		21F0999-08				33.6	44.3	22.1											Brown silty sand with gravel

Date Received: 06.28.21 Reviewed By: Date Reviewed: 07.02.21

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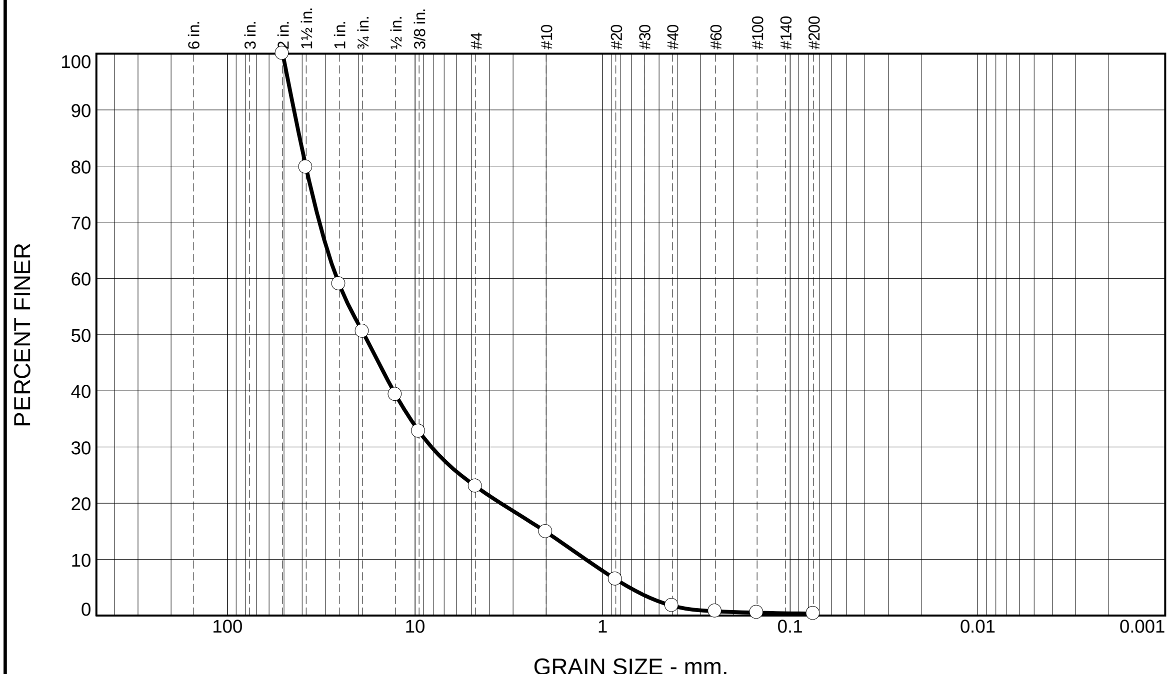
THIELSCH ENGINEERING	195 Frances Avenue Cranston RI, 02910 Phone: (401)-467-6454 Fax: (401)-467-2398 thielsch.com *Let's Build a Solid Foundation*	Client Information: Horsely Witten Group Sandwich, MA PM: Neal Price Assigned By: Neal Price Collected By: Client	Project Information: **Ipswich Mills Dam** **Ipswich, MA** ESS Project Number: 21F0999 Summary Page: 2 of 2 Report Date: 07.02.21

LABORATORY TESTING DATA SHEET, Report No.: 7421-F-235

Boring No	Sample No.	Depth (Ft)	Laboratory No.	Identification Tests: As Received Moisture Content %	LL %	PL %	Gravel %	Sand %	Fines %	Org. %	G_s	Proctor / CBR / Permeability Tests: Dry unit wt. pcf	Test Moisture Content %	γ_d MAX (pcf) W_{opt} (%)	γ_d MAX (pcf) W_{opt} (%) (Corr.)	Target Test Setup as % of Proctor	CBR @ 0.1"	CBR @ 0.2"	Permeability cm/sec	Laboratory Log and Soil Description
				D2216	D4318		D6913			D2974	D854			D1557						
	SS9		21F0999-09				0.0	52.2	47.8											Brown silty sand
	SS10		21F0999-10				0.0	58.6	41.4											Brown silty sand
	SS11		21F0999-11				45.7	16.8	37.5											Brown silty gravel with sand
	SS12		21F0999-12				0.0	86.8	13.2											Brown silty sand
	SS13		21F0999-13				68.2	31.1	0.7											Brown well-graded gravel with sand
	SS14		21F0999-14				0.0	72.6	27.4											Brown silty sand
	SS15		21F0999-15				94.7	4.9	0.4											Brown poorly graded gravel

Date Received: *06.28.21* Reviewed By: [signature] Date Reviewed: *07.02.21*

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	49.5	27.5	8.1	13.2	1.4	0.3	

Test Results (D6913 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
2"	100.0		
1.5"	79.7		
1"	59.0		
0.75"	50.5		
0.5"	39.3		
0.375"	32.8		
#4	23.0		
#10	14.9		
#20	6.4		
#40	1.7		
#60	0.8		
#100	0.5		
#200	0.3		

* (no specification provided)

Material Description

Dark Brown well-graded gravel with sand

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= GW **AASHTO (M 145)=** A-1-a

Coefficients

D_{90}= 44.2628 D_{85}= 41.2151 D_{60}= 26.0967
D_{50}= 18.6867 D_{30}= 8.1739 D_{15}= 2.0269
D_{10}= 1.2291 C_u= 21.23 C_c= 2.08

Remarks

Date Received: 06.28.21 **Date Tested:** 07.01.21

Tested By: MS

Checked By: Rebecca Roth

Title: Laboratory Coordinator

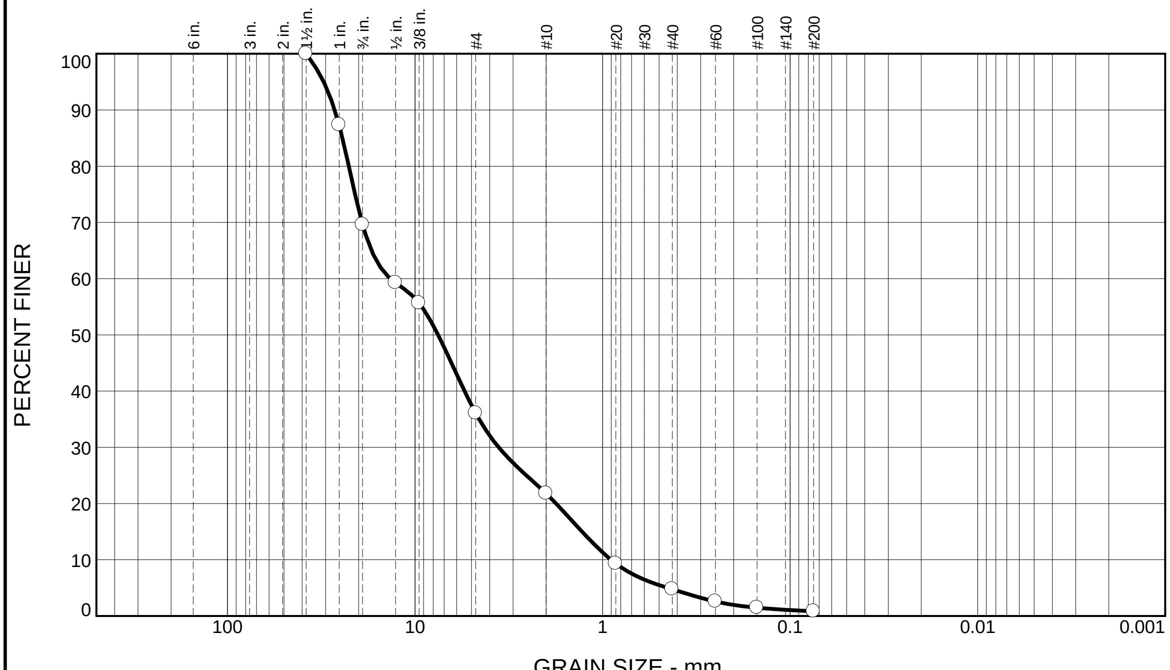
Source of Sample: 21F0999 **Date Sampled:**
Sample Number: SS1

Thielsch Engineering Inc.

Cranston, RI

Client: Horsley Witten Group
Project: Ipswitch Mills Dam
Ipswich, MA
Project No: 21F099 **Figure** 21F0999-01

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	30.4	33.5	14.3	17.0	3.9	0.9	

Test Results (D6913 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1.5"	100.0		
1"	87.3		
0.75"	69.6		
0.5"	59.3		
0.375"	55.7		
#4	36.1		
#10	21.8		
#20	9.3		
#40	4.8		
#60	2.6		
#100	1.5		
#200	0.9		

* (no specification provided)

Material Description

Dark Brown well-graded gravel with sand

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= GW **AASHTO (M 145)=** A-1-a

Coefficients

D_{90}= 26.7978 D_{85}= 24.3891 D_{60}= 13.5193
D_{50}= 7.5296 D_{30}= 3.5812 D_{15}= 1.2861
D_{10}= 0.9023 C_u= 14.98 C_c= 1.05

Remarks

Date Received: 06.28.21 **Date Tested:** 07.01.21

Tested By: MS

Checked By: Rebecca Roth

Title: Laboratory Coordinator

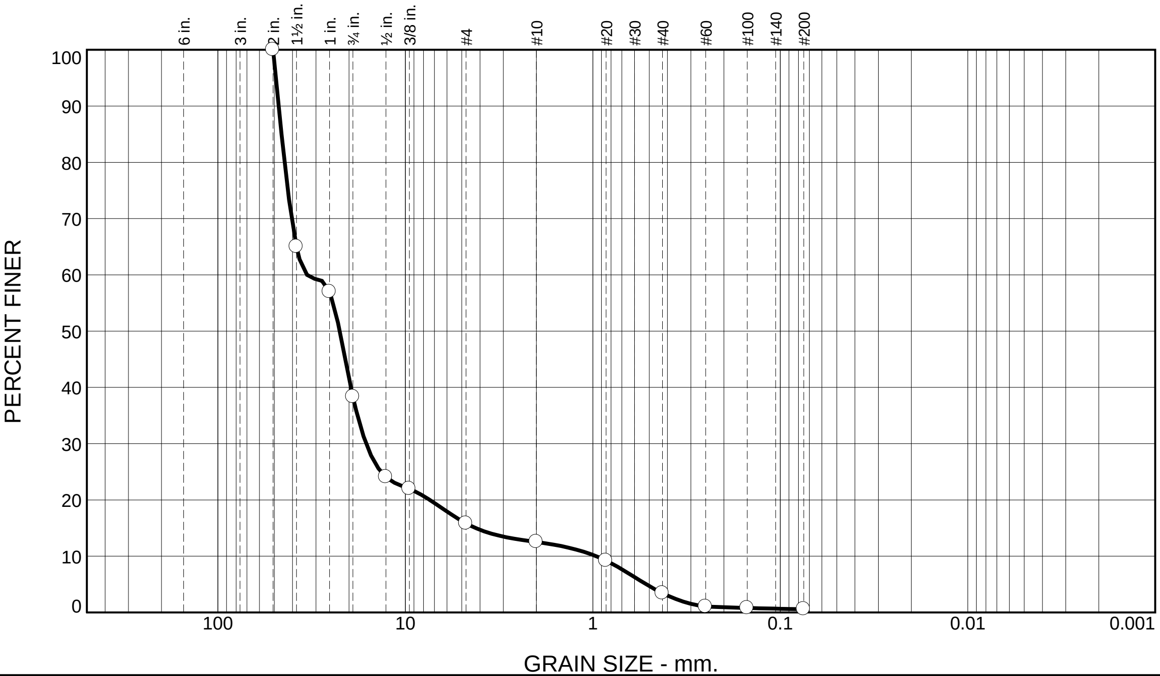
Source of Sample: 21F0999 **Date Sampled:**
Sample Number: SS2

Thielsch Engineering Inc.

Cranston, RI

Client: Horsley Witten Group
Project: Ipswitch Mills Dam
Ipswich, MA
Project No: 21F099 **Figure** 21F0999-02

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	61.7	22.5	3.3	9.1	2.8	0.6	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
2"	100.0		
1.5"	65.0		
1"	57.0		
0.75"	38.3		
0.5"	24.1		
0.375"	22.0		
#4	15.8		
#10	12.5		
#20	9.2		
#40	3.4		
#60	1.0		
#100	0.8		
#200	0.6		

* (no specification provided)

Material Description

Brown poorly graded gravel with sand

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= GP **AASHTO (M 145)=** A-1-a

Coefficients

D_{90}= 47.4566 D_{85}= 45.8050 D_{60}= 33.4714
D_{50}= 22.3914 D_{30}= 16.2044 D_{15}= 4.2133
D_{10}= 0.9609 C_u= 34.83 C_c= 8.16

Remarks

Date Received: 06.28.21 **Date Tested:** 07.01.21

Tested By: MS

Checked By: Rebecca Roth

Title: Laboratory Coordinator

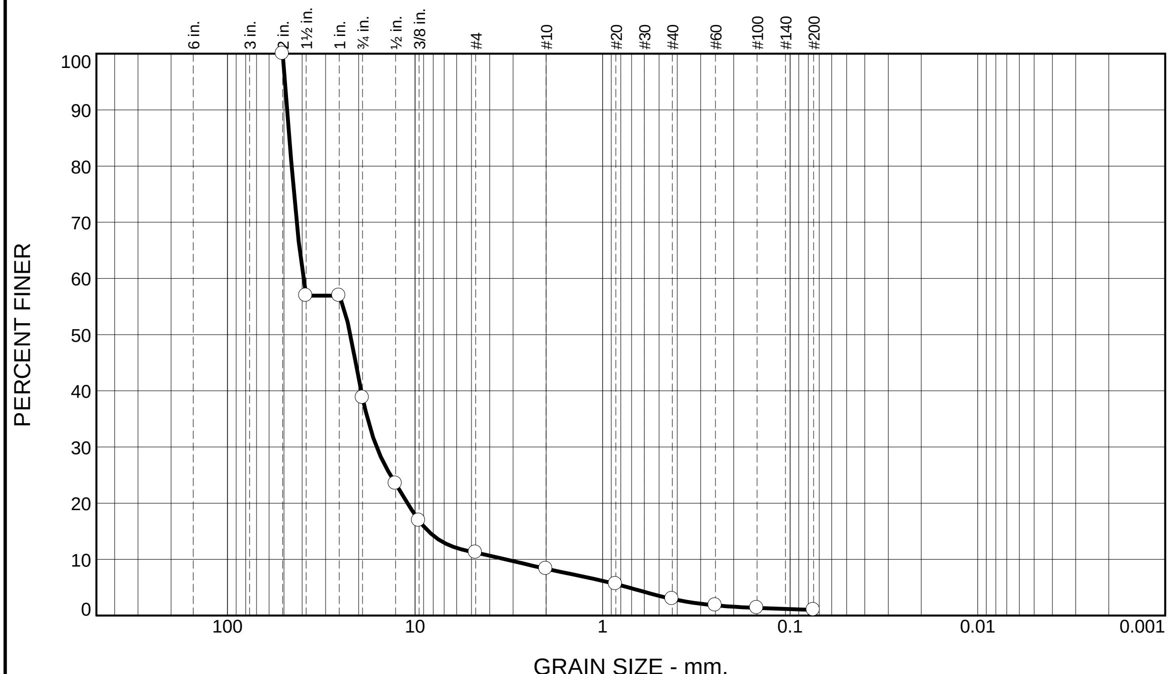
Source of Sample: 21F0999 **Date Sampled:**
Sample Number: SS3

Thielsch Engineering Inc.

Cranston, RI

Client: Horsley Witten Group
Project: Ipswitch Mills Dam
Ipswich, MA
Project No: 21F099 **Figure** 21F0999-03

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	61.2	27.6	2.9	5.3	2.0	1.0	

Test Results (D6913 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
2"	100.0		
1.5"	56.9		
1"	56.9		
0.75"	38.8		
0.5"	23.5		
0.375"	16.9		
#4	11.2		
#10	8.3		
#20	5.6		
#40	3.0		
#60	1.8		
#100	1.4		
#200	1.0		

* (no specification provided)

Material Description

Brown well-graded gravel

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= GW **AASHTO (M 145)=** A-1-a

Coefficients

D_{90}= 48.1584 D_{85}= 46.8545 D_{60}= 39.4865
D_{50}= 22.1365 D_{30}= 16.0441 D_{15}= 8.4826
D_{10}= 3.2849 C_u= 12.02 C_c= 1.98

Remarks

Date Received: 06.28.21 **Date Tested:** 07.01.21

Tested By: MS

Checked By: Rebecca Roth

Title: Laboratory Coordinator

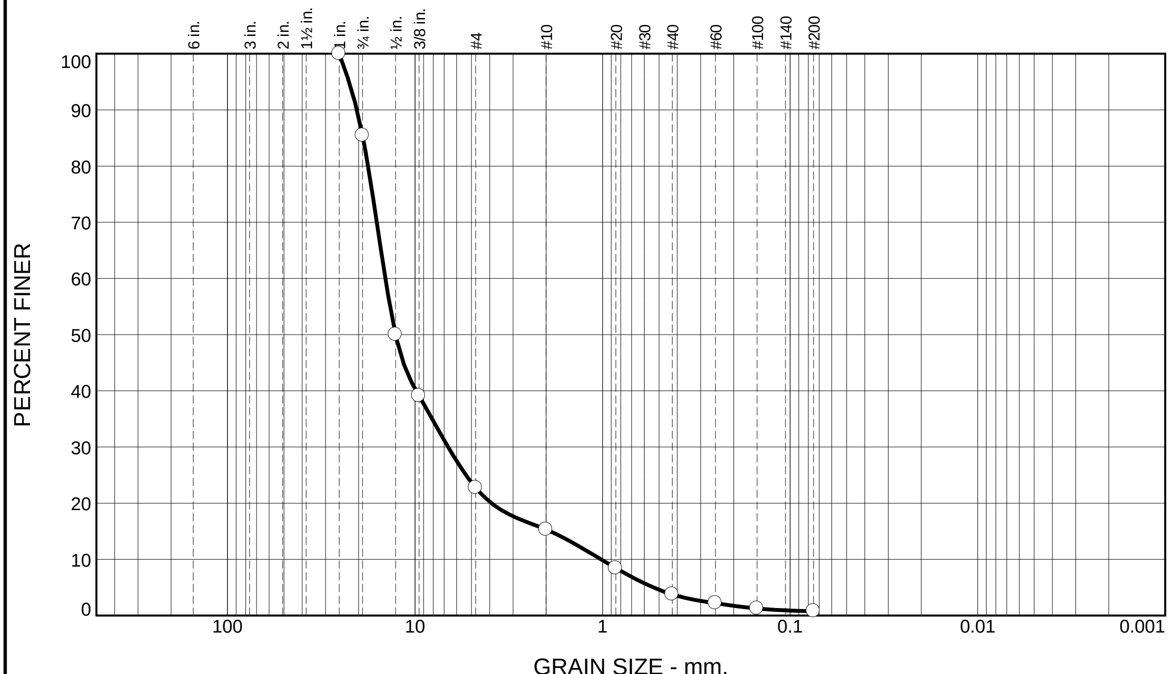
Source of Sample: 21F0999 **Date Sampled:**
Sample Number: SS4

Thielsch Engineering Inc.

Cranston, RI

Client: Horsley Witten Group
Project: Ipswitch Mills Dam
Ipswich, MA
Project No: 21F099 **Figure** 21F0999-04

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	14.6	62.7	7.4	11.5	3.0	0.8	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1"	100.0		
0.75"	85.4		
0.5"	50.0		
0.375"	39.1		
#4	22.7		
#10	15.3		
#20	8.4		
#40	3.8		
#60	2.2		
#100	1.3		
#200	0.8		

* (no specification provided)

Material Description

Brown poorly graded gravel with sand

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= GP **AASHTO (M 145)=** A-1-a

Coefficients

D_{90}= 20.4093 D_{85}= 18.9398 D_{60}= 14.4019
D_{50}= 12.7012 D_{30}= 6.6598 D_{15}= 1.9024
D_{10}= 1.0195 C_u= 14.13 C_c= 3.02

Remarks

Date Received: 06.28.21 **Date Tested:** 07.01.21

Tested By: MS

Checked By: Rebecca Roth

Title: Laboratory Coordinator

Source of Sample: 21F0999
Sample Number: SS5

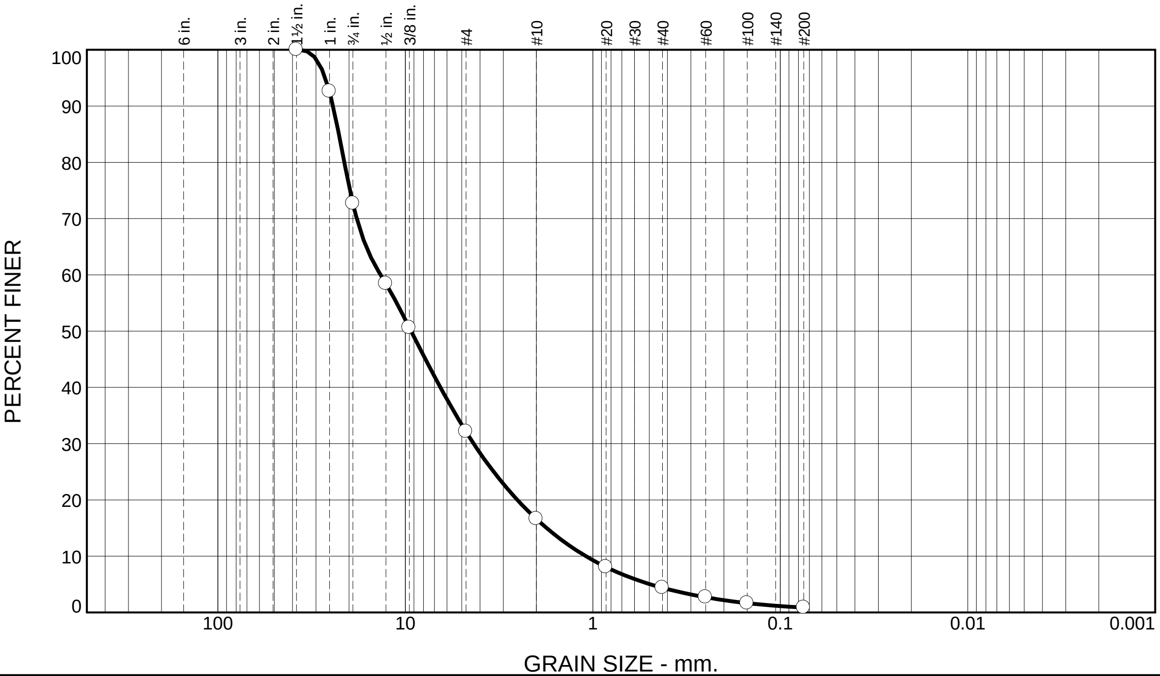
Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

Client: Horsley Witten Group
Project: Ipswitch Mills Dam
Ipswich, MA
Project No: 21F099 **Figure** 21F0999-05

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	27.3	40.6	15.5	12.2	3.5	0.9	

Test Results (D6913 & ASTM D 1140)			
Opening Size	**Percent Finer**	**Spec.* (Percent)**	**Pass? (X=Fail)**
1-1/2"	100.0		
1"	92.6		
3/4"	72.7		
1/2"	58.4		
3/8"	50.6		
#4	32.1		
#10	16.6		
#20	8.1		
#40	4.4		
#60	2.7		
#100	1.6		
#200	0.9		

* (no specification provided)

Material Description

Brown well-graded gravel with sand

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= GW **AASHTO (M 145)=** A-1-a

Coefficients

D_{90}= 24.3090 D_{85}= 22.6224 D_{60}= 13.5506
D_{50}= 9.3223 D_{30}= 4.3216 D_{15}= 1.7637
D_{10}= 1.0889 C_u= 12.44 C_c= 1.27

Remarks

Date Received: 06.28.21 **Date Tested:** 07.01.21

Tested By: MS

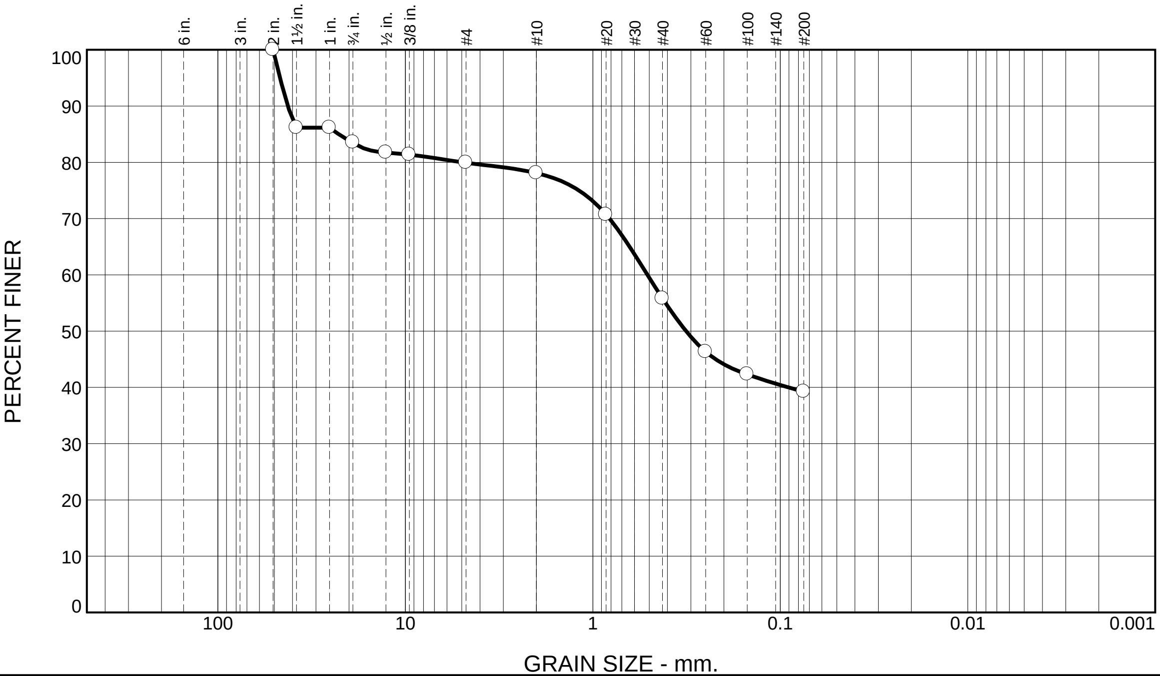
Checked By: Rebecca Roth

Title: Laboratory Coordinator

Source of Sample: 21F0999 **Date Sampled:**
Sample Number: SS6

Thielsch Engineering Inc.	**Client:** Horsley Witten Group
	Project: Ipswitch Mills Dam Ipswich, MA
Cranston, RI	**Project No:** 21F099 **Figure** 21F0999-06

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	16.4	3.7	1.8	22.3	16.6	39.2	

Test Results (D6913 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
2"	100.0		
1.5"	86.2		
1"	86.2		
0.75"	83.6		
0.5"	81.7		
0.375"	81.4		
#4	79.9		
#10	78.1		
#20	70.7		
#40	55.8		
#60	46.3		
#100	42.3		
#200	39.2		

* (no specification provided)

Material Description

Brown silty sand with gravel

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= SM **AASHTO (M 145)=** A-4(0)

Coefficients

D_{90}= 42.3654 D_{85}= 22.6606 D_{60}= 0.5110
D_{50}= 0.3180 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Sample visually classified as non-plastic.

Date Received: 06.28.21 **Date Tested:** 07.01.21

Tested By: MS

Checked By: Rebecca Roth

Title: Laboratory Coordinator

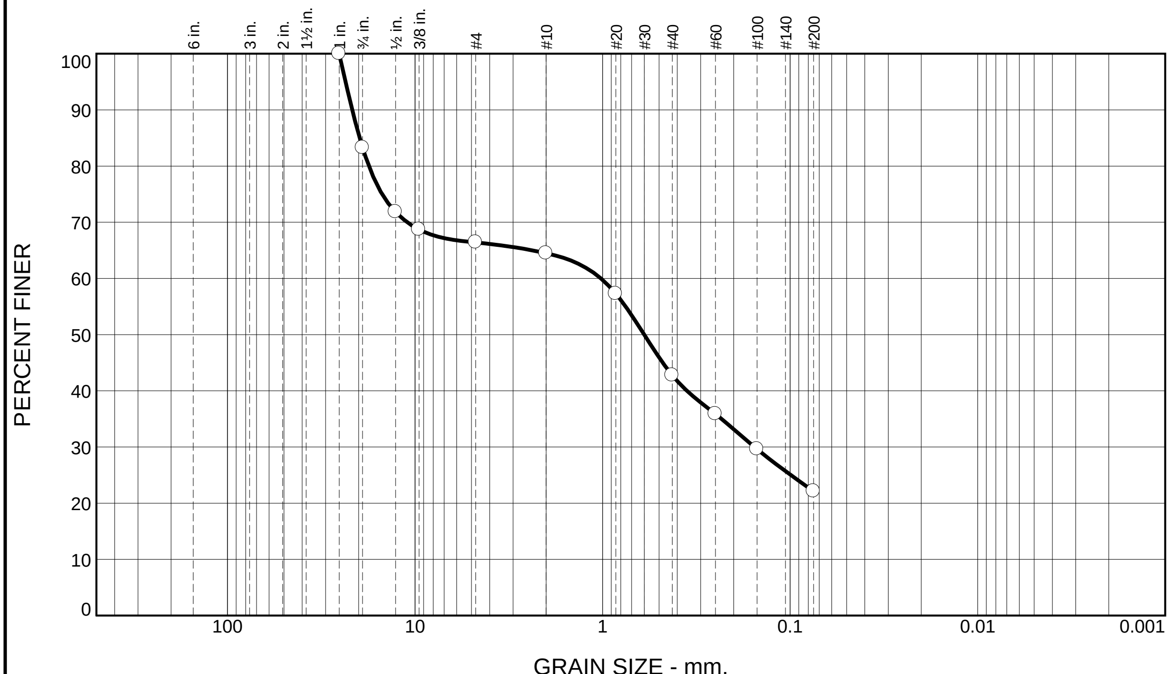
Source of Sample: 21F0999 **Date Sampled:**
Sample Number: SS7

Thielsch Engineering Inc.

Cranston, RI

Client: Horsley Witten Group
Project: Ipswitch Mills Dam
Ipswich, MA
Project No: 21F099 **Figure** 21F0999-07

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	16.7	16.9	1.9	21.7	20.7	22.1	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1"	100.0		
0.75"	83.3		
0.5"	71.8		
0.375"	68.7		
#4	66.4		
#10	64.5		
#20	57.3		
#40	42.8		
#60	35.9		
#100	29.6		
#200	22.1		

* (no specification provided)

Material Description

Brown silty sand with gravel

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= SM **AASHTO (M 145)=** A-1-b

Coefficients

D_{90}= 21.6191 D_{85}= 19.7364 D_{60}= 1.0214
D_{50}= 0.6001 D_{30}= 0.1547 D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 06.28.21 **Date Tested:** 07.01.21

Tested By: MS

Checked By: Rebecca Roth

Title: Laboratory Coordinator

Source of Sample: 21F0999
Sample Number: SS8

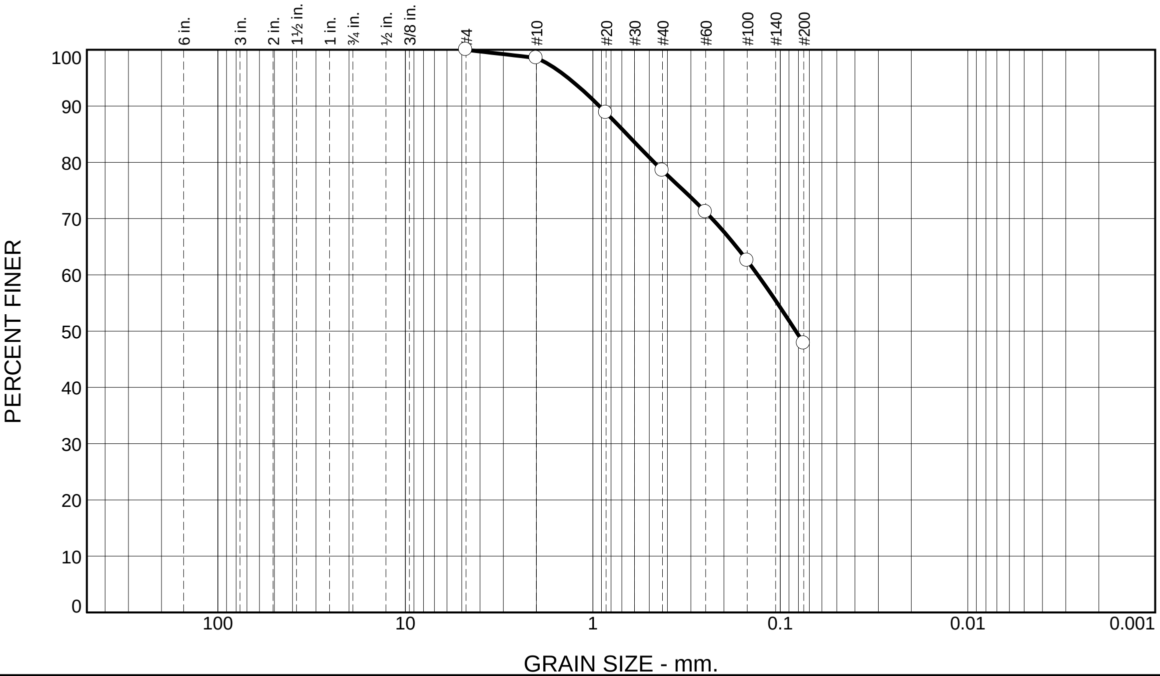
Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

Client: Horsley Witten Group
Project: Ipswitch Mills Dam
Ipswich, MA
Project No: 21F099 **Figure** 21F0999-08

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	1.4	20.1	30.7	47.8	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#4	100.0		
#10	98.6		
#20	88.8		
#40	78.5		
#60	71.2		
#100	62.5		
#200	47.8		

* (no specification provided)

Material Description

Brown silty sand

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= SM **AASHTO (M 145)=** A-4(0)

Coefficients

D_{90}= 0.9233 D_{85}= 0.6585 D_{60}= 0.1319
D_{50}= 0.0827 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Sample visually classified as non-plastic.

Date Received: 06.28.21 **Date Tested:** 07.01.21

Tested By: MS

Checked By: Rebecca Roth

Title: Laboratory Coordinator

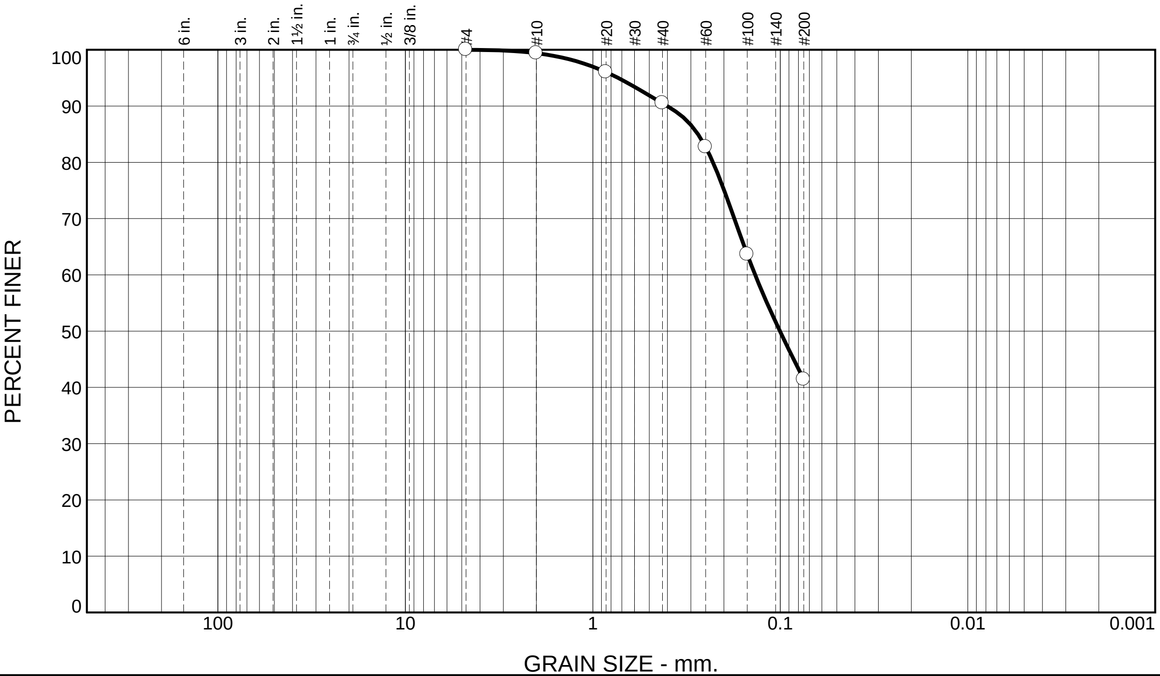
Source of Sample: 21F0999 **Date Sampled:**
Sample Number: SS9

Thielsch Engineering Inc.

Cranston, RI

Client: Horsley Witten Group
Project: Ipswitch Mills Dam
Ipswich, MA
Project No: 21F099 **Figure** 21F0999-09

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.6	8.9	49.1	41.4	

Test Results (D6913 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#4	100.0		
#10	99.4		
#20	96.0		
#40	90.5		
#60	82.7		
#100	63.6		
#200	41.4		

* (no specification provided)

Material Description

Brown silty sand

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= SM **AASHTO (M 145)=** A-4(0)

Coefficients

D_{90}= 0.4006 D_{85}= 0.2747 D_{60}= 0.1361
D_{50}= 0.1005 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Sample visually classified as non-plastic.

Date Received: 06.28.21 **Date Tested:** 07.01.21

Tested By: MS

Checked By: Rebecca Roth

Title: Laboratory Coordinator

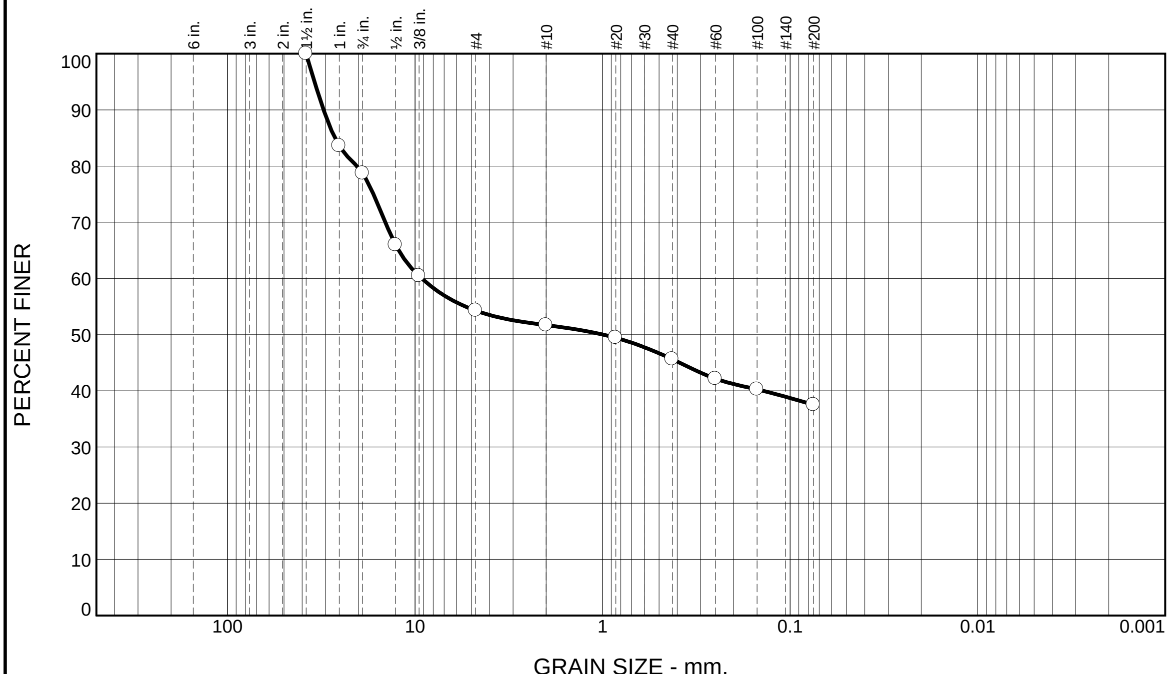
Source of Sample: 21F0999 **Date Sampled:**
Sample Number: SS10

Thielsch Engineering Inc.

Cranston, RI

Client: Horsley Witten Group
Project: Ipswitch Mills Dam
Ipswich, MA
Project No: 21F099 **Figure** 21F0999-10

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	21.3	24.4	2.6	6.1	8.1	37.5	

Test Results (D6913 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1-1/2"	100.0		
1"	83.6		
3/4"	78.7		
1/2"	66.0		
3/8"	60.5		
#4	54.3		
#10	51.7		
#20	49.4		
#40	45.6		
#60	42.2		
#100	40.2		
#200	37.5		

* (no specification provided)

Material Description

Brown silty gravel with sand

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= GM **AASHTO (M 145)=** A-4(0)

Coefficients

D_{90}= 30.7162 D_{85}= 26.7479 D_{60}= 9.2051
D_{50}= 0.9905 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Sample visually classified as non-plastic.

Date Received: 06.28.21 **Date Tested:** 07.01.21

Tested By: MS

Checked By: Rebecca Roth

Title: Laboratory Coordinator

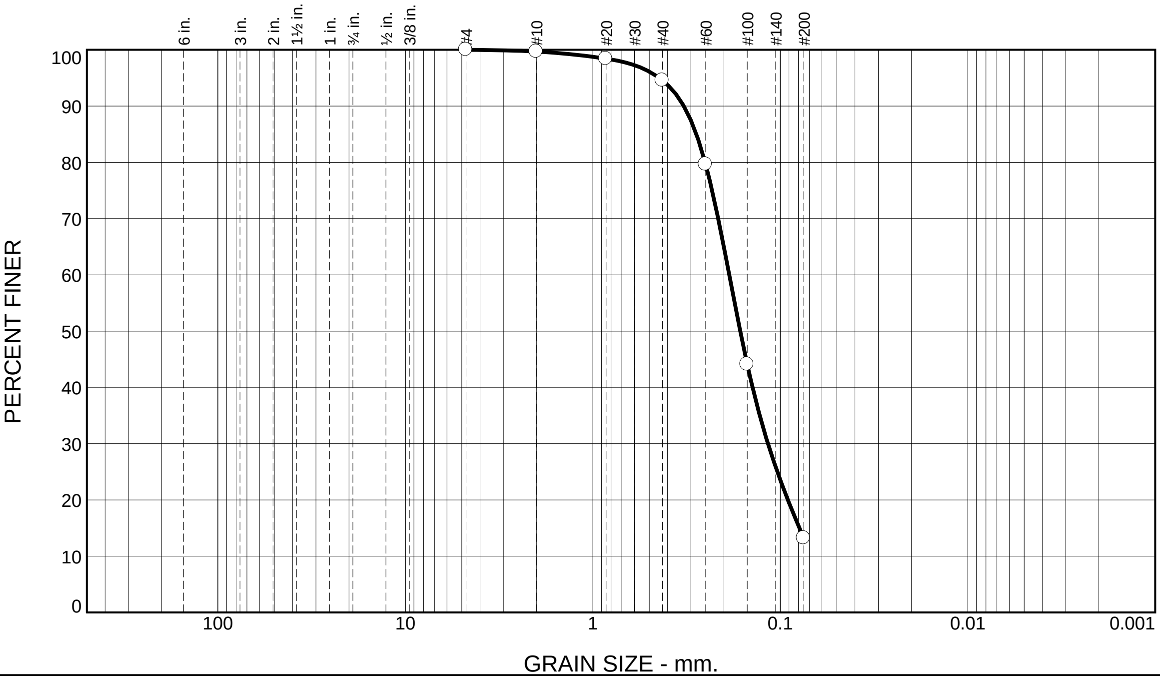
Source of Sample: 21F0999 **Date Sampled:**
Sample Number: SS11

Thielsch Engineering Inc.

Cranston, RI

Client: Horsley Witten Group
Project: Ipswitch Mills Dam
Ipswich, MA
Project No: 21F099 **Figure** 21F0999-11

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.3	5.2	81.3	13.2	

Test Results (D6913 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#4	100.0		
#10	99.7		
#20	98.4		
#40	94.5		
#60	79.6		
#100	44.1		
#200	13.2		

* (no specification provided)

Material Description

Brown silty sand

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= SM **AASHTO (M 145)=** A-2-4(0)

Coefficients

D_{90}= 0.3270 D_{85}= 0.2802 D_{60}= 0.1871
D_{50}= 0.1635 D_{30}= 0.1164 D_{15}= 0.0791
D_{10}= C_u= C_c=

Remarks

Date Received: 06.28.21 **Date Tested:** 07.01.21

Tested By: MS

Checked By: Rebecca Roth

Title: Laboratory Coordinator

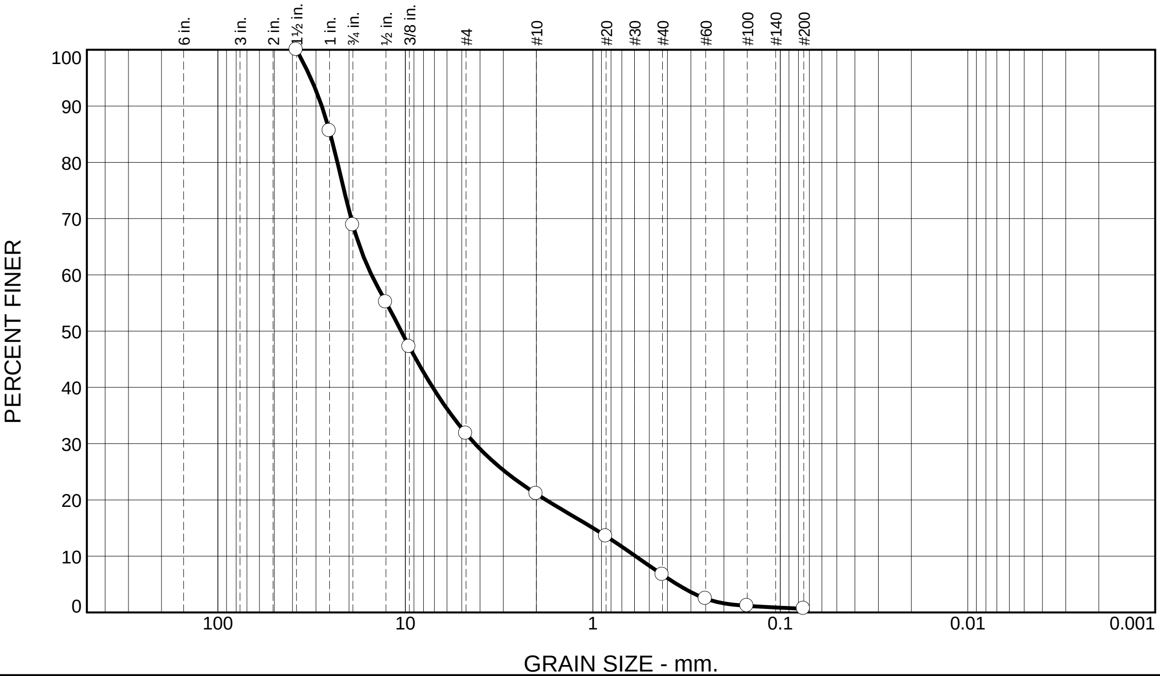
Source of Sample: 21F0999 **Date Sampled:**
Sample Number: SS12

Thielsch Engineering Inc.

Cranston, RI

Client: Horsley Witten Group
Project: Ipswitch Mills Dam
Ipswich, MA
Project No: 21F099 **Figure** 21F0999-12

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	31.1	37.1	10.7	14.4	6.0	0.7	

Test Results (D6913 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1-1/2"	100.0		
1"	85.6		
3/4"	68.9		
1/2"	55.2		
3/8"	47.2		
#4	31.8		
#10	21.1		
#20	13.5		
#40	6.7		
#60	2.5		
#100	1.2		
#200	0.7		

* (no specification provided)

Material Description

Brown well-graded gravel with sand

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= GW **AASHTO (M 145)=** A-1-a

Coefficients

D_{90}= 27.8934 D_{85}= 25.1220 D_{60}= 15.1635
D_{50}= 10.5389 D_{30}= 4.2533 D_{15}= 0.9976
D_{10}= 0.5914 C_u= 25.64 C_c= 2.02

Remarks

Date Received: 06.28.21 **Date Tested:** 07.01.21

Tested By: MS

Checked By: Rebecca Roth

Title: Laboratory Coordinator

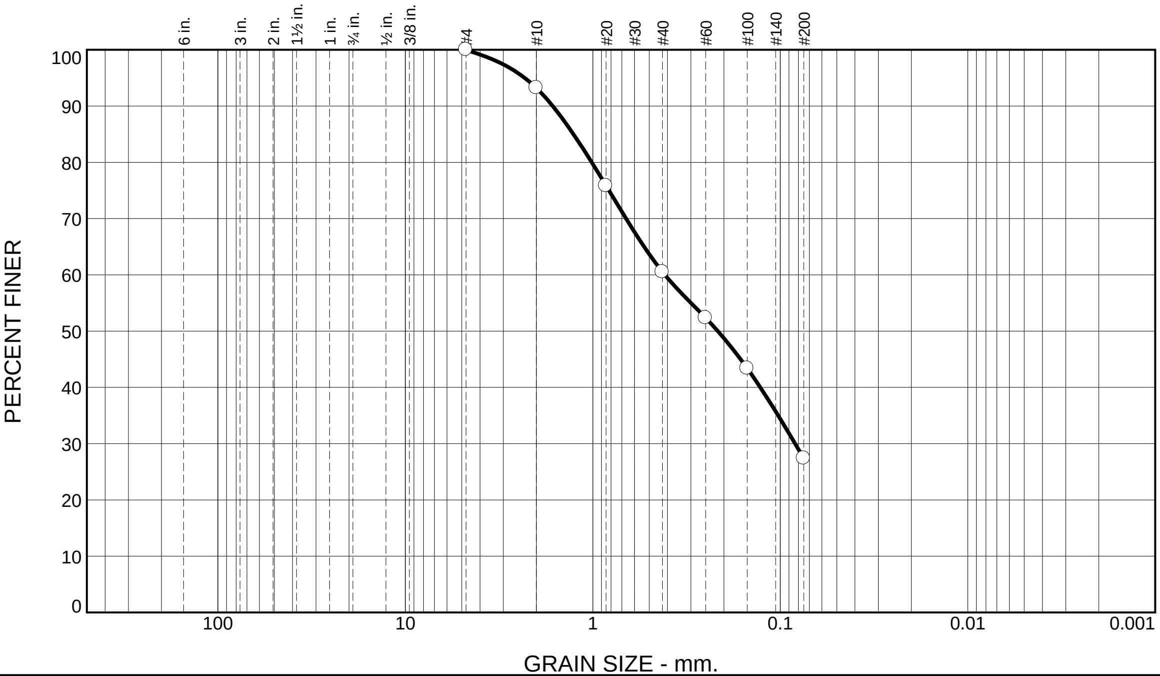
Source of Sample: 21F0999 **Date Sampled:**
Sample Number: SS13

Thielsch Engineering Inc.

Cranston, RI

Client: Horsley Witten Group
Project: Ipswitch Mills Dam
Ipswich, MA
Project No: 21F099 **Figure** 21F0999-13

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	6.8	32.7	33.1	27.4	

Test Results (D6913 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#4	100.0		
#10	93.2		
#20	75.8		
#40	60.5		
#60	52.3		
#100	43.4		
#200	27.4		

* (no specification provided)

Material Description

Brown silty sand

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= SM **AASHTO (M 145)=** A-2-4(0)

Coefficients

$D_{90}=$ 1.6357 $D_{85}=$ 1.2685 $D_{60}=$ 0.4128
$D_{50}=$ 0.2155 $D_{30}=$ 0.0834 $D_{15}=$
$D_{10}=$ $C_u=$ $C_c=$

Remarks

Date Received: 06.28.21 **Date Tested:** 07.01.21

Tested By: MS

Checked By: Rebecca Roth

Title: Laboratory Coordinator

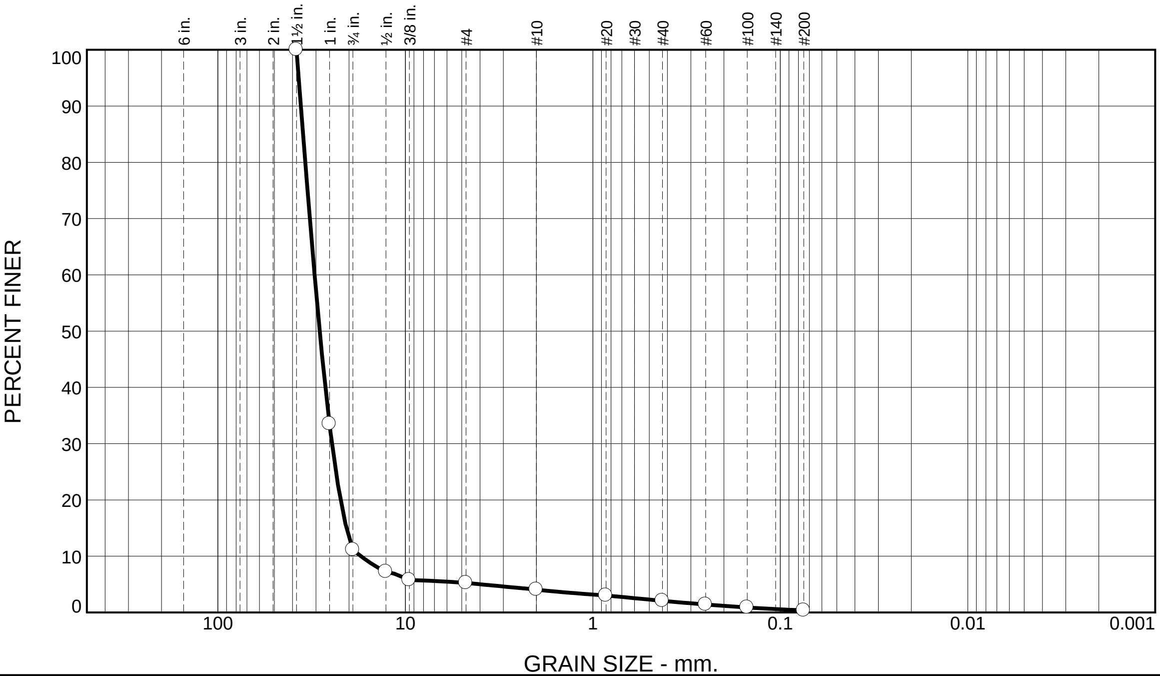
Source of Sample: 21F0999 **Date Sampled:**
Sample Number: SS14

Thielsch Engineering Inc.

Cranston, RI

Client: Horsley Witten Group
Project: Ipswitch Mills Dam
Ipswich, MA
Project No: 21F099 **Figure** 21F0999-14

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	88.9	5.8	1.2	2.0	1.7	0.4	

Test Results (D6913 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1-1/2"	100.0		
1"	33.5		
3/4"	11.1		
1/2"	7.2		
3/8"	5.8		
#4	5.3		
#10	4.1		
#20	3.0		
#40	2.1		
#60	1.4		
#100	0.9		
#200	0.4		

* (no specification provided)

Material Description

Brown poorly graded gravel

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= GP **AASHTO (M 145)=** A-1-a

Coefficients

D_{90}= 36.1001 D_{85}= 35.1324 D_{60}= 30.4793
D_{50}= 28.6320 D_{30}= 24.6435 D_{15}= 20.6197
D_{10}= 17.2830 C_u= 1.76 C_c= 1.15

Remarks

Date Received: 06.28.21 **Date Tested:** 07.01.21

Tested By: MS

Checked By: Rebecca Roth

Title: Laboratory Coordinator

Source of Sample: 21F0999 **Date Sampled:**
Sample Number: SS15

Thielsch Engineering Inc.

Cranston, RI

Client: Horsley Witten Group
Project: Ipswitch Mills Dam
Ipswich, MA
Project No: 21F099 **Figure** 21F0999-15

ESS Laboratory Sample and Cooler Receipt Checklist

Client: Horsley Witten Group - KPB

Shipped/Delivered Via: ESS Courier

ESS Project ID: 21F0999
Date Received: 6/25/2021
Project Due Date: 7/2/2021
Days for Project: 5 Day

1. Air bill manifest present? No
   Air No.: NA
2. Were custody seals present? No
3. Is radiation count <100 CPM? Yes
4. Is a Cooler Present? Yes
   Temp: 2.9 Iced with: Ice
5. Was COC signed and dated by client? Yes

6. Does COC match bottles? Yes
7. Is COC complete and correct? Yes
8. Were samples received intact? Yes
9. **Were labs informed about <u>short holds & rushes</u>?** Yes / No / NA
10. Were any analyses received outside of hold time? Yes / No

11. Any Subcontracting needed? Yes / No
    ESS Sample IDs: 1 - 15
    Analysis: Grainsize
    TAT: 5

12. Were VOAs received? Yes / No
    a. Air bubbles in aqueous VOAs? Yes / No
    b. Does methanol cover soil completely? Yes / No / NA

13. Are the samples properly preserved? Yes / No
    a. If metals preserved upon receipt: Date: ____ Time: ____ By: ____
    b. Low Level VOA vials frozen: Date: ____ Time: ____ By: ____

Sample Receiving Notes:

14. Was there a need to contact Project Manager? Yes / No
    a. Was there a need to contact the client? Yes / No
    Who was contacted? ____ Date: ____ Time: ____ By: ____

Sample Number	Container ID	Proper Container	Air Bubbles Present	Sufficient Volume	Container Type	Preservative	Record pH (Cyanide and 608 Pesticides)
1	181304	Yes	N/A	Yes	Other Glass	NP	
2	181305	Yes	N/A	Yes	Other Glass	NP	
3	181306	Yes	N/A	Yes	Other Glass	NP	
4	181307	Yes	N/A	Yes	Other Glass	NP	
5	181308	Yes	N/A	Yes	Other Glass	NP	
6	181309	Yes	N/A	Yes	Other Glass	NP	
7	181310	Yes	N/A	Yes	Other Glass	NP	
8	181311	Yes	N/A	Yes	Other Glass	NP	
9	181312	Yes	N/A	Yes	Other Glass	NP	
10	181313	Yes	N/A	Yes	Other Glass	NP	
11	181314	Yes	N/A	Yes	1L Amber	NP	
12	181315	Yes	N/A	Yes	1L Amber	NP	
13	181316	Yes	N/A	Yes	1L Amber	NP	
14	181317	Yes	N/A	Yes	1L Amber	NP	
15	181318	Yes	N/A	Yes	1L Amber	NP	

ESS Laboratory Sample and Cooler Receipt Checklist

Client: Horsley Witten Group - KPB

ESS Project ID: 21F0999
Date Received: 6/25/2021

2nd Review

Were all containers scanned into storage/lab? Initials TD

Are barcode labels on correct containers?	Yes / No
Are all Flashpoint stickers attached/container ID # circled?	Yes / No / NA
Are all Hex Chrome stickers attached?	Yes / No / NA
Are all QC stickers attached?	Yes / No / NA
Are VOA stickers attached if bubbles noted?	Yes / No / NA

Completed By: Taylor Davis Date & Time: 6/25/21 1856

Reviewed By: [signature] Date & Time: 6/25/21 1959

ESS LABORATORY

185 Frances Avenue
Cranston, RI 02921
Phone: 401-461-7181
Fax: 401-461-4486
www.esslaboratory.com

CHAIN OF CUSTODY

ESS Lab # 21F0999 | Page of

Turn Time ☐ >5 ☑ 5 ☐ 4 ☐ 3 ☐ 2 ☐ 1 ☐ Same Day

Regulatory State: Criteria:

Is this project for any of the following?: ☐ CT RCP ☐ MA MCP ☐ RGP ☐ Permit ☐ 401 WQ

ELECTRONIC DELIVERABLES (Final Reports are PDF)

☐ Limit Checker ☐ State Forms ☐ EQuIS
☐ Excel ☐ Hard Copy ☐ Enviro Data
☐ CLP-Like Package ☐ Other (Specify) →

CLIENT INFORMATION

Client: Horsley Witten Group
Address: 90 Route 6A
Sandwich, MA 02563
Phone: (508) 833-6600
Email Distribution List: Jonas Procton
Neal Price

PROJECT INFORMATION

Project Name: Ipswich Mills Dam
Project Location: Ipswich, MA
Project Number: 16041E
Project Manager: Neal Price
Bill to:
PO#:
Quote#:

Client acknowledges that sampling is compliant with all EPA / State regulatory programs

REQUESTED ANALYSES

ESS Lab ID	Collection Date	Collection Time	Sample Type	Sample Matrix	Sample ID	Grain Size	Total Number of Bottles
1	6/23/21	10:02 AM	Grab	Soil	SS1	X	
2	6/23/21	10:13 AM			SS2	X	
3	6/23/21	10:26 AM			SS3	X	
4	6/23/21	10:49 AM			SS4	X	
5	6/23/21	11:04 AM			SS5	X	
6	6/23/21	11:20 AM			SS6	X	
7	6/23/21	12:45 PM			SS7	X	
8	6/23/21	1:00 PM			SS8	X	
					hdm 6/28/21		

Container Type: AC-Air Cassette AG-Amber Glass B-BOD Bottle C-Cubitainer J-Jar O-Other P-Poly S-Sterile V-Vial

Container Volume: 1-100 mL 2-2.5 gal 3-250 mL 4-300 mL 5-500 mL 6-1L 7-VOA 8-2 oz 9-4 oz 10-8 oz 11-Other*

Preservation Code: 1-Non Preserved 2-HCl 3-H2SO4 4-HNO3 5-NaOH 6-Methanol 7-Na2S2O3 8-ZnAce, NaOH 9-NH4Cl 10-DI H2O 11-Other*

Sampled by : Jonas Procton

Chain needs to be filled out neatly and completely for on time delivery.

Laboratory Use Only
Cooler Temperature (°C): 2.9
/ Ce

Comments: * Please specify "Other" preservative and containers types in this space

All samples submitted are subject to ESS Laboratory's payment terms and conditions.

Dissolved Filtration
☐ Lab Filter

Relinquished by (Signature)	Date	Time	Received by (Signature)	Relinquished by (Signature)	Date	Time	Received by (Signature)
[signature]	6/25/21	13:05	[signature] 6/25/21 13:05	[signature]	6/25/21	17:09	Taylor Davis 6/25/21 1709

Relinquished by (Signature)	Date	Time	Received by (Signature)	Relinquished by (Signature)	Date	Time	Received by (Signature)

ESS LABORATORY

185 Frances Avenue
Cranston, RI 02921
Phone: 401-461-7181
Fax: 401-461-4486
www.esslaboratory.com

CHAIN OF CUSTODY

ESS Lab # 21F0999 | Page 2 of 2

Turn Time: ☐ >5 ☑ 5 ☐ 4 ☐ 3 ☐ 2 ☐ 1 ☐ Same Day

Regulatory State: | Criteria:

Is this project for any of the following?: ☐ CT RCP ☐ MA MCP ☐ RGP ☐ Permit ☐ 401 WQ

ELECTRONIC DELIVERABLES (Final Reports are PDF)

☐ Limit Checker ☐ State Forms ☐ EQuIS
☐ Excel ☐ Hard Copy ☐ Enviro Data
☐ CLP-Like Package ☐ Other (Specify) →

CLIENT INFORMATION

Client:	Horsley Witten Group
Address:	90 Route 6A
	Sandwich, MA 02563
Phone:	(508) 833-6600
Email Distribution List:	

PROJECT INFORMATION

Project Name:	Ipswich Mills Dam
Project Location:	Ipswich, MA
Project Number:	16041E
Project Manager:	Neal Price
Bill to:	
PO#:	
Quote#:	

Client acknowledges that sampling is compliant with all EPA / State regulatory programs

REQUESTED ANALYSES

ESS Lab ID	Collection Date	Collection Time	Sample Type	Sample Matrix	Sample ID	Grain Size	Total Number of Bottles
9	6/24/21	9:02 AM	Grab	Soil	SS9	X	
10	6/24/21	9:12 AM	↓	↓	SS10	X	
11	6/24/21	9:25 AM	↓	↓	SS11	X	
12	6/24/21	9:35 AM	↓	↓	SS12	X	
13	6/24/21	9:45 AM	↓	↓	SS13	X	
14	6/24/21	10:00 AM	↓	↓	SS14	X	
15	6/24/21	10:10 AM	↓	↓	SS15	X	
					hdm 6/28/21		

Container Type: AC-Air Cassette AG-Amber Glass B-BOD Bottle C-Cubitainer J-Jar O-Other P-Poly S-Sterile V-Vial

Container Volume: 1-100 mL 2-2.5 gal 3-250 mL 4-300 mL 5-500 mL 6-1L 7-VOA 8-2 oz 9-4 oz 10-8 oz 11-Other*

Preservation Code: 1-Non Preserved 2-HCl 3-H2SO4 4-HNO3 5-NaOH 6-Methanol 7-Na2S2O3 8-ZnAce, NaOH 9-NH4Cl 10-DI H2O 11-Other*

Sampled by : Jonas Procton

Chain needs to be filled out neatly and completely for on time delivery.

Laboratory Use Only
Cooler Temperature (°C): 2.7
/ Ce

Comments: * Please specify "Other" preservative and containers types in this space

All samples submitted are subject to ESS Laboratory's payment terms and conditions.

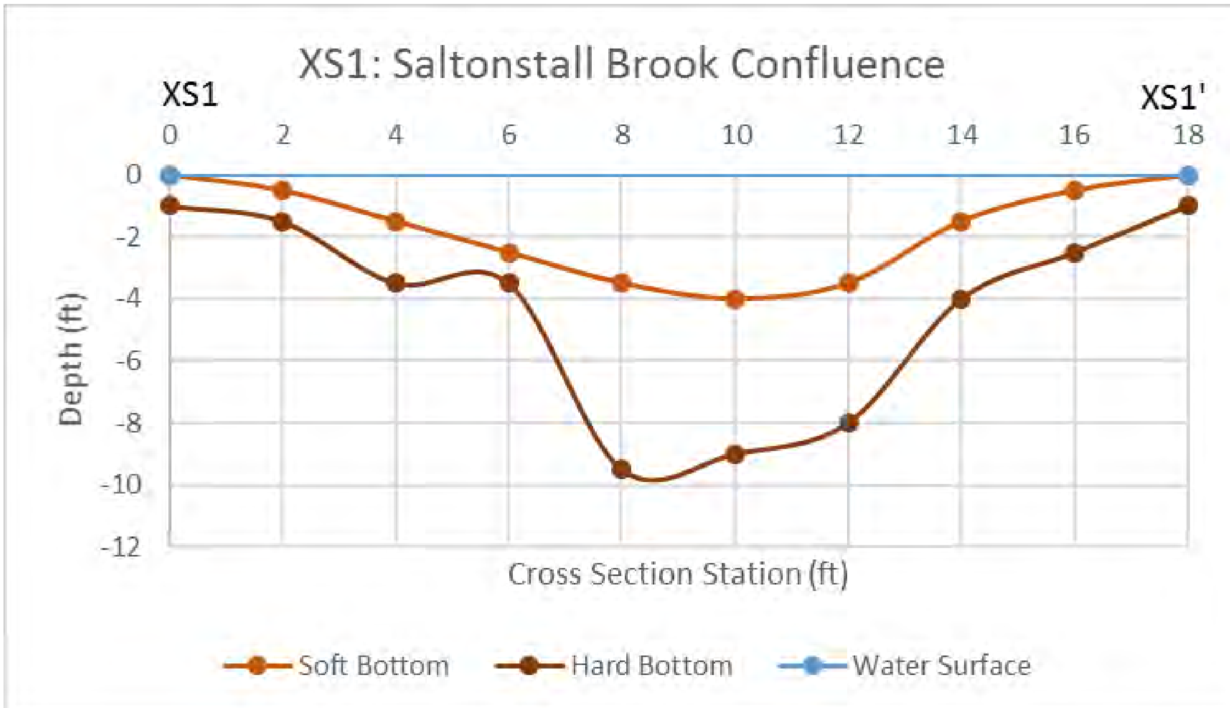
Dissolved Filtration
☐ Lab Filter

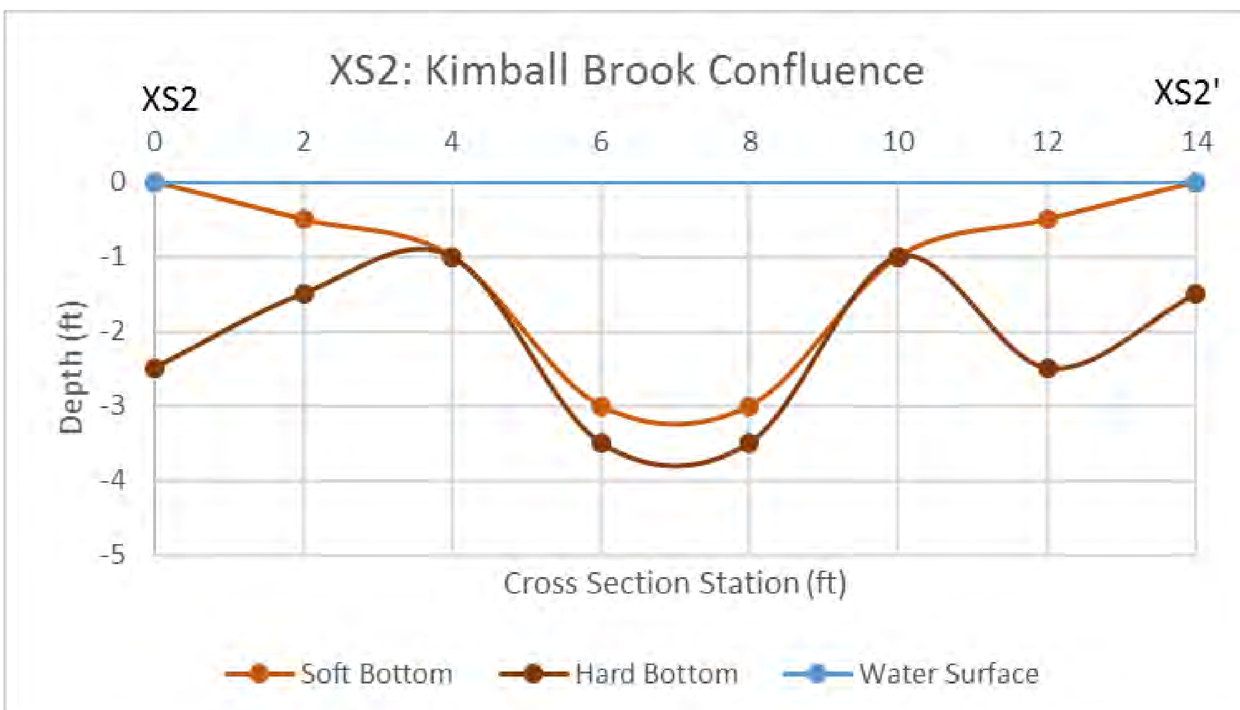
Relinquished by (Signature)	Date	Time	Received by (Signature)	Relinquished by (Signature)	Date	Time	Received by (Signature)
[signature]	6/25/21	13:05	[signature] 6/25/21 13:05	[signature]	6/25/21	17:09	Taylor Davis 6/25/21 1709

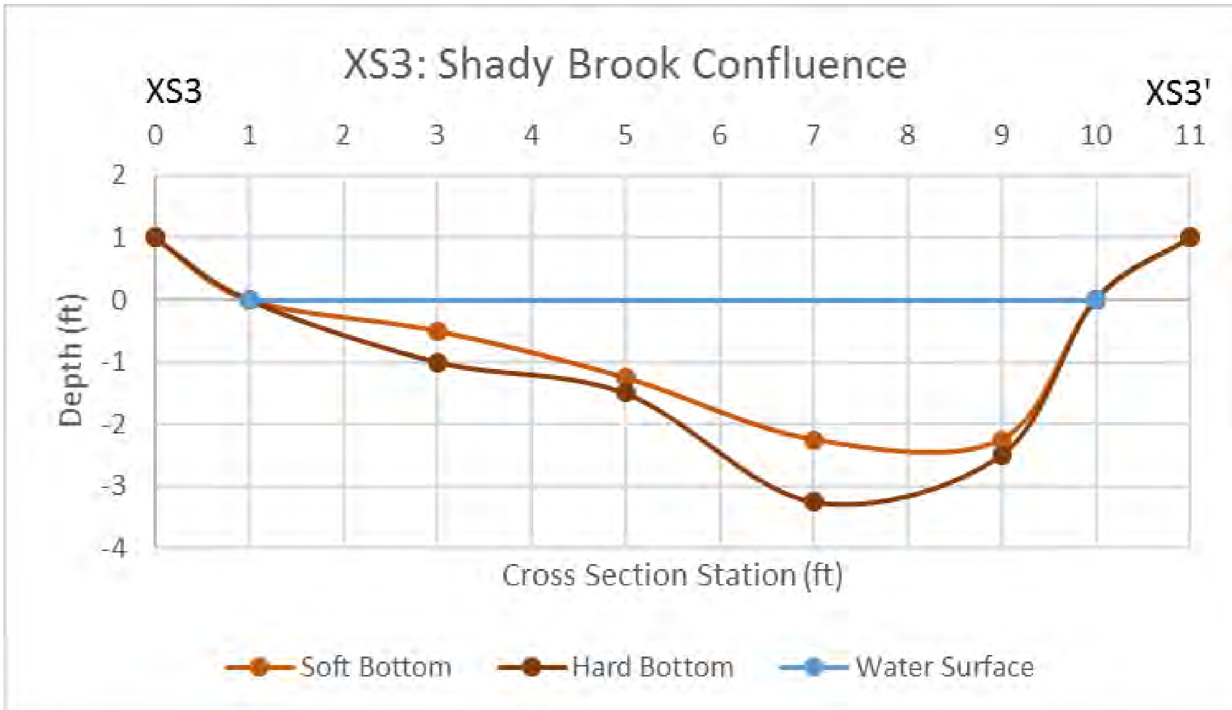
Relinquished by (Signature)	Date	Time	Received by (Signature)	Relinquished by (Signature)	Date	Time	Received by (Signature)

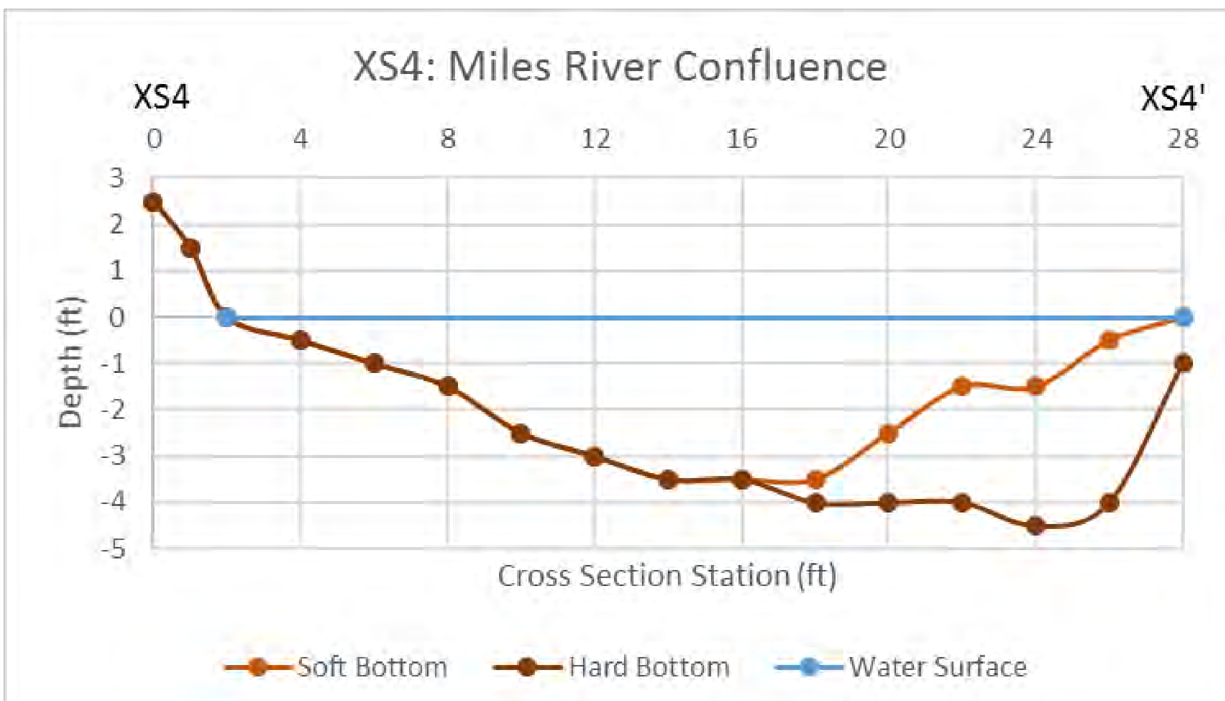
ATTACHMENT B –
2021 Soft Sediment Thickness Cross-Sections

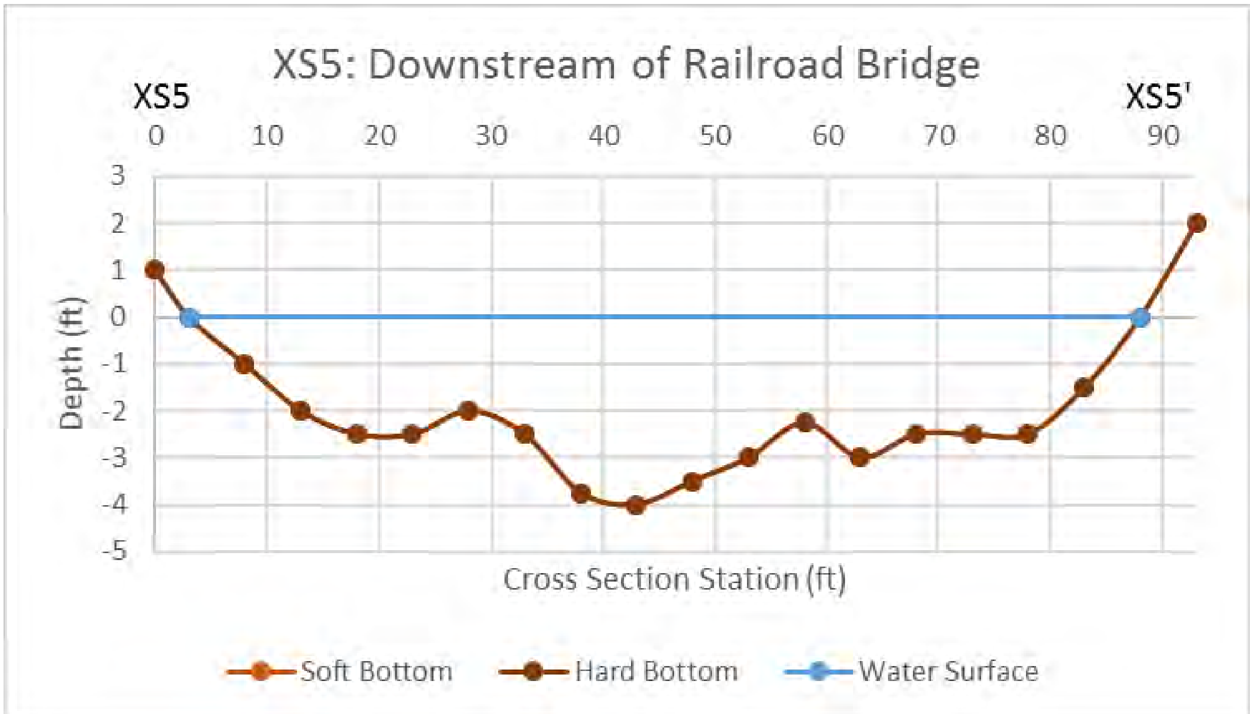
Attachment B. Sediment Probing Cross Sections






XS3: Shady Brook Confluence
XS3
XS3'
0 1 2 3 4 5 6 7 8 9 10 11
2 1 0 -1 -2 -3 -4
Depth (ft)
Cross Section Station (ft)
Soft Bottom
Hard Bottom
Water Surface


XS4: Miles River Confluence
XS4
XS4'
0 4 8 12 16 20 24 28
3 2 1 0 -1 -2 -3 -4 -5
Depth (ft)
Cross Section Station (ft)
Soft Bottom
Hard Bottom
Water Surface



XS5: Downstream of Railroad Bridge
XS5
XS5'
0
10
20
30
40
50
60
70
80
90
3
2
1
0
-1
-2
-3
-4
-5
Depth (ft)
Cross Section Station (ft)
Soft Bottom
Hard Bottom
Water Surface

ATTACHMENT C –
2021 SGH EBSCO Structural Assessment Report with 2021 RSI Geophysical Investigation

7 July 2021
(Revised 27 July 2021)

Mr. Neal Price
Associate Principal / Senior Hydrogeologist
Horsley Witten Group
90 Route 6A
Sandwich, MA 02563

Project 160630.02 – Ipswich Mills Dam Removal Feasibility Study: Evaluation of Potential Impacts on the EBSCO Facility Building Foundations

Additional Phase: Limited Investigation – Targeted Geophysical Study and Visual Observations, Ipswich Mills Dam, Ipswich, MA

Dear Mr. Price:

This letter report summarizes our observations, findings, and conclusions regarding the potential impact(s) of the proposed removal of the Ipswich Mills Dam and the subsequent lowering of the water table on the adjacent EBSCO Facility Building. The current study supplements the findings from our prior two investigations as documented in our letter to you dated 29 June 2018 and revised on 6 July 2018 (SGH July 2018 Letter) and report to you dated 17 February 2017 and revised on 20 February 2018 (SGH February 2018 Report).

If additional information becomes available, we reserve the right to supplement or modify the material presented herein.

1. INTRODUCTION

All elevations in this report are in feet referenced to the North American Vertical Datum of 1988 (ft NAVD 88) unless otherwise noted.

1.1 Background

In 2018, Simpson Gumpertz & Heger Inc. (SGH), under subcontract to the Horsley Witten Group (HW), completed an investigation to evaluate the potential impacts of the proposed dam removal on the EBSCO Facility located adjacent to the Ipswich Mills Dam (refer to the SGH February 2018 Report included in Appendix A). In the summer of 2018, SGH performed a supplemental limited investigation, including soil test borings, to further evaluate potential risks due to the presence of compressible soils (refer to the SGH July 2018 Letter included in Appendix A).

The February 2018 study completed by SGH was limited to test pit investigations adjacent to the EBSCO Facility's riverfront foundation wall (Buildings No. 9 and 10A). The July 2018 supplemental study was limited to soil test borings performed on the exterior of the building to minimize disruption to EBSCO. SGH concluded that the exterior walls and interior columns of Building No. 9, Building No. 10, Building No. 10B, and the northern portion of Building No. 11 are likely founded on shallow spread footings bearing on the relatively shallow Glacial Till stratum or rock. SGH did not observe the foundations supporting interior walls or columns of Buildings No. 9 and 10A (constructed in 1908 and 1912 respectively) or the other buildings on the EBSCO campus (Building Nos. 10, 11, and 11A constructed in 1901, 1918, and 1946 respectively). The depth and thickness of the compressible soils observed at the southeast corner of the EBSCO Facility are such that timber piles may have been driven through the soft compressible soils to bear on the Glacial Till stratum below, to support the building structure in these areas. At this time, it is uncertain as to what extent, if any, compressible soils underlie the EBSCO Facility. Where organics are present, which is likely near the river, lowered groundwater levels could result in settlement of pavement, slabs-on-grade, structures on spread footings, or buried utilities supported above the soft compressible soils.

SGH recommended possible subsequent steps for assessing potential settlement at the EBSCO Facility due to the proposed dam removal and potential lowering of groundwater levels:

- Conduct a subsurface investigation consisting of test pits and soil test borings performed within the EBSCO Facility, focused on Buildings 10A and 11A where the foundation construction is unknown and compressible soils are potentially present. This portion of the EBSCO Facility presents the highest risk of settlement due to drawdown of groundwater levels.

- Develop and implement a precision movement monitoring program to monitor for the potential movement of structures during dam removal construction. Install the instrumentation prior to the start of construction, and also establish acceptable settlement limits with approval from EBSCO.

Subsequent to the February 2018 and July 2018 SGH studies, a geophysical study was performed by Hager Geoscience, Inc. (HGI) in 2020 (2020 HGI Geophysical Report) with the goal of establishing the depth to sound bearing material (glacial till or rock) and determining the potential presence of timber piles. HGI performed Ground Penetration Radar (GPR) and seismic surveys in the southeastern quadrant of the facility and concluded that closely spaced timber piles are present, with the tops of timber piles ranging between El. 7.15 ft and El. 11.17 ft.

You requested that SGH provide engineering consulting services to build on the recently completed geophysical survey studies and render further opinion on anticipated structural impacts to the EBSCO Facility and property from a potential future dam removal scenario. You requested that the study include an updated evaluation and assessment of potential new structural mitigation strategies for the EBSCO Facility, to help the Project Team make an informed decision regarding advancement of the dam removal project. We understand that HW is preparing a report under separate cover to compile water level data collected in 2020 and provide an updated interpretation of the implications of those water level data on potential dam removal.

The Massachusetts Division of Ecological Restoration (MA DER) issued a Request for Proposal (RFP) on 11 March 2021 to perform a hydrologic and structural assessment. MA DER authorized HWG to retain SGH to perform the work.

1.2 Objective

The objective of this additional phase is to supplement prior investigations (February 2018 SGH Report, July 2018 SGH Letter, and 2020 HGI Geophysical Report) and assess the potential impacts of dam removal on the EBSCO Facility.

1.3 Scope of Work

Our Scope of Work included the following:

- Perform a site visit inside the EBSCO Facility to measure column locations, develop a first-floor plan sketch, and document our observations of visible exposed framing elements inside Buildings 10 and 10A.

- Retain a geophysical subconsultant to perform a GPR survey and impact echo survey at three to four locations inside the building to determine slab-on-grade thickness, identify areas of thicker concrete (potential pile caps), and identify locations of pile-supported foundations, if any exist.

- Prepare this letter report summarizing our findings, conclusions, and recommendations.

2. DOCUMENT REVIEW

2.1 2020 HGI Geophysical Report

We reviewed the report titled Final Report of Geophysical Investigation Ipswich Mills Dam Removal Ipswich, Massachusetts, dated 13 August 2020, prepared by HGI for HW. We reviewed sections of the report relevant to the interior and exterior of the EBSCO Facility, which include Appendix III titled EBSCO Building Pile Survey and portions of Appendix II titled On-shore and Near-shore Overburden Stratigraphy. Key extracted sections of the 2020 HGI Geophysical Report are included in Appendix B. The following summarizes our review of this report:

- Elevations in the report are referenced to the NAVD88 vertical datum.

- Plate II-F shows contours of the top of the Glacial Till stratum underlying the EBSCO Facility. The top of Glacial Till within the footprint of the EBSCO Facility ranges between approximately El. -2 ft and El. +8 ft at Building 10 and between El. -4 ft and El. +2 ft at Building 10A. The top of Glacial Till outside the EBSCO Facility ranges between El. -6 ft and El. -2 ft near the southeast corner of Building 10A, and between El. +4 ft and El. +6 ft near the north side of Building 9.

- The top of the interior ground floor slab is located at El. 11.65 ft. HGI states that the thickness of the slab is between 3 and 4 ft in Building 10A; however, it is unclear how this thickness is estimated.

- Plate III-B shows a plan view of the extents of the interior study. HGI performed GPR geophysical investigation using a 400 MHz GPR in three areas (Grids 1 – 3) located at the southeast corner of Building 10A. HGI shows pink colored circles that are the “Location of Pile and/or Foundation Structural Component.” HGI notes that “These structural elements are assumed to be wooden piles based on both the characteristics of the GPR response, which are similar to those we have observed on sites where the nature of the piles was confirmed by excavation, and the typical foundation construction habits in use at the time the building was constructed." Plate III-B shows clusters of piles with an apparently random spacing, and many piles are overlapping adjacent piles.

- HGI notes that the interior survey shows that the apparent pile tops (cutoffs) range between El. 7.15 ft and El. 9.15 ft ("2.5 ft to 4.5 ft below ground level").

- HGI notes that piles appear to be "on the order of approximately 1 ft wide" and that piles are clustered together.

- Plate III-C shows an elevation view of the riverfront wall with estimated vertical structural element locations (timber piles) and pile top (cutoff) elevations based on a 350 MHz GPR survey performed on the exterior wall. HGI shows the pile top elevations varying between El. 9.67 ft and El. 11.17 ft and notes these piles are located 3 ft to 6 ft west of the riverfront wall. The river water level at the time of the exterior HGI survey was El. 9.61 ft. HGI reports that the foundation wall is 2.5 ft thick.

2.2 29 June 2021 Horsley Witten Groundwater Monitoring

Based on the data you provided to SGH via email on 29 June 2021, Horsley Witten monitored groundwater elevations at six groundwater observation wells located around the exterior of the EBSCO Facility and the Ipswich River level staff gauge located near the site for the period between 1 September 2020 and 23 June 2021. HW estimated groundwater levels within the EBSCO Facility ranging between El. 8.5 ft and El. 10.0 ft, based on measured groundwater levels outside the EBSCO Facility ranging between El. 7.3 ft and El. 10.3 ft and the measured river levels ranging between El. 8.8 ft and El. 10.1 ft for the period of record. The groundwater levels are generally within 0.25 ft of the river level on any given measurement date, except for two observation wells located on the north and northwest of the building. Groundwater levels varied from 2.2 ft lower to 0.6 ft higher than the river level and 0.6 ft to 1.8 ft higher than the river level at observation wells HW-2020-4 and HW-2020-5 respectively.

3. FIELD INVESTIGATION

We focused our current field investigation primarily on Buildings 10 and 10A located adjacent to the river.

3.1 Visual Observations

On 3 June 2021, Steven Keppel and Mateus Medeiros of SGH visited the EBSCO Facility to meet with representatives from EBSCO and to develop an investigation plan. On 7 and 10 June 2021 SGH returned to the site to make visual observations of exposed framing and slab-on-grade conditions and observe the field work of our geophysical subconsultant, Radar Solutions International, Inc. (RSI). SGH measured column locations and developed a first-floor plan sketch based on a floorplan image provided by you and our observations of visible exposed framing elements inside Buildings 9, 10, and 10A (Fig. 1). We were not provided access inside Buildings 10B, 11, and 11A. All column location references below are based on this sketch prepared by SGH. The following summarizes our visual observations during our site visits:

- The exposed framing visible from the first floor consists of timber floor planking (supporting the second floor) running north to south, supported on exposed timber beams running east to west (Photo 1). The beams are supported by the mass-masonry bearing walls (Photo 2) and interior columns (Photo 3). The beams are bearing on pockets in the riverfront wall (Column Line “A”) and the interior bearing wall (Column Line “D”) that separates the warehouse space (Building 11) from the office spaces (Buildings 10 and 10A). The following summarizes the framing:

    - The floor planking span (i.e., the beam spacing in the north-south direction) is 10 ft and 10.5 ft in Buildings 10 and 10A respectively.
    - The timber beams are 12 in. deep by 8 in. wide, with spans in the east-west direction ranging between 20 ft in Building 10 and 27 ft in Building 10A.
    - In Building 10A columns consist of 10 in. dia. timber (Photo 3). Columns B1 through B9 have concrete pedestals, approximately 21 in. by 21 in., at the base of the column (Photo 4). The exposed height of the pedestals above the slab/carpet finishes varies between approximately 1/2 in. and 1-1/2 in. The columns have a painted steel corbel at the top and appear to be mechanically anchored to each beam that meets at the centerline of the column (Photo 5).
    - Building 10 columns are 9 in. dia. timber, and the surface is painted white (Photo 6). The finishes on Building 10 columns appear to be plaster or some other cementitious material. We observed one exposed steel column that is 8 in. dia. at Column A’-14 (Photo 7), which is located at the approximate interface between Buildings 10 and 10A.

- The ground floor slab-on-grade shows signs of prior settlement within the southeast corner of the EBSCO Facility (Building 10A). We observed the following:

    - The floor slab slopes downward, away (to the east and west) from the columns at Column Line B (Fig. 1) within Building 10A at the southeast corner of the EBSCO Facility. We measured floor slopes with rises ranging between 3/8 in. and 5/8 in. over a run of 12 in. between Column Lines 2 and 9. The most significant slopes are between Column Lines 3 and 5 (Photos 8 and 9).

- The building framing supporting the second floor shows signs of distortion within the southeast corner of the EBSCO Facility (Building 10A). We observed the following:

    - Several beams spanning between Column Lines A and B and Column Lines B and C appear to have rotated, as indicated by a gap between the beams visible above Column Line B (Photos 10, 11, 12, and 13 show beams above Column Line B at Column Lines 4, 5, 6, and 7 respectively).

- The timber beams typically have checking (cracking parallel to the grain), which is common in timber framing elements (Photo 14). The checking appears to be slightly more severe at Column Lines 5 and 6, where we measured checking up to 1/2 in. wide (Photo 15).

- The timber beams are strengthened at Column Lines 4 and 7 (Photo 16). 7 in. deep steel channels with 3 in. wide flanges have been sistered to the beams spanning between Column Lines A and B (two channels at Column Line 7, one channel at Column Line 4).

- There are openings in the ground-floor slab adjacent to the riverfront wall near Column Lines 1, 3, and 10. Within the openings we observed pumps that discharge to PVC piping, and we observed that the floor slab is between 4 in. and 4-1/2 in. thick at these locations. The pumps were not active during our site visit. There was some water in the sump, located approximately 2 ft below the slab.

- The ground-floor slab is covered by carpet finishes. We temporarily removed carpet tiles to expose the concrete floor slab around Column B5 to provide access for RSI to perform impact echo testing (Photo 17). The floor slab has been covered with a patching or leveling compound material, and there were no cracks visible where the floor is sloped. We observed a shallow surface spall and crack in the patch material southwest of Column B5 (Photo 18). This crack may be in line with a buried foundation element; however, the patching material conceals most of area where we would expect cracking. Note that in general, we did not observe construction joints or cracks through the slab that could potentially indicate the locations of foundation elements below the slab.

- We observed a hole in the concrete riverfront wall, apparently from deterioration, near Column Line 3 (Photo 19). We were unable to measure the hole, since it is located just above the water line; however, it appeared to be 2 in. in diameter and at least as deep.

- We observed step cracking, less than 1/16 in. wide, on the interior face of the masonry bearing wall on the south elevation between Column Lines A and B (Photo 20).

3.2 Information from Others

On 3 June 2021, we spoke with Mr. Matt Churchill of EBSCO. Mr. Churchill said he has worked at the EBSCO Facility for nearly fifteen years. Mr. Churchill indicated that the first floors (in Buildings 10 and 10A) have been covered with carpet finishes for many years, and he is not aware of any repairs to the floor slab or framing, cracks, or any other indications of building settlement during the time he has worked at the facility.

3.3 Geophysical Study

SGH retained Radar Solutions International, Inc. (RSI) to perform a nondestructive survey using geophysical methods on the ground-floor slab and portions of foundation elements that are exposed above-grade at the southeast corner of the EBSCO Facility. RSI performed low frequency (500 MHz antenna) and high frequency (1,500 MHz antenna) ground-penetrating radar (GPR) surveys on the existing floor slab and impact echo (IE) and sonic echo/impulse response (SEIR) on exposed concrete surfaces, including portions of the perimeter foundation walls and select areas of the ground slab near Column B5 where we temporarily removed carpet floor finishes. The following summarizes the letter from RSI to SGH titled “GPR, Impact Echo, and Sonic Echo/Impulse Response Surveys For Structural Assessment of the Slab and Foundation Walls EBSCO Property, Ipswich, Massachusetts” dated 28 June 2021 (Appendix C).

- The foundation wall on the south elevation wall (including the southeast corner) is approximately 24 to 28 in. thick at the top (Appendix C Fig.8) and between 17 and 21 ft deep (Appendix C Fig.9). Based on these results, we estimate that the bottom of the wall is located between El. -4.5 ft and El. -8.5 ft.

- The impact echo results around column B5 (Appendix C Fig. 2) show that the slab-on-grade is nominally approximately 5.5 to 6 in. thick. The high frequency GPR shows that the slab is slightly thicker, but is generally consistent with the impact echo results. RSI was unable to detect the slab thickness in several locations due to interference in the impact echo signal, likely from deteriorated concrete or concrete patch material and buried conduit. RSI also interprets the presence of a 6 in. thick layer of granular material underlying the slab-on-grade.

- The impact echo results around column B5 indicate thicker concrete at Column Lines B and 5, approximately 28 to 35 in. thick, which is consistent with a thickened slab or grade beam foundations aligning with the column lines. RSI indicates that the width of the thickened concrete or grade beams is the same as or slightly less than the 21 in. x 21 in. column pedestal visible above the slab. The column pedestal is approximately 24 in. to 34 in. deep, similar in width and depth to the grade beams. It is possible that the grade beams and pedestals are integral to each other.

- RSI was able to detect pile foundations located below the column pedestal, and the depth to the bottom of the pile(s) is 10.7 to 10.8 ft from the top of the pedestal (Appendix C Fig. 5). Based on these results, we estimate that pile tip elevation(s) are located at approximately El. 0.9 ft. RSI is unable to determine the pile material or the quantity/location of piles underlying the column pedestal; all RSI can identify is that the column is supported on pile foundation elements that are in contact with the base of the pedestal at the three locations where measurements were obtained. It is possible that the column pedestal is supported on a concrete pier with the same cross-sectional area as the pedestal (the joint between the top of the pier and the base of the pedestal would prevent identifying it as a zone of thick concrete). It is also possible that the column pedestal is supported on three timber piles, one at each location where a pile tip depth measurement was obtained.

- RSI prepared depth slice images, which are plan views of their results plotted at various depths for both the high frequency GPR survey ranging from approximately 1 in. to 17 in. below the top of the slab (Appendix C Figs. 4A to 4M) and the low frequency GPR survey ranging from approximately 0.3 ft to 5.5 ft below the top of the slab (Appendix C Figs. 7A to 7M). RSI performed a visual inspection of the GPR data and summarized the results in Appendix C Fig. 6. The GPR results do not indicate the presence of timber piles below the slab; rather, RSI states that the GPR results indicate horizontally oriented targets at depths of 15 to 24 in. below the slab and interprets these targets to potentially be buried conduits. RSI was able to detect several locations with reflections that could indicate voiding or lower-density soils, higher-density materials such as boulders or other structures, or increased moisture.

4. ANALYSIS

4.1 Updated Subsurface Profiles and Wall Section

We updated the subsurface profiles (Figs. 2 and 3) and the Riverfront Foundation Wall section at Building 10A (Fig. 4) based on the results of prior investigations by SGH, the 2020 HGI Geophysical Report, and the recent 2021 RSI Geophysical Report. Based on the 2020 HGI Geophysical Report, the top of the Glacial Till stratum underlying Building 10 and Building 10A is generally between El. 2 ft and El. -2 ft (approximately 10 ft to 14 ft below the top of the slab). Outside the building, the top of the Glacial Till stratum is as low as El. –6 ft. Based on the 2021 RSI Geophysical Report, we estimate the following elevations for foundation elements in Building 10A:

- Bottom of column pedestal and grade beam foundations (and likely top of potential timber piles and/or concrete pier foundations): approximately El. 8.8 to El 9.7 ft.

- Bottom of potential timber pile(s) and/or concrete pier foundation below Column B5 (pile tip elevation): El. 1.0 ft +/-.

- Bottom of Riverfront Foundation Wall near the southeast corner: El. -4.5 ft and El. -6.0 ft, apparently embedded about 4 ft into the Glacial Till stratum.

- Bottom of south elevation foundation wall: El. -5.5 ft and El. -8.5 ft, apparently embedded about 3 ft into the Glacial Till stratum.

- The top of the interior floor slab between Column Lines 3 and 4 near the Riverfront Foundation Wall is between approximately El. 11.5 ft and El. 11.65 ft.

4.2 Updated Potential Mitigation Cost Estimate

In our February 2018 Report (Appendix A), we prepared an order-of-magnitude cost estimate for mitigation of timber pile deterioration and organic soil settlement due to lowered groundwater levels, in the event the project team determines that mitigation is required. For the current study, we prepared a cost estimate for portions of the EBSCO Facility that could be impacted by lowered water levels based on the results of the recent geophysical surveys (Fig. 5).

We considered two mitigation options:

- Option 1 – Cut-and-post underpinning of potential existing timber piles supporting column and grade beam foundations.

- Option 2 – Ground improvement below the slab and column/grade beam foundations using compaction grouting to address potential presence of compressible soils underlying the slab and potential existing timber piles supporting the interior column foundations.

A detailed description of these options can be found in Section 5.1.1 of the SGH February 2018 Report. We do not consider replacing the existing slab-on-grade with a structural slab on pile foundations or installation of new pile foundations to support the existing building interior columns to be cost-effective, viable options at this site due to the significant disruption to the building facility's operations, long construction schedule, and high cost that these options entail.

For purposes of our order-of-magnitude cost estimate we assumed the following:

- Potential mitigation measures will be limited to the interior columns and slabs in Buildings 10 and 10A.
- We incorporate the unit cost for mitigation options developed in the SGH February 2018 Report (based on other projects with similar scope), escalated by 5% per year.
- The perimeter foundation walls are sufficiently deep that they are bearing on glacial till or rock and do not require mitigation.
- Cut-and-post underpinning includes excavation of soils to a depth of approximately 8 ft below the slab.
- Average depth of compaction grout is 12 ft and 16 ft for Building 10 and Building 10A respectively based on the anticipated top of the Glacial Till stratum, which is between 2 ft and 14 ft below the slab in Building 10 and 10 ft and 16 ft below the slab in Building 10A.

Table 1: Order-of-Magnitude Engineer's Cost Estimate for Potential Mitigation Due to Lowered Groundwater Levels

Option No.	Description	Building No.	Estimated Area(1.) (sq ft)	Estimated Direct Unit Cost(2.)	Estimated Direct Cost(2.)	Estimated Burdened Cost(3., 4.)
1	Timber Pile Mitigation (cut-and-post underpinning for columns/grade beams)	10	3,600	$828/sq ft	$ 2,980,800.00	**$ 4,650,048.00**
		10A	2,600	$828/sq ft	$ 2,152,800.00	**$ 3,358,368.00**
Total Building 10 and Building 10A:						**$ 8,008,416.00**
2	Slab Settlement – Ground Improvement Approach (Compaction Grouting)	10	11,400	$317/sq ft	$ 3,613,800.00	**$ 5,637,528.00**
		10A	11,200	$356/sq ft	$ 3,987,200.00	**$ 6,220,032.00**
Total Building 10 and Building 10A:						**$11,857,560.00**

Notes:

1. For cut-and-post underpinning, we assumed that the mitigation will be limited to the grade beams, which are approximately 2 ft wide and located below each column line as shown on Fig. 5. For ground improvement, we assumed mitigation is applied to the total area of the slab as shown on Fig. 5.

2. The direct cost is the unburdened subcontractor cost based on our experience on prior projects involving deteriorating timber piles, underpinning structures, and remediating slab settlement.
3. Total burdened cost assumes the following: 10% general conditions, 10% general contractor markup, 10% design fees, and 20% construction contingency.
4. This order-of-magnitude direct cost may vary greatly depending upon project specifics, including, but not limited to, the existing structure and subsurface conditions, access to repair areas, finishes, and any staging required to maintain building occupancy during the repair work. We did not estimate costs for repairing any elements supported on the slab or buried elements such as utilities below the slab.

5. DISCUSSION

The building structural frame consists of masonry bearing walls, timber beams, and interior columns. The second floor consists of timber planking supported on the timber beams which span between the bearing walls and interior columns. The first floor consists of a thin concrete slab-on-grade, which is separate from the structural frame.

Existing Structures – Exterior Bearing Walls

The recent geophysical results indicate that the exterior walls at Building 10A are deeper than expected, extending up to a depth of 21 ft below top of wall, and embedded in the Glacial Till stratum. We expect that exterior walls at other portions of the EBSCO Facility are also bearing on the Glacial Till stratum or other competent stratum based on the relatively shallow depth to the Glacial Till stratum in these areas as discussed in our prior report (Appendix A) and reported by HGI in 2020 (Appendix B).

Existing Structures – Interior Columns

Our geophysical subconsultant, used the impact echo (IE) method to measure the thickness of concrete elements around one column in Building 10A, which indicate either a thickened slab or pile supported foundation elements such as grade beams located below the slab at the column line locations. This grade beam foundation is about 21 in. wide, which is smaller than expected for a pile cap foundation, since it is only wide enough to accommodate one row of piles, compared to typical pile caps for a building of this vintage that are roughly 4 ft wide to accommodate a minimum of two rows of piles, with a spacing of 3 ft on center, to support a column or bearing wall. The IE method has some limitations in that it may not detect additional layers of concrete or stone located at a greater depth such as a stepped pile cap foundation underlying the column pedestal or grade beams, because any air gap between the layers prevents the signals from getting through. Therefore, it is possible there could be a larger pile cap or deep concrete pier underlying the pedestal and grade beams.

RSI was not able to identify locations of piles below the column, but they used IE to detect the length of potential piles below the column. The length of the piles measured by RSI are consistent with the HGI contour elevations for the top of the Glacial Till stratum. We expect that timber piles, if present, would have been driven through any soft soils to a dense bearing stratum like the Glacial Till stratum. Alternatively, less common foundation types such as concrete piers or caissons may have been excavated or drilled through the soft soils to bear on competent soils. RSI is unable to determine the material of the foundation element underlying the column pedestal and bearing on till. Timber piles were typically used during the time of original construction in the early 1900s, but given the depth of the exterior foundation walls, it is possible that the structures underlying the column are not timber but concrete piers instead. If timber piles are present, the likely pile top (cutoff) elevation corresponds with the bottom of the pedestal and grade beams, which is El. 8.8 ft to El. 9.7 ft based on RSI's IE measurements and assuming no pile embedment in the pedestal.

We observed no signs of settlement of the EBSCO Facility interior structural framing, with exception to minor cracking near the southeast corner. The cracking in the beams and gaps between beams above columns could be due to shrinkage and are not necessarily due to settlement. Assuming the reported recent measured groundwater level fluctuations ranging between El. 8.5 ft and El. 10 ft within the EBSCO Facility are representative of the historical groundwater fluctuations at the site, it is possible that the lack of signs of significant settlement of the columns, if supported on timber piles, is due to the tops of the timber piles remaining saturated at the low bound of the groundwater levels. The presence of organic soils around the tops of the timber piles may help maintain pile top conditions sufficiently saturated such as to not deteriorate within the duration of the period of low groundwater levels. It is also possible that periods of low water levels below pile tops have been brief such that fine-grained or organic soils surrounding the timber piles have not sufficiently dried to result in pile deterioration. The lack of column settlement could also indicate that the piles (or piers) are concrete and not susceptible to drying and fungal attack.

Existing Structures – Slab-on-Grade

A slab-on-grade is a concrete floor slab supported on the ground, typically with minimal steel reinforcement, and independent of the building's structural frame. The 2020 HGI Report showed many piles with a random spacing below the slab-on-grade that do not correlate with column locations, with numerous piles that are adjacent to each other or overlapping, which is questionable. In addition, the HGI Report indicates a wide range of potential pile top elevations supporting the slab-on-grade, between El. 7.15 ft and El. 11.17 ft. However, it is unlikely that piles were cutoff approximately 0.5 ft below the slab and well above groundwater levels (assuming groundwater levels during construction are similar to current groundwater levels). The findings of the 2021 RSI Report which show there are no indications of the presence of piles below the slab, with GPR signal reflections at various depths which potentially indicate deep voids within the soil matrix, or changes in soil moisture, appear more credible based on our experience with similar structures.

Based on the results of the RSI geophysical survey, the exterior walls would have required over 20 ft of excavation if ground surface prior to construction was similar to present day (or significant filling to raise the ground level). Given the depth of the foundation walls, it seems possible that the contractor may have excavated any soft soils underlying the building or left the soft soils in place and excavated a trench for the exterior wall construction. We observed sloped areas of the slab inside the EBSCO Facility, which suggests that some settlement of the slab has taken place in the past assuming it was level when installed, approximately less than 1 in. between column lines based on our recent observations of the slab. Settlement of the slab is most apparent at the southeast corner of the EBSCO Facility, along column line B, between Column Lines 1 and 10 and we observed surface patching material below the carpet finishes at the one location where finishes were removed around column "B5." The location of this settlement correlates with the soft soils encountered in soil test borings drilled on the exterior and the 2020 HGI GPR results showing that the depth to the Glacial Till stratum is greatest in this area, up to 16 ft below the slab. The settlement also correlates well with RSI's findings that there are no piles supporting the slab.

EBSCO has not reported any issues with settlement of the slab indicating that there likely has not been any significant settlement over the past fifteen years.

Mitigation Options

If mitigation of deteriorating timber piles is required at the EBSCO Facility, we consider cut-and-post underpinning and ground improvement to be viable options, as discussed in our prior report (Appendix A). Based on our experience on prior projects involving deteriorating timber piles and slab settlement mitigation, we estimate that the total cost for remediation repair, if needed, would be on the order of $8MM to $12MM for cut-and-post underpinning repairs and ground improvement respectively assuming the following: 10% general conditions, 10% general contractor markup, 10% design fees, 20% contingency.

We assumed that cut-and-post underpinning is performed below grade beams and columns only, there are no piles supporting the existing slab. We also assumed that the anticipated water level is El. 6 ft. If anticipated future water levels are lower than approximately El. 6 ft, then compaction grouting is likely to be a more efficient mitigation option than the cut-and-post underpinning option since the top of the Glacial Till stratum is generally between El. -6 ft and El. 2 ft, and complete removal and replacement of the timber piles would be required in some areas. The cut-and-post underpinning would require removal of portions of the existing slab to provide access for excavations to expose the timber piles (if present) below the anticipated low water level. We assumed excavation to a depth of 8 ft or approximately El. 4 ft for the order-of-magnitude cost. We assumed that compaction grouting would be performed at regular intervals adjacent to the grade beams and columns, and below the existing slab, therefore the compaction grouting could also provide remedial support to the slab. Both mitigation repair options would be disruptive to building operations. Compaction grouting will require a specialty grouting contractor with low head room drilling equipment and cut-and-post underpinning will require a large staging area to temporarily store soil from excavations that could be up to 8 ft deep.

It is possible to perform movement monitoring of the EBSCO Facility to check the performance of the structure upon dam removal to try to detect the onset of settlement, and thus use the movement monitoring data as an indicator of the presence of deteriorating timber piles. However, the rate of timber pile deterioration can be highly variable. In our experience, due to the accuracy of conventional survey methods, by the time that movement is detected, building distress, such as cracking, has already developed. Performing a precision structure deformation survey instead of a conventional survey would provide more accurate results, but not necessarily preclude damage to the structure. Combining the precision structure deformation survey with a groundwater monitoring program, although more of a reactive approach, could be a viable option to plan for the potential mitigation options discussed above.

6. CONCLUSIONS

Based on the limited subsurface information gathered to date and the recent nondestructive geophysical investigation, we have the following conclusions to supplement our July 2018 and February 2018 report regarding the potential impacts of the dam removal on the adjacent EBSCO Facility:

- Existing Building Foundations:

- Building 10 and Building 10A:
  - The interior columns in Buildings No. 10 and No. 10A are very likely supported on pile foundations bearing on the Glacial Till stratum. The depth and thickness of the observed compressible soils in this area are such that piles may have been driven, excavated, or drilled through the soft compressible soils to bear on the Glacial Till stratum below to support the building structure in these areas. The thickness of concrete elements measured in Building 10A suggests either a thickened slab or a pile-supported foundation element such as a grade beam supporting the columns. The interior column foundation elements may consist of timber piles or concrete piers; physical exposure of the foundation elements is required for confirmation.
  - It is very likely that the exterior walls are founded on spread footings bearing on competent soils such as the Glacial Till stratum, Clayey Silt stratum, or rock.
- Other Areas (Building 9, Building 10B, Building 11, and Building 11A):
  - It is very likely that the exterior walls of these portions of the EBSCO Facility are founded on spread footings bearing on competent soils such as the Glacial Till stratum, Clayey Silt stratum, or rock.
  - It is very likely that the interior columns of Building No. 9, Building No. 10B, and the northern portion of Building No. 11 are founded on shallow spread footings bearing on the Glacial Till stratum or rock.

- Existing Slab-on-grade:

  - We observed signs of past settlement of the slab-on-grade within the EBSCO Facility, specifically in the southeast corner of Building 10A.
  - The ground floor in Building 10A likely consists of a slab-on-grade bearing on granular base overlying fill or native soils and is not pile supported as previously reported by others. If compressible soils are present within the EBSCO Facility, they are most likely underlying Building 10A at the southeast corner which has the greatest depth to the Glacial Till Stratum.
  - Other areas of the EBSCO Facility may have slab-on-grade construction similar to Building 10A.
  - At this time, it is uncertain to what extent, if any, compressible soils underlie Buildings 10 and 10A of the EBSCO Facility. We did not encounter soft compressible soils in soil test borings located on the west and north elevations of the building away from the river. Nondestructive GPR surveys performed by others in 2020 inside the EBSCO Facility indicate depths to the top of the Glacial Till stratum consistent with our prior soil test borings.

- Effects of Lowering Groundwater:

  - Interior Columns: Pile tops are located at El. 10 ft or lower, and have been exposed by approximately 2 ft. We expect lowering groundwater levels to increase the amount of exposure resulting in deterioration and settlement if foundations are supported on timber piles. It is possible that the piles supporting the interior columns are concrete and not timber and therefore would not be subject to fungal attack from lowered groundwater levels.

- Exterior walls: we anticipate no settlement of exterior walls which are likely bearing on competent soils or rock at elevations below the anticipated post-dam removal water levels.
- Slab-on-Grade and Shallow Structures: Lowered groundwater levels could result in settlement of pavement, slabs-on-grade and any elements supported on the slab, and structures on spread footings or buried utilities supported by soft compressible soils. We previously estimated a potential total settlement of the soft compressible soils of approximately between 0.9 in. and 1.5 in. respectively due to a water level drawdown of between 1 ft and 5 ft, assuming a remaining service life of fifty years for the EBSCO facility, in those areas where compressible soils are present (Appendix A). Settlement of the slab-on-grade would likely not affect the interior columns and exterior bearing walls.

- Subsequent Steps:

    - Given that the interior column foundation material remains unknown, and that settlement mitigation for interior columns is costly and is only required if timber piles are present, the primary subsequent step remains to perform a targeted interior test pit excavation to observe the column foundations and determine if they are supported on timber piles or concrete pier foundations. SGH provided planning level scope and costs for interior subsurface investigation in our report dated 17 February 2017 and revised on 20 February 2018 (refer to Section 6 - Conclusions and the Appendix titled "Recommended Supplemental Foundation Investigation") (Appendix A).
    - If the project team anticipates that the post-dam removal groundwater levels cannot be maintained at or above El. 6 ft, the following approach could be implemented to mitigate potential settlement of the slab-on-grade (if compressible soils are present) and interior columns of the EBSCO Facility (if timber piles are present):
        - Develop and implement a precision movement monitoring program to monitor for the potential movement of structures during dam removal construction. Install the instrumentation prior to the start of construction, and also establish acceptable settlement limits with approval from EBSCO.
        - Reserve funds for settlement mitigation repairs to interior columns and slabs located within Buildings 10 and 10A, including repairing any elements supported on the slab or buried elements such as utilities below the slab. Additional subsurface investigation inside the building would be required to develop a detailed repair design and confirm the presence of timber pile foundations.
        - If the additional subsurface investigation (interior test pits) indicates the building is supported on concrete piers or caissons, the required mitigation of potential building settlement due to lowered groundwater levels is anticipated to be minimal.
        - If the additional subsurface investigation (soil test borings) indicates there are no compressible soils underlying the slab-on-grade, required mitigation

of slab or shallow structures settlement due to lowered groundwater levels is anticipated to be minimal.

Limitations of Current Investigation

The information presented herein is based on the geotechnical and geophysical information collected to date. The boring logs and geotechnical investigation records depict subsurface conditions only at the specific soil sampling locations. Subsurface conditions at other locations may differ from conditions observed at specific sample depths and exploration locations. We are relying on geophysical information collected and interpreted by our subconsultant and others, much of which has not been calibrated with destructive physical testing such as a test pit excavation or cores through the subsurface concrete elements. There is no warranty or guarantee, either expressed or implied, that the conditions indicated by such investigations or records thereof are representative of those existing throughout such areas, or any part thereof, or that unexpected developments may not occur, or that materials other than, or in proportions different from, those indicated may not be encountered.

Sincerely yours,

COMMONWEALTH OF MASSACHUSETTS
GIULIANA ZELADA-TUMIALAN
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REGISTERED PROFESSIONAL ENGINEER
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Steven F. Keppel, P.E
Senior Consulting Engineer
MA License No. 49861

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Encls.

List of Attachments

Photos 1 through 20

Figures
Figure 1 – EBSCO Facility Plan
Figure 2 – N/S Soil Profile – EBSCO Facility
Figure 3 – E/W Soil Profile – EBSCO Facility
Figure 4 – Exterior Wall Section View
Figure 5 – Potential Mitigation Extents

Appendices

Appendix A – Letter to Neal Price of Horsley Witten Group, titled Ipswich Mills Dam Removal Feasibility Study: Evaluation of Potential Impacts on the EBSCO Facility Building Foundations, Supplemental Limited Subsurface Investigation, Ipswich, MA, prepared by SGH, dated 29 June 2018 and revised 6 July 2018.

Appendix B – Excerpts from the 2020 HGI Report titled Final Report of Geophysical Investigation Ipswich Mills Dam Removal Ipswich, Massachusetts dated August 2020 prepared by Hager Geoscience, Inc. for Horsley Witten Group.

Appendix C – 2021 RSI Report: letter from RSI to SGH titled “GPR, Impact Echo, and Sonic Echo/Impulse Response Surveys For Structural Assessment of the Slab and Foundation Walls EBSCO Property, Ipswich, Massachusetts,” dated 28 June 2021.

Photo 1

Exposed timber beams running east to west.

Photo 2

Perimeter Mass masonry bearing wall.

Photo 3

Typical columns in Building 10A.

Photo 4

Pedestal at the base of Column B5.

Photo 5

Painted steel corbel at the top of columns.

Photo 6

Example of a typical column in Building 10.

Photo 7

Column A'-14, located at the approximate interface between Buildings 10 and 10A.

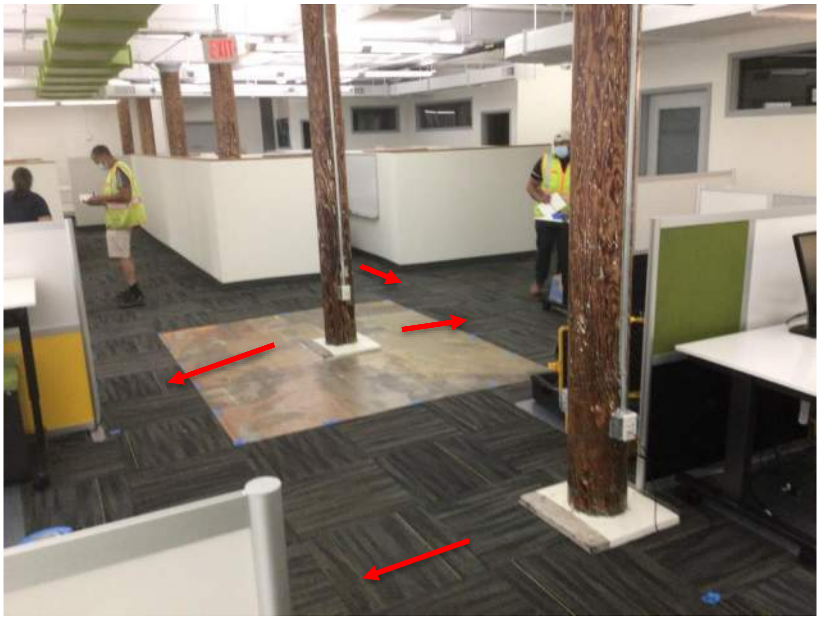


Photo 8

We measured floor slopes with rises generally ranging between 3/8 in. and 5/8 in. over a run of 12 in. between Column Lines 2 and 9. The most significant slopes are between Column Lines 3 and 5, up to 1 in. over 12 in. near Column B4.

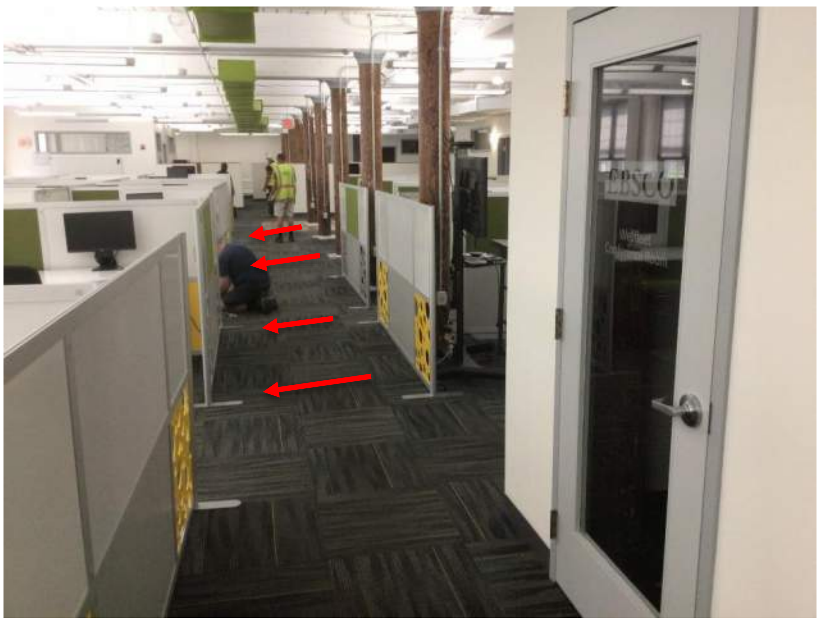


Photo 9

We measured floor slopes with rises generally ranging between 3/8 in. and 5/8 in. over a run of 12 in. between Column Lines 2 and 9. The most significant slopes are between Column Lines 3 and 5, up to 1 in. over 12 in. near Column B4.

Photo 10

Gap between beams above Column B4. View looking north.

Photo 11

Gap between beams above column B5, view looking south.

Photo 12

Gap between beams above Column B6, view looking south.

Photo 13

Gap between beams above Column B7.

Photo 14

Timber beams with checking (cracking parallel to the grain).

Photo 15

Checking on beam at Column Line 6 spanning between Column Lines A to B.

Photo 16

The timber beam at Column Line 7 spanning between Column Lines A and B has been strengthened with steel channels sistered to the beam.

Photo 17

We temporarily removed carpet tiles to expose the concrete floor slab around Column B5 to provide access for RSI to perform impact echo testing. The floor slab has been covered with a patching or leveling compound material, and there were no cracks visible where the floor is sloped.

Photo 18

We removed carpet finishes around Column B5 and observed a shallow surface spall and crack in the patch material southwest of Column B5. This crack may be in line with a buried foundation element.

Photo 19

Hole on the east (riverside) wall, highlighted with arrow and red circle.

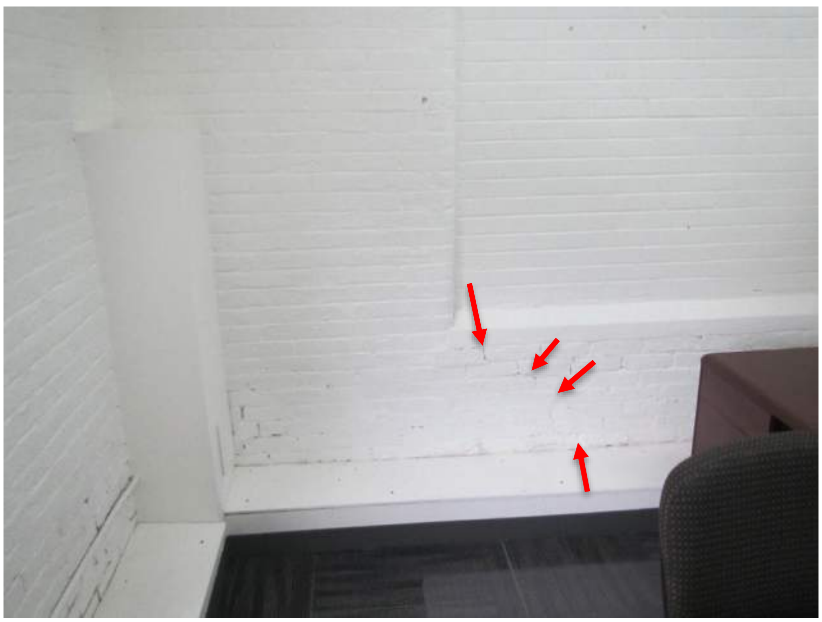
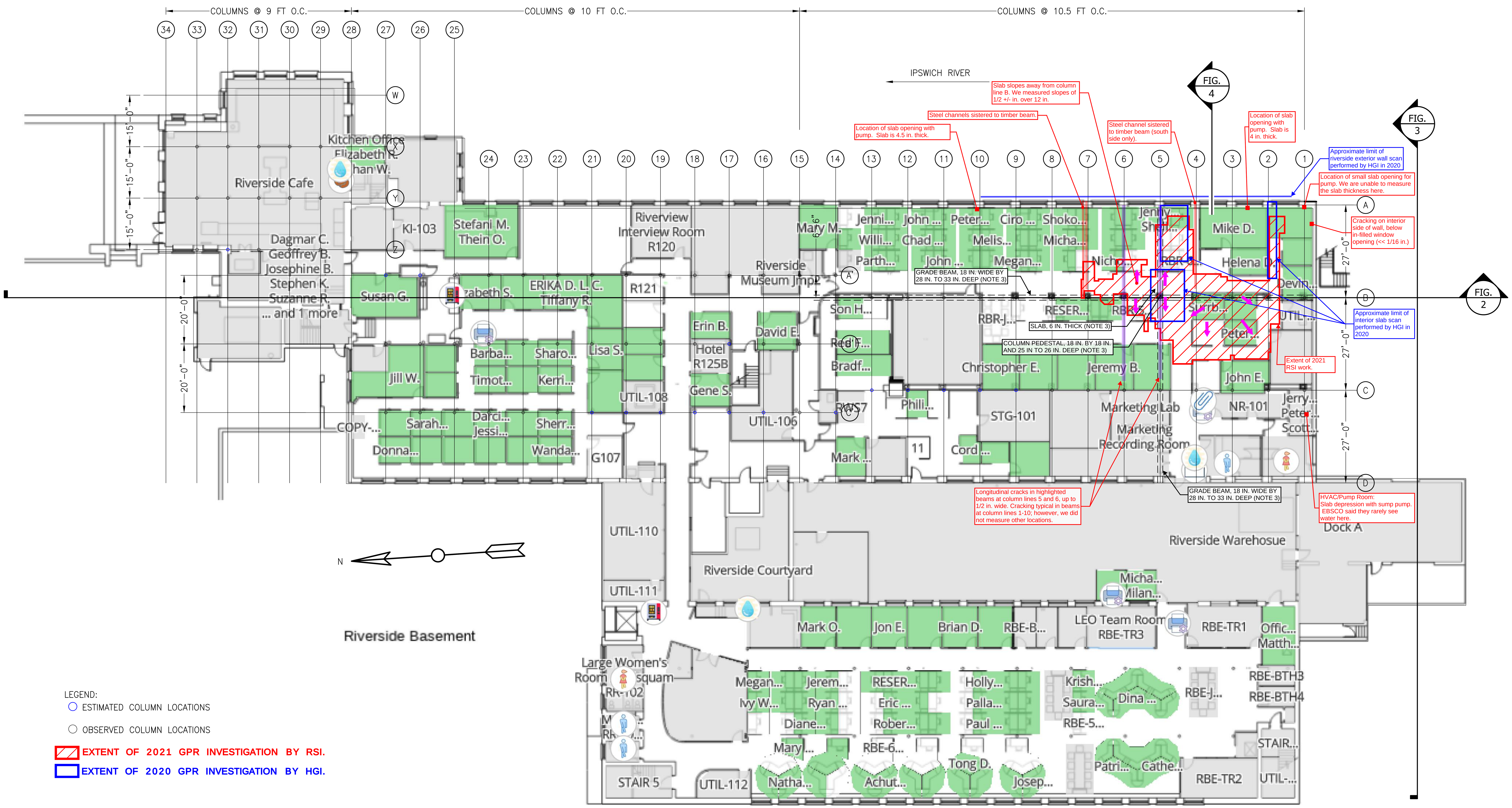


Photo 20

Step cracking, less than 1/16 in. wide, on the interior face of the south elevation masonry bearing wall, between Column Lines A and B.



LEGEND:

ESTIMATED COLUMN LOCATIONS

OBSERVED COLUMN LOCATIONS

EXTENT OF 2021 GPR INVESTIGATION BY RSI.

EXTENT OF 2020 GPR INVESTIGATION BY HGI.

NOTES:

1. BASE PLAN PROVIDED BY HORSLEY WITTEN GROUP VIA EMAIL ON 3 JUNE 2021. WE UNDERSTAND THAT THIS BASE IMAGE FILE IS USED BY EBSCO FOR PLANNING PURPOSES AND IS NOT TO SCALE.
2. COLUMN LINES ARE BASED ON FIELD OBSERVATIONS BY SGH ON 3 JUNE 2021. SGH HAS SCALED THE BASE PLAN TO MATCH LIMITED FIELD MEASUREMENTS. ALL LOCATIONS ARE APPROXIMATE.
3. THE LOCATIONS AND EXTENTS OF BELOW GRADE ELEMENTS (SUCH AS GRADE BEAMS AND PILES ARE ESTIMATED FROM RESULTS OF THE GEOPHYSICAL SURVEY PERFORMED BY RSI IN JUNE 2021. SGH ASSUMED GRADE BEAM LOCATIONS SHOWN ABOVE BASED ON RSI RESULTS AT COLUMN B5.

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Chicago
Houston
Los Angeles
New York
San Francisco
Washington, DC

Consultant

No.	Date	Description	By
0	6/25/21	REPORT FIGURE	MCM

ADDITIONAL PHASE: LIMITED INVESTIGATION – TARGETED GEOPHYSICAL STUDY AND VISUAL OBSERVATIONS

FEASIBILITY STUDY AND CONCEPTUAL PLAN FOR IPSWICH MILLS DAM REMOVAL: EVALUATION OF POTENTIAL IMPACTS TO THE EBSCO BUILDINMG, IPSWICH MA

Project

EBSCO FACILITY PLAN

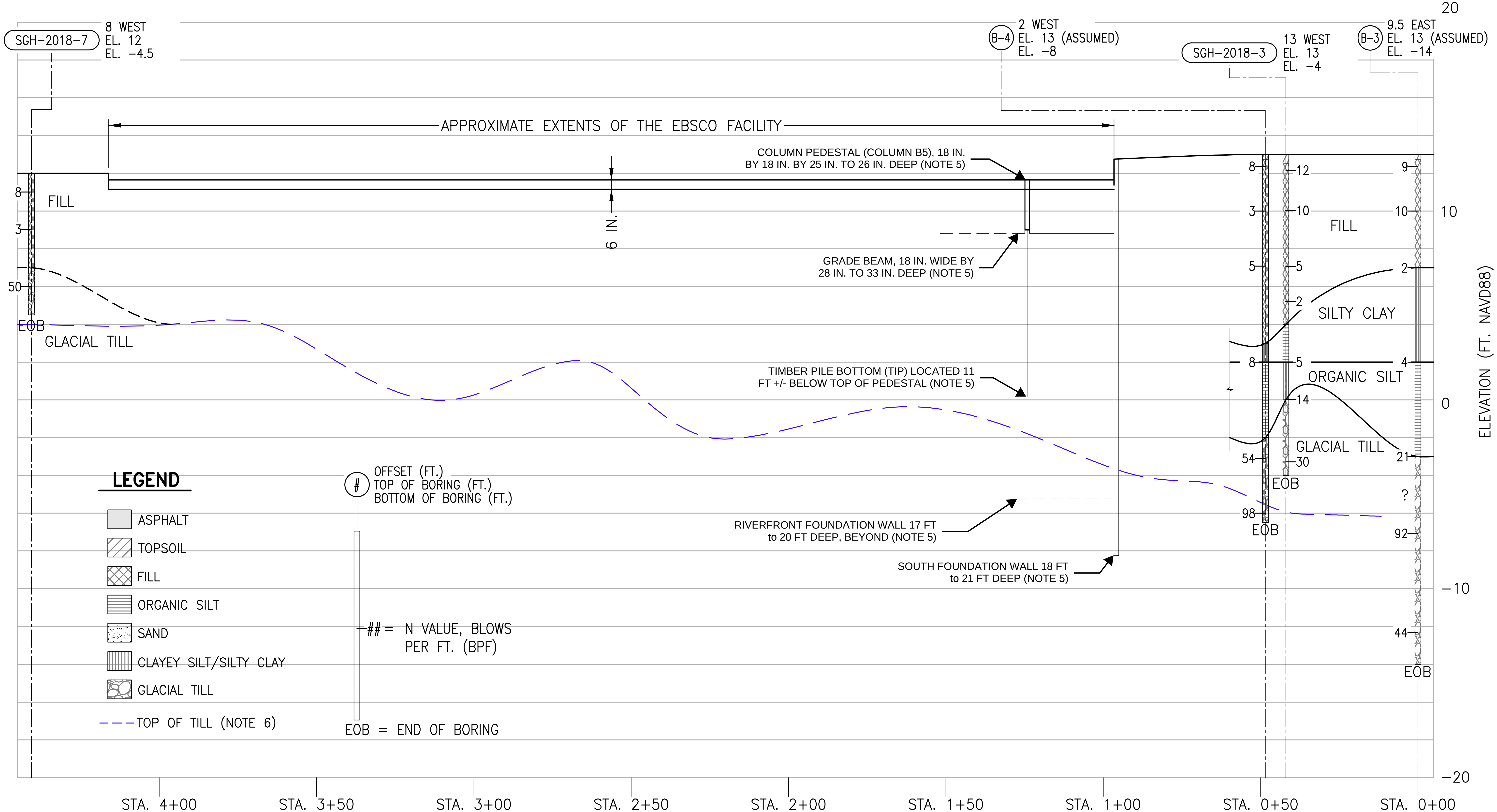
Drawing Title

Project No.	Checked	Date
160630.02	SFK	06/25/2021
Drawn	Approved	Scale
MCM		1/16"=1'-0"

Drawing No.

FIG. 1

Seal



NOTES:

1. ALL ELEVATIONS ARE ESTIMATED AND REFERENCED TO THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88).
2. SOIL STRATA ARE GENERALIZED PROFILES INTERPRETED FROM BORING LOGS PREPARED BY SGH AND OTHERS. REFER TO SOURCE BORING LOGS FOR MORE DETAILED SOIL DESCRIPTIONS.
3. STRATIFICATION LINES REPRESENT APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES; TRANSITIONS MAY BE GRADUAL.
4. STATION NUMBERS FOR SOIL PROFILE ASSUME STATION 0+00 AT THE LOCATION OF B-3 BOREHOLE.
5. DIMENSIONS OF STRUCTURAL ELEMENTS WERE OBTAINED FROM RSI'S GEOPHYSICAL ANALYSIS.
6. TOP OF TILL ESTIMATED FROM THE CONTOUR PLAN PROVIDED IN THE 2020 GEOPHYSICAL STUDY PREPARED BY HAGER GEOSCIENCE, INC. (HGI).

SCALES
HORIZONTAL 1" = 30'
VERTICAL 1" = 5'

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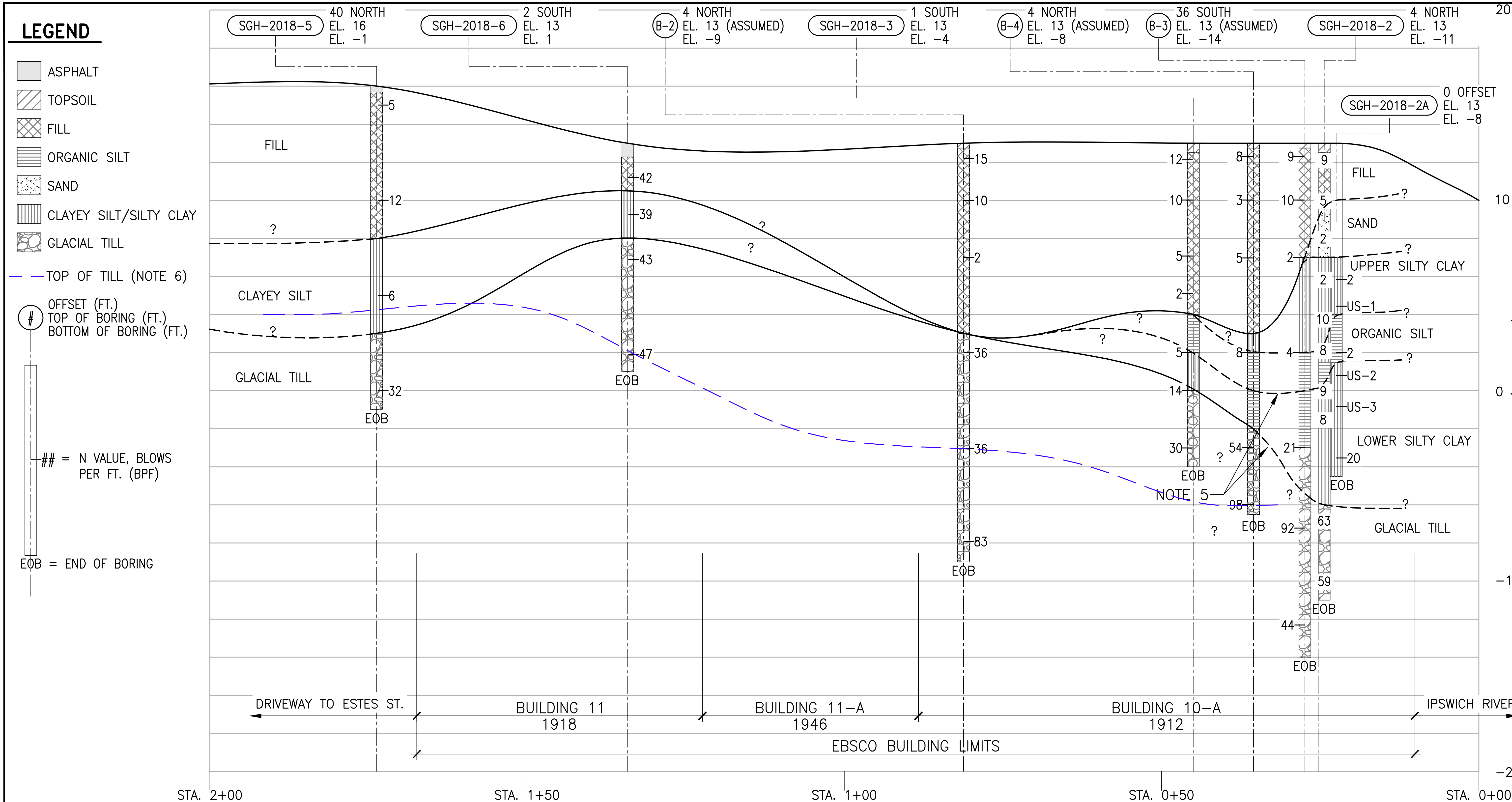
Project: FEASIBILITY STUDY FOR IPSWICH MILLS DAM REMOVAL IPSWICH, MA			Drawing No.: FIG. 2
Title: N/S SOIL PROFILE – EBSCO FACILITY			Scale: AS NOTED
Drawn: MCM	Checked:	Approved:	Project No.: 160630.02
			Date: 06/25/2021



NOTES:

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3. STRATIFICATION LINES REPRESENT APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES; TRANSITIONS MAY BE GRADUAL.
4. STATION NUMBERS FOR SOIL PROFILE ASSUME STATION 0+00 IS LOCATED AT THE EAST ELEVATION OF THE EBSCO FACILITY ADJACENT TO THE RIVER (I.E. THE RIVERFRONT FOUNDATION WALL). SEE FIGURE 2 FOR SECTION ALIGNMENT.
5. ORGANIC SILT AND CLAY STRATUM THICKNESS HAVE BEEN MODIFIED DUE TO THE LACK OF CONTINUOUS SAMPLING THROUGH THE ORGANIC SILT STRATUM PERFORMED BY OTHERS IN 2009, AT B-3 AND B-4.
6. TOP OF TILL ESTIMATED FROM THE CONTOUR PLAN PROVIDED IN THE 2020 GEOPHYSICAL STUDY PREPARED BY HAGER GEOSCIENCE, INC. (HGI).

SCALES
HORIZONTAL 1" = 15'
VERTICAL 1" = 5'

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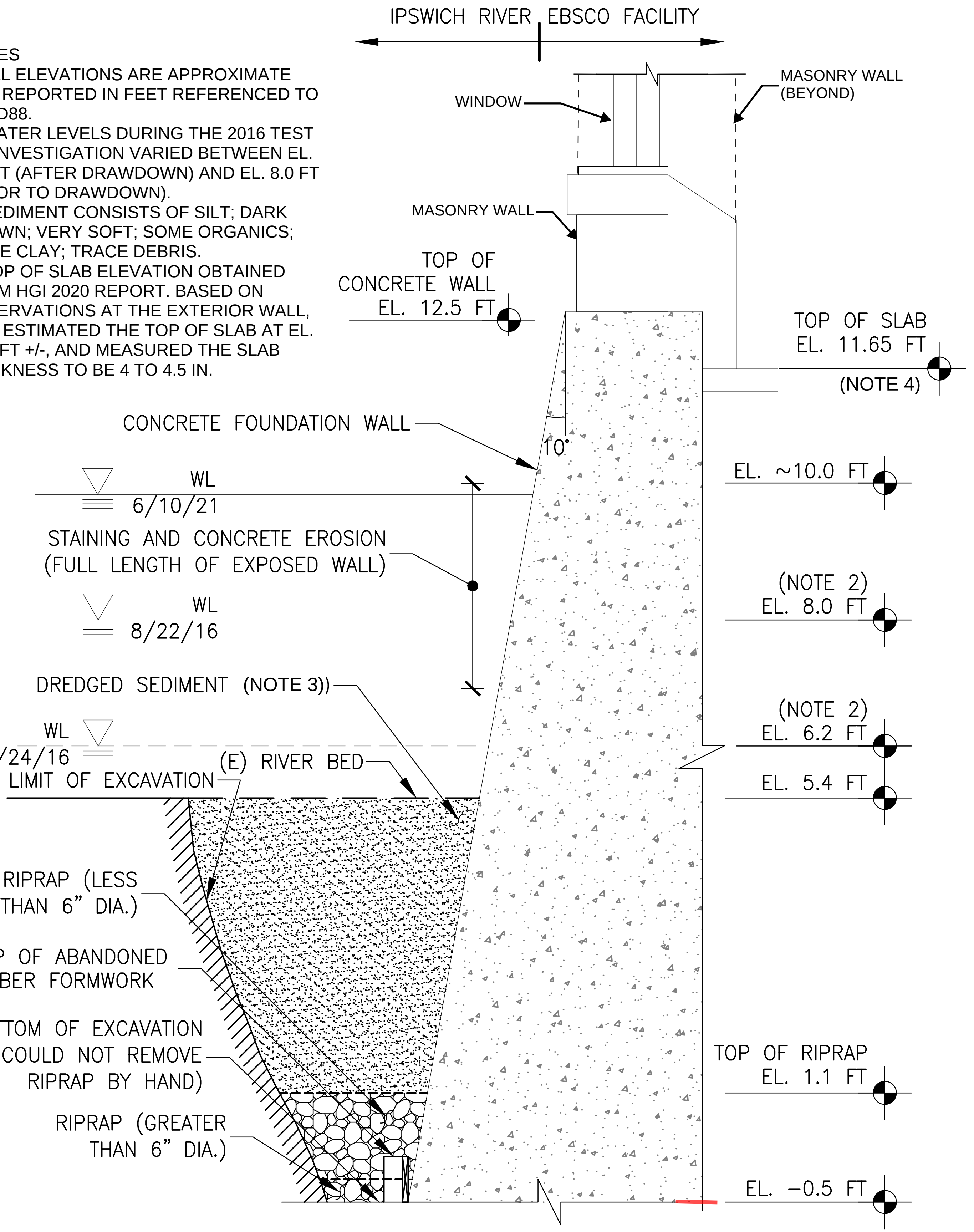
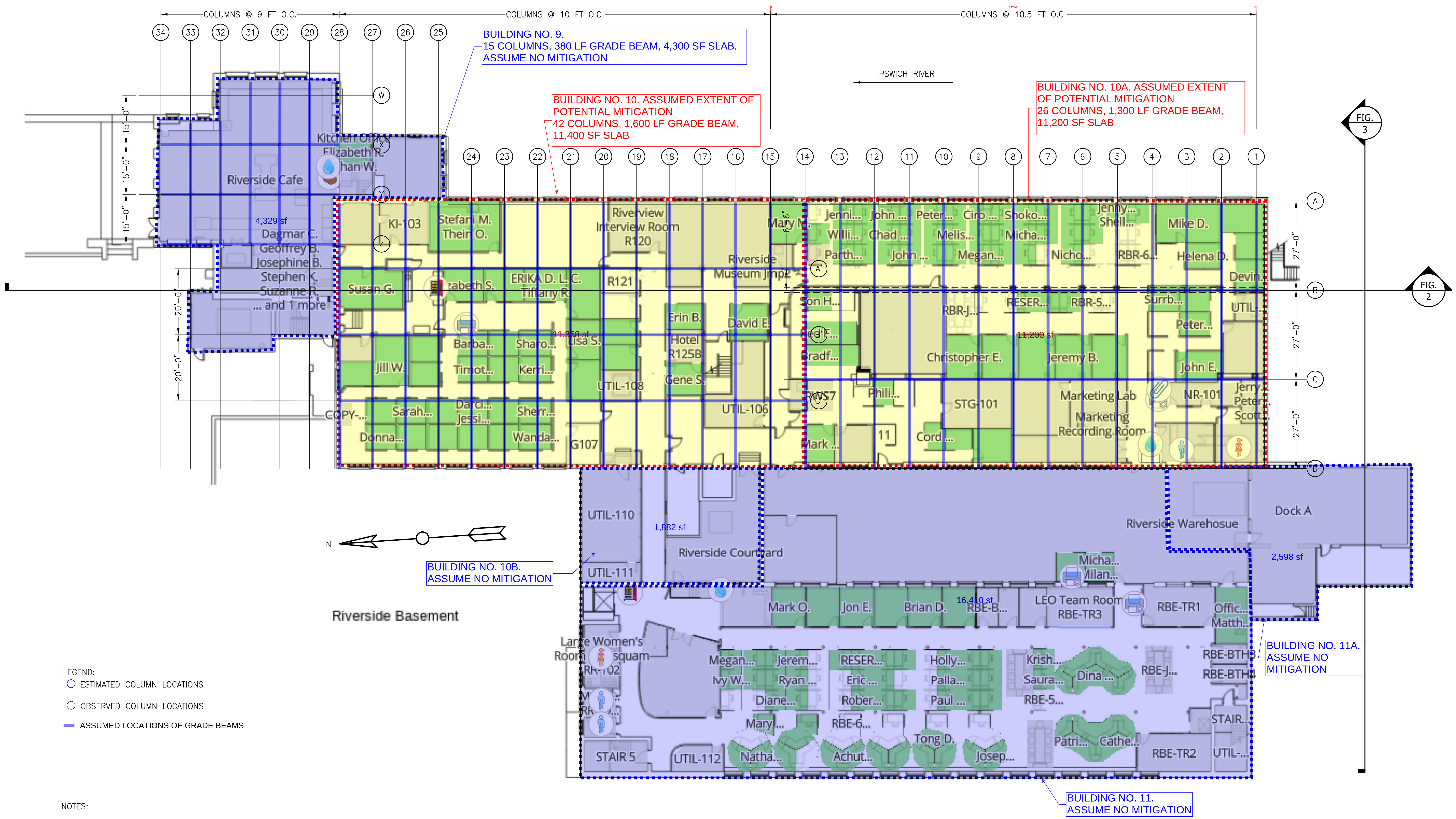
Project:	FEASIBILITY STUDY FOR IPSWICH MILLS DAM REMOVAL IPSWICH, MA
Title:	E/W SOIL PROFILE – EBSCO FACILITY
Drawn:	MCM
Checked:	
Approved:	
Project No.:	160630.02
Drawing No.:	FIG. 3
Scale:	AS NOTED
Date:	06/25/2021

NOTES

1. ALL ELEVATIONS ARE APPROXIMATE AND REPORTED IN FEET REFERENCED TO NAVD88.
2. WATER LEVELS DURING THE 2016 TEST PIT INVESTIGATION VARIED BETWEEN EL. 6.2 FT (AFTER DRAWDOWN) AND EL. 8.0 FT (PRIOR TO DRAWDOWN).
3. SEDIMENT CONSISTS OF SILT; DARK BROWN; VERY SOFT; SOME ORGANICS; SOME CLAY; TRACE DEBRIS.
4. TOP OF SLAB ELEVATION OBTAINED FROM HGI 2020 REPORT. BASED ON OBSERVATIONS AT THE EXTERIOR WALL, SGH ESTIMATED THE TOP OF SLAB AT EL. 11.5 FT +/-, AND MEASURED THE SLAB THICKNESS TO BE 4 TO 4.5 IN.



SIMPSON GUMPERTZ & HEGER Engineering of Structures and Building Enclosures Simpson Gumpertz & Heger Inc. 41 Seyon Street, Building 1, Suite 500 Waltham, Massachusetts 02453 781.907.9000 fax: 781.907.9009 www.sgh.com	Project: FEASIBILITY STUDY FOR IPSWICH MILLS DAM REMOVAL IPSWICH, MA		Drawing No.: FIG. 4
	Title: EXTERIOR WALL SECTION VIEW		Scale: 1/2"=1'−0"
	Drawn: MCM	Checked:	Approved:	Project No.: 160630.02	Date: 06/25/2021



LEGEND:
- ESTIMATED COLUMN LOCATIONS
- OBSERVED COLUMN LOCATIONS
- ASSUMED LOCATIONS OF GRADE BEAMS

NOTES:

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Consultant

No.	Date	Description	By
0	6/25/21	REPORT FIGURE	MCM

ADDITIONAL PHASE: LIMITED INVESTIGATION – TARGETED GEOPHYSICAL STUDY AND VISUAL OBSERVATIONS

FEASIBILITY STUDY AND CONCEPTUAL PLAN FOR IPSWICH MILLS DAM REMOVAL: EVALUATION OF POTENTIAL IMPACTS TO THE EBSCO BUILDINMG, IPSWICH MA

Project

POTENTIAL MITIGATION EXTENTS

Drawing Title

Project No.	Checked	Date
160630.02	SFK	06/25/2021
Drawn	Approved	Scale
MCM		1/16"=1'–0"

Drawing No.

FIG. 5

Seal

2021 SGH APPENDIX A

Letter to Neil Price of Horsley Witten Group titled, "Ipswich Mills Dam Removal Feasibility Study: Evaluation of Potential Impacts on the EBSCO Facility Building Foundations, Supplemental Limited Subsurface Investigation, Ipswich, MA," prepared by SGH, dated 29 June 2018 and revised 6 July 2018.

29 June 2018
(Revised 6 July 2018)

Mr. Neal Price
Senior Hydrogeologist / Senior Project Manager
Horsley Witten Group
90 Route 6A
Sandwich, MA 02563

Project 160630.01 – Ipswich Mills Dam Removal Feasibility Study: Evaluation of Potential Impacts on the EBSCO Facility Building Foundations, Supplemental Limited Subsurface Investigation, Ipswich, MA

Dear Mr. Price:

This letter report summarizes our observations, findings, and conclusions regarding the potential impact(s) of the proposed removal of the Ipswich Mills Dam and the subsequent lowering of the water table on the EBSCO Facility Building. The current study supplements the findings from our initial investigation as documented in our report to you dated 17 February 2017 and revised on 20 February 2018.

If additional information becomes available, we reserve the right to supplement or modify the material presented herein.

1. INTRODUCTION

All elevations in this report are in feet referenced to the North American Vertical Datum of 1988 (ft NAVD 88) unless otherwise noted.

1.1 Background

Simpson Gumpertz & Heger Inc. (SGH) completed an investigation to evaluate the potential impacts of the proposed dam removal on the EBSCO Facility Building located adjacent to the Ipswich Mills Dam; refer to our investigation report titled Feasibility Study and Conceptual Plan for Ipswich Mills Dam Removal: Evaluation of Potential Impacts on the EBSCO Facility Building Foundations, dated 17 February 2018 and revised 20 February 2018, referred to herein as the February 2018 SGH Report (Appendix A). SGH's scope of work was part of a larger feasibility study and concept plan for the dam removal, led by Horsley Witten Group (HWG) and prepared for the Ipswich Mills Dam Removal Feasibility Study Project Team (Project Team). The Project Team includes the Town of Ipswich, the Ipswich River Watershed Association, EBSCO, the NOAA Restoration Center, US Fish and Wildlife Service, the Massachusetts Division of Marine Fisheries, Trout Unlimited, the Massachusetts Division of Ecological Restoration, and others.

The February 2018 SGH Report was limited to two test pit investigations adjacent to the EBSCO Facility Riverfront Foundation Wall at the north end (Building No. 9, constructed in 1908) and the south end (Building No. 10-A, constructed in 1912). SGH concluded that the riverfront wall foundations of Buildings No. 9 and 10-A are bearing on rock and/or are bearing on soils or piled foundations at an elevation lower than the currently estimated low-water level of the Ipswich River at the site after dam removal (El. 3 ft to El. 6 ft). SGH did not observe the foundations supporting interior walls or columns of Buildings No. 9 and 10A or the other buildings on the EBSCO campus (Buildings No. 10, 11, and 11A, constructed in 1901, 1918, and 1946, respectively).

The three borings directed by SGH in August 2016 were located outside of the EBSCO site and did not encounter compressible soils. The 2009 borings performed by others at the south end of Building No. 10A indicate the presence of localized soft compressible soils, including organics, along the riverfront. Where organics are present, which is likely near the river, lowered groundwater levels could result in settlement of pavement, slabs-on-grade, structures on spread footings, or buried utilities supported above the soft compressible soils.

SGH recommended that additional test pits be excavated in the interior and exterior of the EBSCO Facility to obtain more definitive information regarding the presence of timber piles and soft compressible soils within the footprint of the EBSCO Facility. Alternatively, if EBSCO did not provide access to the inside of its facility or access for test pit investigations on the exterior of the facility, SGH recommended that a limited soil test boring investigation be performed around the building exterior to provide some subsurface information for the EBSCO Facility site and allow the project team to further evaluate the potential risks due to compressible soils and timber piles, if any were deemed to be present.

The Massachusetts Division of Ecological Restoration (MA DER) issued a Request for Proposal (RFP) on 12 April 2018 to perform a limited subsurface investigation. MA DER authorized HWG to retain SGH to perform the work.

1.2 Objective

The objective of the supplemental limited subsurface investigation is to provide some subsurface information for the EBSCO Facility site and allow the project team to further evaluate the potential risks due to compressible soils and assess the likelihood of the presence of timber piles based on the depth to an adequate soil bearing stratum. The current limited subsurface investigation on the EBSCO Facility site will supplement the existing February 2018 SGH Report.

1.3 Scope of Work

Our Scope of Work included the following:

- Perform eight soil test borings around the perimeter of the EBSCO Facility.
- Retain a third-party soil testing laboratory to perform testing on relatively undisturbed soft compressible soil samples.
- Prepare this letter report summarizing our findings, conclusions, and recommendations.

2. FIELD INVESTIGATION – SOIL TEST BORINGS

SGH performed the supplemental limited subsurface investigation at the site on 1 and 2 June 2018. The investigation consisted of eight soil test borings (SGH-2018-1, SGH-2018-2, SGH-2018-2A, SGH-2018-3, SGH-2018-4, SGH-2018-5, SGH-2018-6, and SGH-2018-7) located around the perimeter of the EBSCO Facility. Soil test boring locations are shown in Figs. 1 and 2. Soil test borehole drilling was performed by Carr-Dee Corporation (Carr-Dee), of Medford, Massachusetts under SGH supervision. Site access was provided by EBSCO, and SGH coordinated boring locations with facilities personnel from the EBSCO Facility. EBSCO requested that SGH not perform soil test borings located adjacent to the north elevation of the EBSCO Facility due to existing buried utilities (including a buried fiber optic cable), the specific locations of which are currently unknown. EBSCO also requested that we not disturb paver site finishes for the patio area. There was also limited access due to steep-sloped site finishes at portions of the facility on the west elevation.

Steven Keppel and Zachary Boswell from SGH were present during the field work to observe drilling, assist in obtaining samples, and prepare a descriptive log of each test boring. The sampling intervals, soil descriptions, Standard Penetration Test (SPT) blowcounts, and other pertinent field data are summarized in the individual soil boring logs included in Appendix B. The soil test borings were performed on the north, south, and west elevations of the EBSCO Facility (Photos 1, 2, and 3). The east elevation of the EBSCO Facility (referred to as the Riverfront Foundation wall in our previous report) borders the Ipswich River.

Carr-Dee drilled soil test borings with a Mobile soil scout track drill rig or truck-mounted drill rig using the case and wash method with a 4.5 in O.D. casing. One borehole (SGH-2018-3) was drilled using a 2-1/4 in. I.D. hollow stem auger. Soil samples were obtained using 2 in. O.D. split spoon samplers driven using a 140 lb donut hammer falling 30 in. with a rope cathead. Relatively undisturbed soil samples of soft compressible soils were obtained using a thin-walled Shelby tube.

Soil test borings extended into dense Glacial Till or to split spoon refusal, which ranged between El. -11 ft and El. 8.6 ft. (i.e., 7.5 to 24 ft below ground surface, bgs). Split spoon samples were obtained at 5 ft intervals, except in soft compressible soils where continuous SPT sampling and Shelby tubes were obtained. We encountered wood debris in the wash water while drilling through the soft compressible soils at Boring SGH-2018-2. We had poor sample recovery rates at the same depths where we observed wood in the wash. We drilled Boring SGH-2018-2A, located approximately 4 ft from SGH-2018-2, in order to collect Shelby tube samples in the soft compressible soils. Therefore, we terminated this boring prior to reaching the Glacial Till stratum or refusal. All samples were secured, sealed, and transported to the SGH office at the end of the soil test boring program.

We attempted to drill a soil test boring through an existing concrete pad adjacent to Building No. 10-A. After two attempts we abandoned this location after reaching refusal at a depth of approximately 11 in. on steel reinforcement placed both ways within the slab. We moved the drill rig just outside the slab and completed Soil Test Boring SGH-2018-3.

We estimated elevations based on our measurements for the top of the Riverfront Foundation Wall in the February 2018 SGH Report.

3. ANALYSIS

3.1 Subsurface Conditions

Ground surface conditions consist of asphalt pavement or topsoil. The asphalt pavement is 4 in. to 9 in. thick and was encountered at Soil Test Borings SGH-2018-1, SGH-2018-5, and SGH-2018-6. The topsoil consists of 3 to 6 in. of a brown, dry, sandy silty loam. Topsoil was encountered in Soil Test Borings SGH-2018-2, SGH-2018-2A, SGH-2018-3, SGH-2018-4, and SGH-2018-7.

We prepared a subsurface profile, transverse to the Ipswich River, along the south elevation of the EBSCO Facility based on the results of the soil test borings performed by SGH (SGH-2018-1 to SGH-2018-7) and others (B-2 to B-4) (Fig. 3). We summarize the subsurface strata encountered in the following sections.

3.1.1 Subsurface Conditions - Southeast of EBSCO Facility

We summarize the soil strata encountered southeast of the EBSCO Facility (Soil Test Borings B2, B-3, B-4, SGH-2018-2, SGH-2018-2A, and SGH-2018-3) as follows:

- **Stratum 1 – Fill:** This stratum consists of 3 to 10 ft of a loose to medium dense, brown, dry to wet, silty sand to sandy gravel, fine to coarse grained, poorly graded, subangular, with trace wood and trace brick. SPT blow counts ranged from 2 to 15 blows per foot (bpf).

- **Stratum 2a – Sand and Silt:** This stratum consists of 3 ft of very loose brown, sand and silt, fine grained. SPT blow counts were 2 bpf. This stratum was encountered in Soil Test Boring SGH-2018-2 underlying the Fill stratum.

- **Stratum 3 – Upper Silty Clay:** This stratum consists of 1 to 5.5 ft of very soft to stiff, gray to olive gray, moist to wet silty clay. SPT blow counts ranged from 2 to 10 bpf. This stratum was encountered in Soil Test Borings SGH-2018-3, B-3, and B-4 underlying the Fill stratum and in Soil Test Boring SGH-2018-2 and SGH-2018-2A underlying the Sand and Silt stratum. Fine- to coarse-grained silty sand seams were observed within this stratum at Soil Test Borings SGH-2018-2A, B-3, and B-4. We encountered wood debris in the wash water while drilling at Soil Test Boring SGH-2018-2.

- **Stratum 4 – Organic Silt:** This stratum consists of 1.5 to 2.5 ft of very soft to firm, grey to black, wet, organic silt, with trace to some fine sand. SPT blow counts range from 2 to 5 bpf. This stratum was encountered at Soil Test Borings SGH-2018-2, SGH-2018-2A, SGH-2018-3, B-3, and B-4, underlying the Fill or Upper Silty Clay strata. The measured organic content ranges from 10.3% to 11%.

  A 5 ft thick organic silt stratum was identified as a Peat stratum by others in Soil Test Borings B-3 and B-4. However, the soil description in the logs indicates “fine Sand and Silt with some organics (PEAT).” Furthermore, only 12 in. of this organic soil material was sampled; no other sampling was performed within this stratum. In our soil profile (Fig. 3) we classify this stratum as Organic Silt with a thickness of 2.5 ft (instead of 5 ft shown on logs for B-3 and B-4) based on the description of the soils in these boring logs, the lack of continuous sampling by others through this stratum, and our visual observations and laboratory test results at the soil test borings performed in 2018 that are located within close proximity to Soil Test Borings B-3 and B-4.

- **Stratum 5 – Lower Silty Clay:** This stratum consists of 2 to 6 ft of firm to very stiff, gray to olive gray, moist to wet silty clay. SPT blow counts ranged from 5 to 20 bpf. This stratum was encountered in Soil Test Borings SGH-2018-2 (Photo 4), SGH-2018-2A, and SGH-2018-3 underlying the Organic Silt stratum. Fine- to coarse-grained silty sand seams were observed within this stratum at Soil Test Boring SGH-2018-2A.[1] We encountered wood debris in the wash water while drilling at Soil Test Boring SGH-2018-2.

- **Stratum 6 – Glacial Till**: This stratum consists of medium dense to very dense, light reddish brown to olive grey, dry to wet, sand to sandy gravel, fine to coarse, well to poorly graded, subangular, with trace silt. SPT blow counts ranged from 21 bpf to refusal. This stratum was encountered at Soil Test Boring B-2, underlying the Fill stratum; at Soil Test Boring SGH-2018-2, underlying the Lower Silty Clay stratum; and at Soil Test Borings B-3 and B-4, underlying the Organic Silt stratum. Soil test borings were terminated in the glacial till layer and SGH or others did not determine the stratum thickness at these locations. See Table 1 below for the approximate elevation of the top of the Glacial Till stratum.

- **Stratum 7 – Rock Ledge:** The elevation of the top of the rock ledge varies at the site. Prior test pit investigations and bathymetric survey results indicate that the rock ledge varies between approximately El. 3.0 ft and El. 7.5 ft near the Ipswich Mills Dam.

3.1.2 Subsurface Conditions North, West, and Southwest of EBSCO Facility

We summarize the strata encountered on the north, west, and southwest elevations of the EBSCO Facility (Soil Test Borings SGH-2018-1, SGH-2018-4, SGH-2018-5, SGH-2018-6, SGH-2018-7, SGH-2016-1, and SGH-2016-2) as follows:

- **Stratum 1 – Fill:** This stratum consists of 2.5 to 8 ft of a loose to very dense, brown, dry to wet, silty sand to sandy gravel, fine to coarse grained, poorly graded, subangular, with trace wood and trace brick. SPT blow counts ranged from 4 to 73 bpf.

- **Stratum 2b – Silty Sand:** This stratum consists of 2 to 4.5 ft of medium dense to very dense, light orange brown, dry, silty sand to gravelly sand, fine to coarse grained, uniform to well-graded, subangular. SPT blow counts ranged from 16 to 99 bpf. This stratum was encountered at Soil Test Borings SGH-2016-2 and SGH-2016-3 underlying the Fill stratum.

- **Stratum 2c – Clayey Silt:** This stratum consists of 2.5 to 5 ft of medium stiff to hard, brown, gray, or olive, clayey silt with trace fine sand. SPT blow counts ranged from 6 to 39 bpf. This stratum was encountered in Soil Test Borings SGH-2018-5 and SGH-2018-6, located at the southwest corner of the EBSCO Facility, underlying the Fill stratum.

[1] The laboratory reports Sample SGH-2018-3 US-3 (depth 13.5 to 15.5) as Grey Varved Soil. We did not observe varved soil during drilling.

- **Stratum 7 – Glacial Till**: This stratum consists of medium dense to very dense, light reddish brown to olive grey, dry to wet, sand to sandy gravel, fine to coarse, well to poorly graded, subangular, with trace silt. SPT blow counts ranged from 28 bpf to refusal.

  This stratum was encountered at Soil Test Borings SGH-2018-1, SGH-2018-4, SGH-2018-7, and SGH-2016-1 underlying the Fill stratum; at Soil Test Borings SGH-2018-5 and SGH-2018-6 underlying the Clayey Silt stratum, and at Soil Test Borings SGH-2016-2 and SGH-2016-3 underlying the Silty Sand Stratum. See Table 1 below for approximate elevations of the top of the Glacial Till stratum.

3.1.3 Summary of Top of Glacial Till Stratum Elevations

Table 1 summarizes the depth and elevation of the top of the Glacial Till stratum:

Table 1: Top of Glacial Till Stratum

	Soil Test Borings											
	SGH-2018-1	**SGH-2018-2**	**SGH-2018-4**	**SGH-2018-5**	**SGH-2018-6**	**SGH-2018-7**	**SGH-2016-1**	**SGH-2016-2**	**SGH-2016-3**	**B-2**	**B-3**	**B-4**
Depth (ft)	6.0	19.0	6.0	13.0	5.0	5.0	4.5	7.5	8.0	10.0	16.0	16.0
Elevation (ft)(1)	11.0	-6.0	10.0	3.0	8.0	7.0	12.5	10.5	6.0	3.0	-3.0	-3.0

(1) Elevations are estimated and referenced to the NAVD88 datum.

3.2 Groundwater Conditions

SGH did not measure groundwater levels during drilling at the soil test borings due to the cased-and-washed drilling method artificially raising the water levels within the borehole. After drilling was completed, SGH observed the groundwater level at approximately El. 8 ft at Soil Test Borings SGH-2018-2, SGH-2018-2A, and SGH-2018-3 located on the south elevation of the EBSCO Facility, which was generally consistent with the level of the Ipswich River during drilling.

We did not observe groundwater in the soil test borings performed in 2016 in Estes Street and Saltonstall Street (SGH-2016-1 and SGH-2016-2), which were terminated at approximately El. 6 ft and El. 7.5 ft respectively.

3.3 Settlement of Compressible Soils

Soft compressible soils are present at the southeast elevation of the EBSCO Facility (Soil Test Borings B3, B4, SGH-2018-2, SGH-2018-2A, and SGH-2018-3). For our settlement analysis, we assumed a soil profile similar to the conditions encountered at Soil Test Borings SGH-2018-2/2A and B-3. We assumed that the soil profile consists of a 3 ft thick Fill stratum overlying a 1 ft thick Sand stratum overlying a 5 ft thick Upper Silty Clay stratum overlying a 2.5 ft thick Organic Silt stratum overlying a 6 ft thick Lower Silty Clay stratum. We estimated soil properties for the Upper Silty Clay, Organic Silt, and Lower Silty Clay strata based on laboratory consolidation tests performed on relatively undisturbed soil samples obtained from Soil Test Boring SGH-2018-2A.

We estimated the potential settlement of compressible soils due to primary consolidation imposed by an increase in effective stress due to lowered groundwater levels, and secondary compression of the organic soils after primary consolidation is complete[2]. For this analysis we assumed that the organic soils are normally consolidated and the additional stress applied to the organic soils due to groundwater drawdown will reinitiate secondary compression.

The laboratory test results show that the Lower Silty Clay stratum is over-consolidated; we estimate an over-consolidation ratio of approximately 4 for this stratum[3]. We assumed that the Upper Silty Clay stratum has similar consolidation parameters as the test sample from the Lower Silty Clay stratum, except that we assumed the Upper Silty Clay is normally consolidated to match the underlying Organic Silt stratum conditions determined from the laboratory test results.

In the February 2018 SGH Report, we estimated an initial groundwater level at approximately El. 6 ft based on our observations of the river staff gauge during our investigation in August 2016 and on groundwater data collected at one observation well. We assumed that the overburden soils under the EBSCO Facility have experienced groundwater levels as low as El. 6 ft; we calculated the range of potential settlement of the clay and organic soils resulting from a 1, 2, and 3 ft drop in groundwater levels. We understand that HWG has not yet completed the hydraulic study of post-dam-removal river levels; however, the revised preliminary estimated lower-bound water level after the proposed dam removal is at approximately El. 1 ft (i.e. a 5 ft drop in groundwater level), and will likely be higher (between El. 3 ft and El. 6 ft). The water level is subject to change pending results from the hydraulic analysis performed by HWG. For our analysis of the potential settlement of compressible soils, we considered a range of potential low-water river elevations between El. 1 ft and El. 5 ft.

Table 2 summarizes the soil consolidation soil parameters used in our analysis:

Table 2 – Clay and Organic Silt Strata Consolidation Parameters(1)

Soil Stratum	Depth to Mid-Layer [ft, bgs]	σ'_{vo}(2) [psf]	σ'_{vf}(2) [psf]					σ'_p(2) [psf]	Initial Void Ratio e_0	C_c(3)	C_r(3)	C_α(3)
			Case 1	Case 2	Case 3	Case 4	Case 5					
Upper Silty Clay	7.75	872	919	919	919	919	919	872	0.897	0.36	0.05	--
Organic Silt	10.25	991	1,054	1,116	1,178	1194	1194	1,000	2.87	1.18	0.17	0.021
Lower Silty Clay	14.5	1,206	1,268	1,331	1,393	1455	1518	5,000	0.897	0.36	0.05	--

(1) Case Nos. 1 through 5, correspond with a groundwater level drawdown of 1 ft through 5 ft respectively (i.e. groundwater level at El. 5 ft, El. 4 ft, El. 3 ft, El. 2 ft, and El. 1 ft respectively).
(2) σ'vo is the estimated existing overburden or in situ vertical effective stress at midlayer (prior to dam removal). σ'vf is the estimated vertical effective stress after dam removal (lowered groundwater level). σ'p is the maximum past pressure experienced by the soil estimated from laboratory test results for SGH-2018-2A US-1 and US-3. The soil profile is estimated from Soil Test Borings SGH-2018-2 and SGH-2018-2A.
(3) Cc is the primary consolidation index, Cr is the recompression index, Cα is the secondary compression index. We did not estimate secondary compression for the Silty Clay.
(3) We assumed the lower half (depths ranging between 6.5 ft and 9 ft) of the Upper Silty Clay stratum contributes to settlement and is normally consolidated. We assumed consolidation parameters for the Upper Silty Clay are similar to the test results for the Lower Silty Clay stratum.

[2] Primary consolidation settlement is load-dependent and occurs when load is transferred to the soil structure and pore water is squeezed out of the soil mass. Secondary compression settlement is time-dependent and occurs after primary consolidation is complete. Secondary compression occurs under constant load and can be significant for organic soils due to creep, and compression and degradation of the organic material.

[3] Normally consolidated and over-consolidated are terms that refer to the current vertical overburden pressure on the soil relative to the maximum vertical overburden pressure the soil has ever experienced. A normally consolidated soil has a current pressure equal or nearly equal to the maximum experienced pressure. An over-consolidated soil has previously experienced a higher pressure, which can be due to natural or man-made causes, compared to the current pressure. The over-consolidation ratio is the ratio of the maximum past pressure relative to the current pressure.

Based on the assumptions listed above, we estimate that the average primary settlement due to lowering groundwater levels by between 1 ft and 5 ft in the post-dam-removal conditions is in the order of between 0.4 in. and 1.0 in., in those areas where compressible soils are present.

We estimate that secondary compression, which is time-dependent strain, of the organic silt stratum will be about 0.5 in. For the purpose of this calculation we assumed a remaining service life of 50 yrs for the EBSCO Facility (we have not performed a service life evaluation of the structure). Including primary consolidation for the 1 ft to 5 ft drawdown scenarios, the total estimated settlement of the soft compressible soils is approximately between 0.9 in. and 1.5 in. after 50 yrs from drawdown.

Table 3 summarizes our estimated settlement of compressible soils resulting from drawdown of groundwater levels:

Table 3 - Estimated Settlement due to Drawdown

Case No. / Drawdown [ft]	Primary Consolidation Settlement(1) [in.]	Secondary Compression(2) [in.]	Total Settlement [in.]
1	0.4	0.5	0.9
2	0.7	0.5	1.2
3	0.9	0.5	1.4
4	1.0	0.5	1.5
5	1.0	0.5	1.5

Notes
(1) See Table 2 for soil properties.
(2) Estimated secondary compression 50 yrs after end of primary consolidation, assuming normally consolidated soils.

4. DISCUSSION

4.1 Supplemented EBSCO Facility Elevations

We estimated ground surface and soil strata elevations based on our reported elevations for the Riverfront Foundation Wall in the February 2018 SGH Report. Table 4 below summarizes elevations pertinent to the EBSCO Facility updated to include additional elevations for the top of Glacial Till based on the recent field investigation. We understand from HWG that the preliminary estimate for the low river level is likely in the range of El. 1 ft to El. 6 ft after dam removal (elevation is subject to change pending the results of the hydrologic and hydraulic analysis).

Table 4 – Water Level, Glacial Till, and EBSCO Foundation Wall Elevations

Description	Elevation [ft, NAVD 88]
Water Levels	
Estimated Low River Level Elevation After Dam Removal (Preliminary Estimate from HWG)	1 to 6
South End of the EBSCO Facility	
Top of Foundation Wall at Building No. 10-A.	12.5
Maximum Elevation of Bottom of Foundation Wall at Building No. 10-A / Bottom of Test Pit No. 2 (TP-2)	–0.5
Approximate Range of top of Organic Silt Stratum at South End of Building No. 10A	1.5 to 4
Approximate Range of top of Glacial Till Stratum at South End of Building No. 10A	-6 to 3
North End of the EBSCO Facility (Closest to Dam)	
Top of Foundation Wall at Building No. 9.	11.4
Apparent Bottom of Foundation Wall at Building No. 9 / Bottom of Test Pit No. 1 (TP-1)	3.2
Dam Crest	8.9
Approximate Top of Glacial Till Stratum at North End of Building No. 9	7
Average Elevation of Rock Ledge at Toe of Dam	2.9
Top of Abandoned Timber Formwork and Abandoned Timber Wall	5.7

4.2 Likelihood of Presence of Timber Pile Foundations at EBSCO Facility

We encountered a shallow depth to the top of the Glacial Till stratum in the soil test borings located on the northern end of Buildings No. 9, No. 10, No. 10B and No. 11 (Soil Test Borings SGH-2018-1, SGH-2018-4, SGH-2018-7, SGH-2016-1, and SGH-2016-2). The depth to the top of the Glacial Till stratum in this area ranged from 4.5 ft to 7.5 ft below ground surface (bgs) (i.e., El. 7 to 12.5). Considering a minimum depth to bottom of footing of approximately 3 ft bgs for exterior foundations and approximately 1 ft below top of slab-on-grade for interior foundations, it is unlikely that timber piles were installed in these areas, as the timber piles would be in the order of 3.5 to 6.5 ft long at most. It is likely that the original foundation construction in this area included over-excavation to place shallow footings bearing directly on the Glacial Till stratum. Based on the limited subsurface information gathered to date, it is very likely that the exterior walls and interior columns of Building No. 9, Building No. 10, Building No. 10B, and the northern portion of Building No. 11 are founded on shallow spread footings bearing on the Glacial Till stratum or rock.

We encountered the top of the Glacial Till stratum on the south end of Building No. 11 (Soil Test Borings SGH-2018-5 and SGH-2018-6) at depths ranging between 5 to 13 ft bgs (El. 3 to 8 ft). At Soil Test Boring SGH-2018-5, where the depth to the top of Glacial Till was 13 ft bgs, the Glacial Till stratum was overlain by 2.5 to 5 ft of medium stiff to hard fine-grained soils (Clayey Silt stratum); that is, the top of the Clayey Silt stratum is at 2.5 to 8 ft bgs in this area. Considering a minimum depth to bottom of footing of approximately 3 ft bgs for exterior foundations and approximately 1 ft below top of slab-on-grade for interior foundations, it is possible but unlikely that timber piles were installed in this area, as the timber piles would be in the order of 4 to 12 ft long at most if bearing on the Glacial Till stratum, and 1.5 to 5 ft long at most if bearing on the Clayey Silt stratum. It is highly likely that shallow soil bearing foundations bearing on the medium-stiff to hard natural fine-grained soils (Clayey Silt statum) were used in this area. Based on the limited subsurface information gathered to date, it is likely that the exterior walls and interior columns of the southern portion of Building No. 11 are founded on shallow spread footings bearing on the Clayey Silt stratum.

The top of the Glacial Till stratum is generally deeper on the south end of Building No. 10A and Building No. 11A (Soil Test Borings B-2, B-3, B-4, SGH-2018-2, SGH-2018-2A, and SGH-2018-3). The depth to the top of the Glacial Till stratum in these borings ranged from 13 to 19 ft bgs (El. -6 ft to 3 ft). The Glacial Till stratum is overlain by 4 to 13 ft of soft to medium-stiff fine-grained soils, including about 2.5 ft of Organic Silt. In our February 2018 SGH Report we noted that it was unlikely that the Riverfront Foundation Wall at Building No. 10A was founded on timber piles given the depth to which excavation was performed to install the wall (greater than 10 ft) and the maximum 5.5 ft depth to top of Glacial Till stratum from the estimated maximum elevation of bottom of Riverfront Foundation Wall (El. -0.5 ft). The construction of the land-side exterior foundation walls and interior column foundations at Buildings No. 10A and No. 11A is not known. The depth and thickness of the observed compressible soils in this area is such that timber piles may have been driven through the soft compressible soils to bear on the Glacial Till stratum below to support the building structure in these areas.

4.3 Settlement of Compressible Soils

Lowered groundwater levels due to potential drawdown after dam removal would increase effective stresses in soft compressible soils and result in settlement of pavement, slabs-on-grade, and structures on spread footings or buried utilities supported above these soft compressible soils. We did not encounter soft compressible soils in soil test borings located on the west and north elevations of the building away from the river; however, we observed soft compressible soils at all soil test borings on the southeast elevation, near the river. Our observations at Soil Test Borings SGH-2018-2, SGH-2018-2A, and SGH-2018-3 indicate that the compressible organic stratum may not be as thick as indicated on prior soil test borings performed by others. Prior soil test borings also indicated the presence of peat. Our visual observations and laboratory results show soils consistent with organic silt. The laboratory test results for organic content measured between 10.3% and 11% percent organic content of the two samples tested, supporting this change in soil description.

In the February 2018 SGH Report we estimated settlement of the soft compressible soils observed by others using assumed soil properties based on ranges of published values for organic soils and local clays. We updated our calculations using revised strata thicknesses, depths, and consolidation parameters determined from our laboratory testing. We tested relatively undisturbed (Shelby tube) samples for one Lower Silty Clay sample and one Organic Silt sample, and we estimated consolidation parameters from the test results. The tested samples are representative of the soil strata, but the in-situ soil properties may vary compared to our limited sample testing results. The laboratory test results show that the Organic Silt is nearly normally consolidated; therefore, a relatively small increase in stress due to drawdown of groundwater levels will result in potentially significant consolidation settlement. Our estimated consolidation coefficients based on laboratory testing of the Lower Silty Clay are consistent with the average values used in our prior analysis, and the estimated consolidation coefficients from laboratory testing of the Organic Silt are consistent with lower-bound published values. Our refined total settlement estimate of the localized soft compressible soils based on laboratory data of the sampled in-situ soils shows a somewhat smaller magnitude of settlement compared to our previous estimate based on only published data. We estimate that the average primary settlement due to lowering groundwater levels by between 1 ft and 5 ft in the post-dam removal conditions is in the order of between 0.4 in. and 1.0 in., in those areas where compressible soils are present. At this time it is uncertain to what extent, if any, compressible soils may or may not underlie the EBSCO Facility. Based on results of the soil test boring located on the building exterior, it is unlikely that organics underlie the northern and western portion of the EBSCO Facility.

We also updated our settlement estimate to include secondary compression, which is long-term time-dependent compressive strain that will occur for many years after primary consolidation is complete. We estimated secondary compression 50 yrs after the end of primary consolidation, assuming normally consolidated organic soils. We estimate secondary compression of the organic silt stratum will be nearly 0.5 in. after 50 yrs. Including primary consolidation for scenarios ranging between 1 ft and 5 ft of drawdown, the total settlement of the soils would range between 0.9 in. and 1.5 in. respectively after 50 yrs.

5. CONCLUSIONS

Based on the recent June 2018 investigation, we have the following conclusions to supplement our February 2018 SGH Report regarding the potential impacts of the dam removal on the adjacent EBSCO Facility:

- Existing Foundations:

    - Based on the limited subsurface information gathered to date, it is very likely that the exterior walls and interior columns of Building No. 9, Building No. 10, Building No. 10B, and the northern portion of Building No. 11 are founded on shallow spread footings bearing on the Glacial Till stratum or rock.
    - Based on the limited subsurface information gathered to date, it is likely that the exterior walls and interior columns of the southern portion of Building No. 11 are founded on shallow spread footings bearing on the Clayey Silt stratum.
    - It is unlikely that the Riverfront Foundation Wall at Building No. 10A was founded on timber piles given the depth to which excavation was performed to install the wall (greater than 10 ft) and the maximum 5.5 ft depth to the top of the Glacial Till stratum from the estimated maximum elevation of the bottom of Riverfront Foundation Wall (El. -0.5 ft). The construction of the land-side exterior foundation walls and interior column foundations at Buildings No. 10A and No. 11A is not known. The depth and thickness of the observed compressible soils in this area are such that timber piles may have been driven through the soft compressible soils to bear on the Glacial Till stratum below to support the building structure in these areas.
    - At this time it is uncertain to what extent, if any, compressible soils underlie the EBSCO Facility. We did not encounter soft compressible soils in soil test borings located on the west and north elevations of the building away from the river.

- Effects of Lowering Groundwater: Lowered groundwater levels could result in settlement of pavement, slabs-on-grade, and structures on spread footings or buried utilities supported by soft compressible soils. We estimate a potential total settlement of the soft compressible soils of approximately between 0.9 in. and 1.5 in. respectively due to a water level drawdown of between 1 ft and 5 ft, assuming a remaining service life of 50 years for the EBSCO facility, in those areas where compressible soils are present.

- Subsequent Steps: If the project team anticipates that the post-dam removal groundwater levels cannot be maintained at or above El. 6 ft, the following approach could be implemented to assess potential settlement of structures bearing on compressible soils:

    - Conduct a subsurface investigation consisting of test pits and soil test borings performed within the EBSCO Facility, focused on Buildings 10A and 11A where the foundation construction is unknown and compressible soils are potentially present. This portion of the EBSCO Facility presents the highest risk of settlement due to drawdown of groundwater levels.
    - Develop and implement a precision movement monitoring program to monitor for the potential movement of structures during dam removal construction. Install the instrumentation prior to the start of construction, and also establish acceptable settlement limits with approval from EBSCO.

Limitations of Current Investigation

The information presented herein is based on the geotechnical information collected to date. The boring logs and geotechnical investigation records depict subsurface conditions only at the specific soil sampling locations. Subsurface conditions at other locations may differ from conditions observed at specific sample depths and exploration locations. There is no warranty or guarantee, either expressed or implied, that the conditions indicated by such investigations or records thereof are representative of those existing throughout such areas, or any part thereof, or that unexpected developments may not occur, or that materials other than, or in proportions different from, those indicated may not be encountered.

Sincerely yours,

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REGISTERED
PROFESSIONAL ENGINEER
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List of Attachments

Photos 1 through 4

Figures
Figure 1 – Subsurface Investigation Plan
Figure 2 – EBSCO Facility and Investigation Location Plan
Figure 3 – Soil Profile - EBSCO Facility

Appendices
Appendix A - Report titled Feasibility Study and Conceptual Plan for Ipswich Mills Dam Removal: Evaluation of Potential Impacts, EBSCO Facility Building, Ipswich, MA" prepared by SGH dated 17 February 2018 and revised 20 February 2018
Appendix B - 2018 Soil Test Boring Logs
Appendix C - 2018 Laboratory Test Results

29 June 2018

Mr. Neal Price
Senior Hydrogeologist / Senior Project Manager
Horsley Witten Group
90 Route 6A
Sandwich, MA 02563

Project 160630.01 – Ipswich Mills Dam Removal Feasibility Study: Evaluation of Potential Impacts on the EBSCO Facility Building Foundations, Supplemental Limited Subsurface Investigation, Ipswich, MA

Dear Mr. Price:

This letter report summarizes our observations, findings, and conclusions regarding the potential impact(s) of the proposed removal of the Ipswich Mills Dam and the subsequent lowering of the water table on the EBSCO Facility Building. The current study supplements the findings from our initial investigation as documented in our report to you dated 17 February 2017 and revised on 20 February 2018.

If additional information becomes available, we reserve the right to supplement or modify the material presented herein.

1. INTRODUCTION

All elevations in this report are in feet referenced to the North American Vertical Datum of 1988 (ft NAVD 88) unless otherwise noted.

1.1 Background

Simpson Gumpertz & Heger Inc. (SGH) completed an investigation to evaluate the potential impacts of the proposed dam removal on the EBSCO Facility Building located adjacent to the Ipswich Mills Dam; refer to our investigation report titled Feasibility Study and Conceptual Plan for Ipswich Mills Dam Removal: Evaluation of Potential Impacts on the EBSCO Facility Building Foundations, dated 17 February 2018 and revised 20 February 2018, referred to herein as the February 2018 SGH Report (Appendix A). SGH's scope of work was part of a larger feasibility study and concept plan for the dam removal, led by Horsley Witten Group (HWG) and prepared for the Ipswich Mills Dam Removal Feasibility Study Project Team (Project Team). The Project Team includes the Town of Ipswich, the Ipswich River Watershed Association, EBSCO, the NOAA Restoration Center, US Fish and Wildlife Service, the Massachusetts Division of Marine Fisheries, Trout Unlimited, the Massachusetts Division of Ecological Restoration, and others.

The February 2018 SGH Report was limited to two test pit investigations adjacent to the EBSCO Facility Riverfront Foundation Wall at the north end (Building No. 9, constructed in 1908) and the south end (Building No. 10-A, constructed in 1912). SGH concluded that the riverfront wall foundations of Buildings No. 9 and 10-A are bearing on rock and/or are bearing on soils or piled foundations at an elevation lower than the currently estimated low-water level of the Ipswich River at the site after dam removal (El. 3 ft to El. 6 ft). SGH did not observe the foundations supporting interior walls or columns of Buildings No. 9 and 10A or the other buildings on the EBSCO campus (Buildings No. 10, 11, and 11A, constructed in 1901, 1918, and 1946, respectively).

The three borings directed by SGH in August 2016 were located outside of the EBSCO site and did not encounter compressible soils. The 2009 borings performed by others at the south end of Building No. 10A indicate the presence of localized soft compressible soils, including organics, along the riverfront. Where organics are present, which is likely near the river, lowered groundwater levels could result in settlement of pavement, slabs-on-grade, structures on spread footings, or buried utilities supported above the soft compressible soils.

SGH recommended that additional test pits be excavated in the interior and exterior of the EBSCO Facility to obtain more definitive information regarding the presence of timber piles and soft compressible soils within the footprint of the EBSCO Facility. Alternatively, if EBSCO did not provide access to the inside of its facility or access for test pit investigations on the exterior of the facility, SGH recommended that a limited soil test boring investigation be performed around the building exterior to provide some subsurface information for the EBSCO Facility site and allow the project team to further evaluate the potential risks due to compressible soils and timber piles, if any were deemed to be present.

The Massachusetts Division of Ecological Restoration (MA DER) issued a Request for Proposal (RFP) on 12 April 2018 to perform a limited subsurface investigation. MA DER authorized HWG to retain SGH to perform the work.

1.2 Objective

The objective of the supplemental limited subsurface investigation is to provide some subsurface information for the EBSCO Facility site and allow the project team to further evaluate the potential risks due to compressible soils and assess the likelihood of the presence of timber piles based on the depth to an adequate soil bearing stratum. The current limited subsurface investigation on the EBSCO Facility site will supplement the existing February 2018 SGH Report.

1.3 Scope of Work

Our Scope of Work included the following:

- Perform eight soil test borings around the perimeter of the EBSCO Facility.
- Retain a third-party soil testing laboratory to perform testing on relatively undisturbed soft compressible soil samples.
- Prepare this letter report summarizing our findings, conclusions, and recommendations.

2. FIELD INVESTIGATION – SOIL TEST BORINGS

SGH performed the supplemental limited subsurface investigation at the site on 1 and 2 June 2018. The investigation consisted of eight soil test borings (SGH-2018-1, SGH-2018-2, SGH-2018-2A, SGH-2018-3, SGH-2018-4, SGH-2018-5, SGH-2018-6, and SGH-2018-7) located around the perimeter of the EBSCO Facility. Soil test boring locations are shown in Figs. 1 and 2. Soil test borehole drilling was performed by Carr-Dee Corporation (Carr-Dee), of Medford, Massachusetts under SGH supervision. Site access was provided by EBSCO, and SGH coordinated boring locations with facilities personnel from the EBSCO Facility. EBSCO requested that SGH not perform soil test borings located adjacent to the north elevation of the EBSCO Facility due to existing buried utilities (including a buried fiber optic cable), the specific locations of which are currently unknown. EBSCO also requested that we not disturb paver site finishes for the patio area. There was also limited access due to steep-sloped site finishes at portions of the facility on the west elevation.

Steven Keppel and Zachary Boswell from SGH were present during the field work to observe drilling, assist in obtaining samples, and prepare a descriptive log of each test boring. The sampling intervals, soil descriptions, Standard Penetration Test (SPT) blowcounts, and other pertinent field data are summarized in the individual soil boring logs included in Appendix B. The soil test borings were performed on the north, south, and west elevations of the EBSCO Facility (Photos 1, 2, and 3). The east elevation of the EBSCO Facility (referred to as the Riverfront Foundation wall in our previous report) borders the Ipswich River.

Carr-Dee drilled soil test borings with a Mobile soil scout track drill rig or truck-mounted drill rig using the case and wash method with a 4.5 in O.D. casing. One borehole (SGH-2018-3) was drilled using a 2-1/4 in. I.D. hollow stem auger. Soil samples were obtained using 2 in. O.D. split spoon samplers driven using a 140 lb donut hammer falling 30 in. with a rope cathead. Relatively undisturbed soil samples of soft compressible soils were obtained using a thin-walled Shelby tube.

Soil test borings extended into dense Glacial Till or to split spoon refusal, which ranged between El. -11 ft and El. 8.6 ft. (i.e., 7.5 to 24 ft below ground surface, bgs). Split spoon samples were obtained at 5 ft intervals, except in soft compressible soils where continuous SPT sampling and Shelby tubes were obtained. We encountered wood debris in the wash water while drilling through the soft compressible soils at Boring SGH-2018-2. We had poor sample recovery rates at the same depths where we observed wood in the wash. We drilled Boring SGH-2018-2A, located approximately 4 ft from SGH-2018-2, in order to collect Shelby tube samples in the soft compressible soils. Therefore, we terminated this boring prior to reaching the Glacial Till stratum or refusal. All samples were secured, sealed, and transported to the SGH office at the end of the soil test boring program.

We attempted to drill a soil test boring through an existing concrete pad adjacent to Building No. 10-A. After two attempts we abandoned this location after reaching refusal at a depth of approximately 11 in. on steel reinforcement placed both ways within the slab. We moved the drill rig just outside the slab and completed Soil Test Boring SGH-2018-3.

We estimated elevations based on our measurements for the top of the Riverfront Foundation Wall in the February 2018 SGH Report.

3. ANALYSIS

3.1 Subsurface Conditions

Ground surface conditions consist of asphalt pavement or topsoil. The asphalt pavement is 4 in. to 9 in. thick and was encountered at Soil Test Borings SGH-2018-1, SGH-2018-5, and SGH-2018-6. The topsoil consists of 3 to 6 in. of a brown, dry, sandy silty loam. Topsoil was encountered in Soil Test Borings SGH-2018-2, SGH-2018-2A, SGH-2018-3, SGH-2018-4, and SGH-2018-7.

We prepared a subsurface profile, transverse to the Ipswich River, along the south elevation of the EBSCO Facility based on the results of the soil test borings performed by SGH (SGH-2018-1 to SGH-2018-7) and others (B-2 to B-4) (Fig. 3). We summarize the subsurface strata encountered in the following sections.

3.1.1 Subsurface Conditions - Southeast of EBSCO Facility

We summarize the soil strata encountered southeast of the EBSCO Facility (Soil Test Borings B2, B-3, B-4, SGH-2018-2, SGH-2018-2A, and SGH-2018-3) as follows:

- **Stratum 1 – Fill:** This stratum consists of 3 to 10 ft of a loose to medium dense, brown, dry to wet, silty sand to sandy gravel, fine to coarse grained, poorly graded, subangular, with trace wood and trace brick. SPT blow counts ranged from 2 to 15 blows per foot (bpf).

- **Stratum 2a – Sand and Silt:** This stratum consists of 3 ft of very loose brown, sand and silt, fine grained. SPT blow counts were 2 bpf. This stratum was encountered in Soil Test Boring SGH-2018-2 underlying the Fill stratum.

- **Stratum 3 – Upper Silty Clay:** This stratum consists of 1 to 5.5 ft of very soft to stiff, gray to olive gray, moist to wet silty clay. SPT blow counts ranged from 2 to 10 bpf. This stratum was encountered in Soil Test Borings SGH-2018-3, B-3, and B-4 underlying the Fill stratum and in Soil Test Boring SGH-2018-2 and SGH-2018-2A underlying the Sand and Silt stratum. Fine- to coarse-grained silty sand seams were observed within this stratum at Soil Test Borings SGH-2018-2A, B-3, and B-4. We encountered wood debris in the wash water while drilling at Soil Test Boring SGH-2018-2.

- **Stratum 4 – Organic Silt:** This stratum consists of 1.5 to 2.5 ft of very soft to firm, grey to black, wet, organic silt, with trace to some fine sand. SPT blow counts range from 2 to 5 bpf. This stratum was encountered at Soil Test Borings SGH-2018-2, SGH-2018-2A, SGH-2018-3, B-3, and B-4, underlying the Fill or Upper Silty Clay strata. The measured organic content ranges from 10.3% to 11%.

  A 5 ft thick organic silt stratum was identified as a Peat stratum by others in Soil Test Borings B-3 and B-4. However, the soil description in the logs indicates "fine Sand and Silt with some organics (PEAT)." Furthermore, only 12 in. of this organic soil material was sampled; no other sampling was performed within this stratum. In our soil profile (Fig. 3) we classify this stratum as Organic Silt with a thickness of 2.5 ft (instead of 5 ft shown on logs for B-3 and B-4) based on the description of the soils in these boring logs, the lack of continuous sampling by others through this stratum, and our visual observations and laboratory test results at the soil test borings performed in 2018 that are located within close proximity to Soil Test Borings B-3 and B-4.

- **Stratum 5 – Lower Silty Clay:** This stratum consists of 2 to 6 ft of firm to very stiff, gray to olive gray, moist to wet silty clay. SPT blow counts ranged from 5 to 20 bpf. This stratum was encountered in Soil Test Borings SGH-2018-2 (Photo 4), SGH-2018-2A, and SGH-2018-3 underlying the Organic Silt stratum. Fine- to coarse-grained silty sand seams were observed within this stratum at Soil Test Boring SGH-2018-2A.[1] We encountered wood debris in the wash water while drilling at Soil Test Boring SGH-2018-2.

- **Stratum 6 – Glacial Till**: This stratum consists of medium dense to very dense, light reddish brown to olive grey, dry to wet, sand to sandy gravel, fine to coarse, well to poorly graded, subangular, with trace silt. SPT blow counts ranged from 21 bpf to refusal. This stratum was encountered at Soil Test Boring B-2, underlying the Fill stratum; at Soil Test Boring SGH-2018-2, underlying the Lower Silty Clay stratum; and at Soil Test Borings B-3 and B-4, underlying the Organic Silt stratum. Soil test borings were terminated in the glacial till layer and SGH or others did not determine the stratum thickness at these locations. See Table 1 below for the approximate elevation of the top of the Glacial Till stratum.

- **Stratum 7 – Rock Ledge:** The elevation of the top of the rock ledge varies at the site. Prior test pit investigations and bathymetric survey results indicate that the rock ledge varies between approximately El. 3.0 ft and El. 7.5 ft near the Ipswich Mills Dam.

3.1.2 Subsurface Conditions North, West, and Southwest of EBSCO Facility

We summarize the strata encountered on the north, west, and southwest elevations of the EBSCO Facility (Soil Test Borings SGH-2018-1, SGH-2018-4, SGH-2018-5, SGH-2018-6, SGH-2018-7, SGH-2016-1, and SGH-2016-2) as follows:

- **Stratum 1 – Fill:** This stratum consists of 2.5 to 8 ft of a loose to very dense, brown, dry to wet, silty sand to sandy gravel, fine to coarse grained, poorly graded, subangular, with trace wood and trace brick. SPT blow counts ranged from 4 to 73 bpf.

- **Stratum 2b – Silty Sand:** This stratum consists of 2 to 4.5 ft of medium dense to very dense, light orange brown, dry, silty sand to gravelly sand, fine to coarse grained, uniform to well-graded, subangular. SPT blow counts ranged from 16 to 99 bpf. This stratum was encountered at Soil Test Borings SGH-2016-2 and SGH-2016-3 underlying the Fill stratum.

- **Stratum 2c – Clayey Silt:** This stratum consists of 2.5 to 5 ft of medium stiff to hard, brown, gray, or olive, clayey silt with trace fine sand. SPT blow counts ranged from 6 to 39 bpf. This stratum was encountered in Soil Test Borings SGH-2018-5 and SGH-2018-6, located at the southwest corner of the EBSCO Facility, underlying the Fill stratum.

[1] The laboratory reports Sample SGH-2018-3 US-3 (depth 13.5 to 15.5) as Grey Varved Soil. We did not observe varved soil during drilling.

- **Stratum 7 – Glacial Till**: This stratum consists of medium dense to very dense, light reddish brown to olive grey, dry to wet, sand to sandy gravel, fine to coarse, well to poorly graded, subangular, with trace silt. SPT blow counts ranged from 28 bpf to refusal.

  This stratum was encountered at Soil Test Borings SGH-2018-1, SGH-2018-4, SGH-2018-7, and SGH-2016-1 underlying the Fill stratum; at Soil Test Borings SGH-2018-5 and SGH-2018-6 underlying the Clayey Silt stratum, and at Soil Test Borings SGH-2016-2 and SGH-2016-3 underlying the Silty Sand Stratum. See Table 1 below for approximate elevations of the top of the Glacial Till stratum.

3.1.3 Summary of Top of Glacial Till Stratum Elevations

Table 1 summarizes the depth and elevation of the top of the Glacial Till stratum:

Table 1: Top of Glacial Till Stratum

	Soil Test Borings											
	SGH-2018-1	**SGH-2018-2**	**SGH-2018-4**	**SGH-2018-5**	**SGH-2018-6**	**SGH-2018-7**	**SGH-2016-1**	**SGH-2016-2**	**SGH-2016-3**	**B-2**	**B-3**	**B-4**
Depth (ft)	6.0	19.0	6.0	13.0	5.0	5.0	4.5	7.5	8.0	10.0	16.0	16.0
Elevation (ft)$^{(1)}$	11.0	-6.0	10.0	3.0	8.0	7.0	12.5	10.5	6.0	3.0	-3.0	-3.0

(1) Elevations are estimated and referenced to the NAVD88 datum.

3.2 Groundwater Conditions

SGH did not measure groundwater levels during drilling at the soil test borings due to the cased-and-washed drilling method artificially raising the water levels within the borehole. After drilling was completed, SGH observed the groundwater level at approximately El. 8 ft at Soil Test Borings SGH-2018-2, SGH-2018-2A, and SGH-2018-3 located on the south elevation of the EBSCO Facility, which was generally consistent with the level of the Ipswich River during drilling.

We did not observe groundwater in the soil test borings performed in 2016 in Estes Street and Saltonstall Street (SGH-2016-1 and SGH-2016-2), which were terminated at approximately El. 6 ft and El. 7.5 ft respectively.

3.3 Settlement of Compressible Soils

Soft compressible soils are present at the southeast elevation of the EBSCO Facility (Soil Test Borings B3, B4, SGH-2018-2, SGH-2018-2A, and SGH-2018-3). For our settlement analysis, we assumed a soil profile similar to the conditions encountered at Soil Test Borings SGH-2018-2/2A and B-3. We assumed that the soil profile consists of a 3 ft thick Fill stratum overlying a 1 ft thick Sand stratum overlying a 5 ft thick Upper Silty Clay stratum overlying a 2.5 ft thick Organic Silt stratum overlying a 6 ft thick Lower Silty Clay stratum. We estimated soil properties for the Upper Silty Clay, Organic Silt, and Lower Silty Clay strata based on laboratory consolidation tests performed on relatively undisturbed soil samples obtained from Soil Test Boring SGH-2018-2A.

We estimated the potential settlement of compressible soils due to primary consolidation imposed by an increase in effective stress due to lowered groundwater levels, and secondary compression of the organic soils after primary consolidation is complete[2]. For this analysis we assumed that the organic soils are normally consolidated and the additional stress applied to the organic soils due to groundwater drawdown will reinitiate secondary compression.

The laboratory test results show that the Lower Silty Clay stratum is over-consolidated; we estimate an over-consolidation ratio of approximately 4 for this stratum[3]. We assumed that the Upper Silty Clay stratum has similar consolidation parameters as the test sample from the Lower Silty Clay stratum, except that we assumed the Upper Silty Clay is normally consolidated to match the underlying Organic Silt stratum conditions determined from the laboratory test results.

In the February 2018 SGH Report, we estimated an initial groundwater level at approximately El. 6 ft based on our observations of the river staff gauge during our investigation in August 2016 and on groundwater data collected at one observation well. We assumed that the overburden soils under the EBSCO Facility have experienced groundwater levels as low as El. 6 ft; we calculated the range of potential settlement of the clay and organic soils resulting from a 1, 2, and 3 ft drop in groundwater levels. We understand that HWG has not yet completed the hydraulic study of post-dam-removal river levels; however, the revised preliminary estimated lower-bound water level after the proposed dam removal is at approximately El. 1 ft (i.e. a 5 ft drop in groundwater level), and will likely be higher (between El. 3 ft and El. 6 ft). The water level is subject to change pending results from the hydraulic analysis performed by HWG. For our analysis of the potential settlement of compressible soils, we considered a range of potential low-water river elevations between El. 1 ft and El. 5 ft.

Table 2 summarizes the soil consolidation soil parameters used in our analysis:

[2] Primary consolidation settlement is load-dependent and occurs when load is transferred to the soil structure and pore water is squeezed out of the soil mass. Secondary compression settlement is time-dependent and occurs after primary consolidation is complete. Secondary compression occurs under constant load and can be significant for organic soils due to creep, and compression and degradation of the organic material.

[3] Normally consolidated and over-consolidated are terms that refer to the current vertical overburden pressure on the soil relative to the maximum vertical overburden pressure the soil has ever experienced. A normally consolidated soil has a current pressure equal or nearly equal to the maximum experienced pressure. An over-consolidated soil has previously experienced a higher pressure, which can be due to natural or man-made causes, compared to the current pressure. The over-consolidation ratio is the ratio of the maximum past pressure relative to the current pressure.

Table 2 – Clay and Organic Silt Strata Consolidation Parameters(1)

Soil Stratum	Depth to Mid-Layer [ft, bgs]	σ'_{vo}(2) [psf]	σ'_{vf}(2) [psf] Case 1	Case 2	Case 3	Case 4	Case 5	σ'_p(2) [psf]	Initial Void Ratio e_0	C_c(3)	C_r(3)	C_α(3)
Upper Silty Clay	7.75	872	919	919	919	919	919	872	0.897	0.36	0.05	--
Organic Silt	10.25	991	1,054	1,116	1,178	1194	1194	1,000	2.87	1.18	0.17	0.021
Lower Silty Clay	14.5	1,206	1,268	1,331	1,393	1455	1518	5,000	0.897	0.36	0.05	--

(1) Case Nos. 1 through 5, correspond with a groundwater level drawdown of 1 ft through 5 ft respectively (i.e. groundwater level at El. 5 ft, El. 4 ft, El. 3 ft, El. 2 ft, and El. 1 ft respectively).
(2) σ'vo is the estimated existing overburden or in situ vertical effective stress at midlayer (prior to dam removal). σ'vf is the estimated vertical effective stress after dam removal (lowered groundwater level). σ'p is the maximum past pressure experienced by the soil estimated from laboratory test results for SGH-2018-2A US-1 and US-3. The soil profile is estimated from Soil Test Borings SGH-2018-2 and SGH-2018-2A.
(3) Cc is the primary consolidation index, Cr is the recompression index, Cα is the secondary compression index. We did not estimate secondary compression for the Silty Clay.
(3) We assumed the lower half (depths ranging between 6.5 ft and 9 ft) of the Upper Silty Clay stratum contributes to settlement and is normally consolidated. We assumed consolidation parameters for the Upper Silty Clay are similar to the test results for the Lower Silty Clay stratum.

Based on the assumptions listed above, we estimate that the average primary settlement due to lowering groundwater levels by between 1 ft and 5 ft in the post-dam-removal conditions is in the order of between 0.4 in. and 1.0 in., in those areas where compressible soils are present.

We estimate that secondary compression, which is time-dependent strain, of the organic silt stratum will be about 0.5 in. For the purpose of this calculation we assumed a remaining service life of 50 yrs for the EBSCO Facility (we have not performed a service life evaluation of the structure). Including primary consolidation for the 1 ft to 5 ft drawdown scenarios, the total estimated settlement of the soft compressible soils is approximately between 0.9 in. and 1.5 in. after 50 yrs from drawdown.

Table 3 summarizes our estimated settlement of compressible soils resulting from drawdown of groundwater levels:

Table 3 - Estimated Settlement due to Drawdown

Case No. / Drawdown [ft]	Primary Consolidation Settlement(1) [in.]	Secondary Compression(2) [in.]	Total Settlement [in.]
1	0.4	0.5	0.9
2	0.7	0.5	1.2
3	0.9	0.5	1.4
4	1.0	0.5	1.5
5	1.0	0.5	1.5

Notes
(1) See Table 2 for soil properties.
(2) Estimated secondary compression 50 yrs after end of primary consolidation, assuming normally consolidated soils.

4. DISCUSSION

4.1 Supplemented EBSCO Facility Elevations

We estimated ground surface and soil strata elevations based on our reported elevations for the Riverfront Foundation Wall in the February 2018 SGH Report. Table 4 below summarizes elevations pertinent to the EBSCO Facility updated to include additional elevations for the top of Glacial Till based on the recent field investigation. We understand from HWG that the preliminary estimate for the low river level is likely in the range of El. 1 ft to El. 6 ft after dam removal (elevation is subject to change pending the results of the hydrologic and hydraulic analysis).

Table 4 – Water Level, Glacial Till, and EBSCO Foundation Wall Elevations

Description	Elevation [ft, NAVD 88]
Water Levels	
Estimated Low River Level Elevation After Dam Removal (Preliminary Estimate from HWG)	1 to 6
South End of the EBSCO Facility	
Top of Foundation Wall at Building No. 10-A.	12.5
Maximum Elevation of Bottom of Foundation Wall at Building No. 10-A / Bottom of Test Pit No. 2 (TP-2)	–0.5
Approximate Range of top of Organic Silt Stratum at South End of Building No. 10A	1.5 to 4
Approximate Range of top of Glacial Till Stratum at South End of Building No. 10A	-6 to 3
North End of the EBSCO Facility (Closest to Dam)	
Top of Foundation Wall at Building No. 9.	11.4
Apparent Bottom of Foundation Wall at Building No. 9 / Bottom of Test Pit No. 1 (TP-1)	3.2
Dam Crest	8.9
Approximate Top of Glacial Till Stratum at North End of Building No. 9	7
Average Elevation of Rock Ledge at Toe of Dam	2.9
Top of Abandoned Timber Formwork and Abandoned Timber Wall	5.7

4.2 Likelihood of Presence of Timber Pile Foundations at EBSCO Facility

We encountered a shallow depth to the top of the Glacial Till stratum in the soil test borings located on the northern end of Buildings No. 9, No. 10, No. 10B and No. 11 (Soil Test Borings SGH-2018-1, SGH-2018-4, SGH-2018-7, SGH-2016-1, and SGH-2016-2). The depth to the top of the Glacial Till stratum in this area ranged from 4.5 ft to 7.5 ft below ground surface (bgs) (i.e., El. 7 to 12.5). Considering a minimum depth to bottom of footing of approximately 3 ft bgs for exterior foundations and approximately 1 ft below top of slab-on-grade for interior foundations, it is unlikely that timber piles were installed in these areas, as the timber piles would be in the order of 3.5 to 6.5 ft long at most. It is likely that the original foundation construction in this area included over-excavation to place shallow footings bearing directly on the Glacial Till stratum. Based on the limited subsurface information gathered to date, it is very likely that the exterior walls and interior columns of Building No. 9, Building No. 10, Building No. 10B, and the northern portion of Building No. 11 are founded on shallow spread footings bearing on the Glacial Till stratum or rock.

We encountered the top of the Glacial Till stratum on the south end of Building No. 11 (Soil Test Borings SGH-2018-5 and SGH-2018-6) at depths ranging between 5 to 13 ft bgs (El. 3 to 8 ft). At Soil Test Boring SGH-2018-5, where the depth to the top of Glacial Till was 13 ft bgs, the Glacial

Till stratum was overlain by 2.5 to 5 ft of medium stiff to hard fine-grained soils (Clayey Silt stratum); that is, the top of the Clayey Silt stratum is at 2.5 to 8 ft bgs in this area. Considering a minimum depth to bottom of footing of approximately 3 ft bgs for exterior foundations and approximately 1 ft below top of slab-on-grade for interior foundations, it is possible but unlikely that timber piles were installed in this area, as the timber piles would be in the order of 4 to 12 ft long at most if bearing on the Glacial Till stratum, and 1.5 to 5 ft long at most if bearing on the Clayey Silt stratum. It is highly likely that shallow soil bearing foundations bearing on the medium-stiff to hard natural fine-grained soils (Clayey Silt statum) were used in this area. Based on the limited subsurface information gathered to date, it is likely that the exterior walls and interior columns of the southern portion of Building No. 11 are founded on shallow spread footings bearing on the Clayey Silt stratum.

The top of the Glacial Till stratum is generally deeper on the south end of Building No. 10A and Building No. 11A (Soil Test Borings B-2, B-3, B-4, SGH-2018-2, SGH-2018-2A, and SGH-2018-3). The depth to the top of the Glacial Till stratum in these borings ranged from 13 to 19 ft bgs (El. -6 ft to 3 ft). The Glacial Till stratum is overlain by 4 to 13 ft of soft to medium-stiff fine-grained soils, including about 2.5 ft of Organic Silt. In our February 2018 SGH Report we noted that it was unlikely that the Riverfront Foundation Wall at Building No. 10A was founded on timber piles given the depth to which excavation was performed to install the wall (greater than 10 ft) and the maximum 5.5 ft depth to top of Glacial Till stratum from the estimated maximum elevation of bottom of Riverfront Foundation Wall (El. -0.5 ft). The construction of the land-side exterior foundation walls and interior column foundations at Buildings No. 10A and No. 11A is not known. The depth and thickness of the observed compressible soils in this area is such that timber piles may have been driven through the soft compressible soils to bear on the Glacial Till stratum below to support the building structure in these areas.

4.3 Settlement of Compressible Soils

Lowered groundwater levels due to potential drawdown after dam removal would increase effective stresses in soft compressible soils and result in settlement of pavement, slabs-on-grade, and structures on spread footings or buried utilities supported above these soft compressible soils. We did not encounter soft compressible soils in soil test borings located on the west and north elevations of the building away from the river; however, we observed soft compressible soils at all soil test borings on the southeast elevation, near the river. Our observations at Soil Test Borings SGH-2018-2, SGH-2018-2A, and SGH-2018-3 indicate that the compressible organic stratum may not be as thick as indicated on prior soil test borings performed by others. Prior soil test borings also indicated the presence of peat. Our visual observations and laboratory results show soils consistent with organic silt. The laboratory test results for organic content measured between 10.3% and 11% percent organic content of the two samples tested, supporting this change in soil description.

In the February 2018 SGH Report we estimated settlement of the soft compressible soils observed by others using assumed soil properties based on ranges of published values for organic soils and local clays. We updated our calculations using revised strata thicknesses, depths, and consolidation parameters determined from our laboratory testing. We tested relatively undisturbed (Shelby tube) samples for one Lower Silty Clay sample and one Organic Silt sample, and we estimated consolidation parameters from the test results. The tested samples are representative of the soil strata, but the in-situ soil properties may vary compared to our limited sample testing results. The laboratory test results show that the Organic Silt is nearly normally consolidated; therefore, a relatively small increase in stress due to drawdown of groundwater levels will result in potentially significant consolidation settlement. Our estimated consolidation coefficients based on laboratory testing of the Lower Silty Clay are consistent with the average values used in our prior analysis, and the estimated consolidation coefficients from laboratory testing of the Organic Silt are consistent with lower-bound published values. Our refined total settlement estimate of the localized soft compressible soils based on laboratory data of the sampled in-situ soils shows a somewhat smaller magnitude of settlement compared to our previous estimate based on only published data. We estimate that the average primary settlement due to lowering groundwater levels by between 1 ft and 5 ft in the post-dam removal conditions is in the order of between 0.4 in. and 1.0 in., in those areas where compressible soils are present. At this time it is uncertain to what extent, if any, compressible soils may or may not underlie the EBSCO Facility. Based on results of the soil test boring located on the building exterior, it is unlikely that organics underlie the northern and western portion of the EBSCO Facility.

We also updated our settlement estimate to include secondary compression, which is long-term time-dependent compressive strain that will occur for many years after primary consolidation is complete. We estimated secondary compression 50 yrs after the end of primary consolidation, assuming normally consolidated organic soils. We estimate secondary compression of the organic silt stratum will be nearly 0.5 in. after 50 yrs. Including primary consolidation for scenarios ranging between 1 ft and 5 ft of drawdown, the total settlement of the soils would range between 0.9 in. and 1.5 in. respectively after 50 yrs.

5. CONCLUSIONS

Based on the recent June 2018 investigation, we have the following conclusions to supplement our February 2018 SGH Report regarding the potential impacts of the dam removal on the adjacent EBSCO Facility:

- Existing Foundations:
  - Based on the limited subsurface information gathered to date, it is very likely that the exterior walls and interior columns of Building No. 9, Building No. 10, Building No. 10B, and the northern portion of Building No. 11 are founded on shallow spread footings bearing on the Glacial Till stratum or rock.
  - Based on the limited subsurface information gathered to date, it is likely that the exterior walls and interior columns of the southern portion of Building No. 11 are founded on shallow spread footings bearing on the Clayey Silt stratum.
  - It is unlikely that the Riverfront Foundation Wall at Building No. 10A was founded on timber piles given the depth to which excavation was performed to install the wall (greater than 10 ft) and the maximum 5.5 ft depth to the top of the Glacial Till stratum from the estimated maximum elevation of the bottom of Riverfront Foundation Wall (El. -0.5 ft). The construction of the land-side exterior foundation walls and interior column foundations at Buildings No. 10A and No. 11A is not known. The depth and thickness of the observed compressible soils in this area are such that timber piles may have been driven through the soft compressible soils to bear on the Glacial Till stratum below to support the building structure in these areas.
  - At this time it is uncertain to what extent, if any, compressible soils underlie the EBSCO Facility. We did not encounter soft compressible soils in soil test borings located on the west and north elevations of the building away from the river.

- Effects of Lowering Groundwater: Lowered groundwater levels could result in settlement of pavement, slabs-on-grade, and structures on spread footings or buried utilities supported by soft compressible soils. We estimate a potential total settlement of the soft compressible soils of approximately between 0.9 in. and 1.5 in. respectively due to a water level drawdown of between 1 ft and 5 ft, assuming a remaining service life of 50 years for the EBSCO facility, in those areas where compressible soils are present.

- Subsequent Steps: If the project team anticipates that the post-dam removal groundwater levels cannot be maintained at or above El. 6 ft, the following approach could be implemented to assess potential settlement of structures bearing on compressible soils:

  - Conduct a subsurface investigation consisting of test pits and soil test borings performed within the EBSCO Facility, focused on Buildings 10A and 11A where the foundation construction is unknown and compressible soils are potentially present. This portion of the EBSCO Facility presents the highest risk of settlement due to drawdown of groundwater levels.
  - Develop and implement a precision movement monitoring program to monitor for the potential movement of structures during dam removal construction. Install the instrumentation prior to the start of construction, and also establish acceptable settlement limits with approval from EBSCO.

Limitations of Current Investigation

The information presented herein is based on the geotechnical information collected to date. The boring logs and geotechnical investigation records depict subsurface conditions only at the specific soil sampling locations. Subsurface conditions at other locations may differ from conditions observed at specific sample depths and exploration locations. There is no warranty or guarantee, either expressed or implied, that the conditions indicated by such investigations or records thereof are representative of those existing throughout such areas, or any part thereof, or that unexpected developments may not occur, or that materials other than, or in proportions different from, those indicated may not be encountered.

Sincerely yours,

William P. Konicki

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List of Attachments

Photos 1 through 4

Figures
Figure 1 – Subsurface Investigation Plan
Figure 2 – EBSCO Facility and Investigation Location Plan
Figure 3 – Soil Profile - EBSCO Facility

Appendices
Appendix A - Report titled Feasibility Study and Conceptual Plan for Ipswich Mills Dam Removal: Evaluation of Potential Impacts, EBSCO Facility Building, Ipswich, MA" prepared by SGH dated 17 February 2018 and revised 20 February 2018
Appendix B - 2018 Soil Test Boring Logs
Appendix C - 2018 Laboratory Test Results

PHOTOS

Photo 1

Location of Soil Test Boring SGH-2018-7 on the north elevation of the site adjacent to the patio area at the EBSCO Facility

Photo 2

West elevation of Building No. 10 and north elevation of Building Nos. 10-B and 11. The construction cone indicates the location of Soil Test Boring SGH-2018-4.

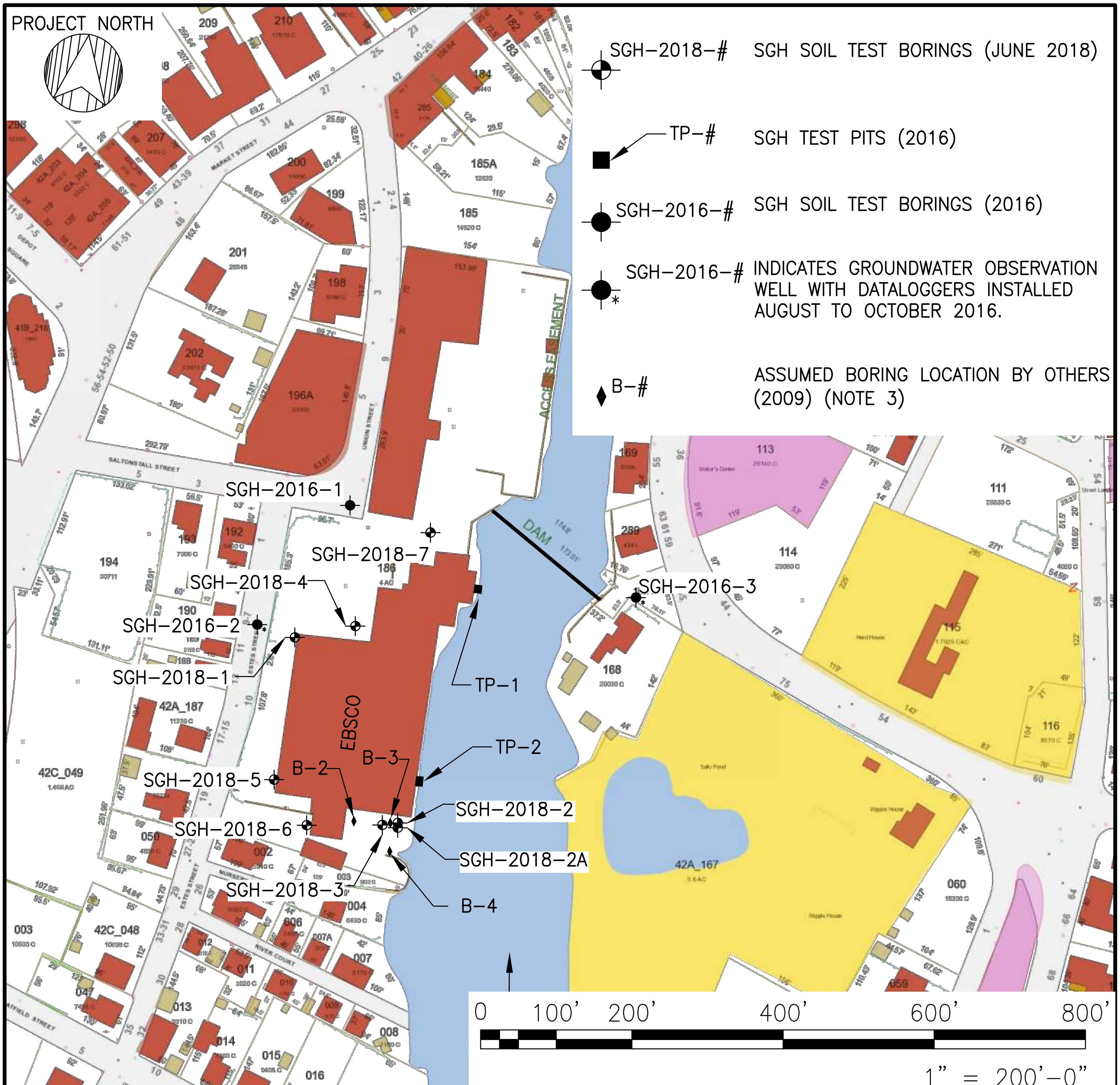
Photo 3

Drill rig set up at SGH-2018-3 on the south elevation of Building No. 10-A, adjacent to the Ipswich River. Arrow indicates the general location of Soil Test Borings SGH-2018-2 and SGH-2018-2A.

Photo 4

Split spoon sample collected in the Lower Silty Clay stratum at Soil Test Boring SGH-2018-2.

FIGURES



NOTES:

1. BASE PLAN OBTAINED FROM MASS GIS ONLINE VIEWER.
2. SGH TEST PIT AND BORING LOCATIONS ARE APPROXIMATE.
3. LOCATIONS OF THE BORINGS PERFORMED BY OTHERS IN 2009 ARE ESTIMATED FROM THE MEMORANDUM FROM GEI TO THE TOWN OF IPSWICH RE: EVALUATION OF POTENTIAL IMPACTS ON EBSCO BUILDINGS FROM PROPOSED REMOVAL OF IPSWICH MILLS DAM DATED 13 DECEMBER 2013 REVISED 14 FEBRUARY 2014.

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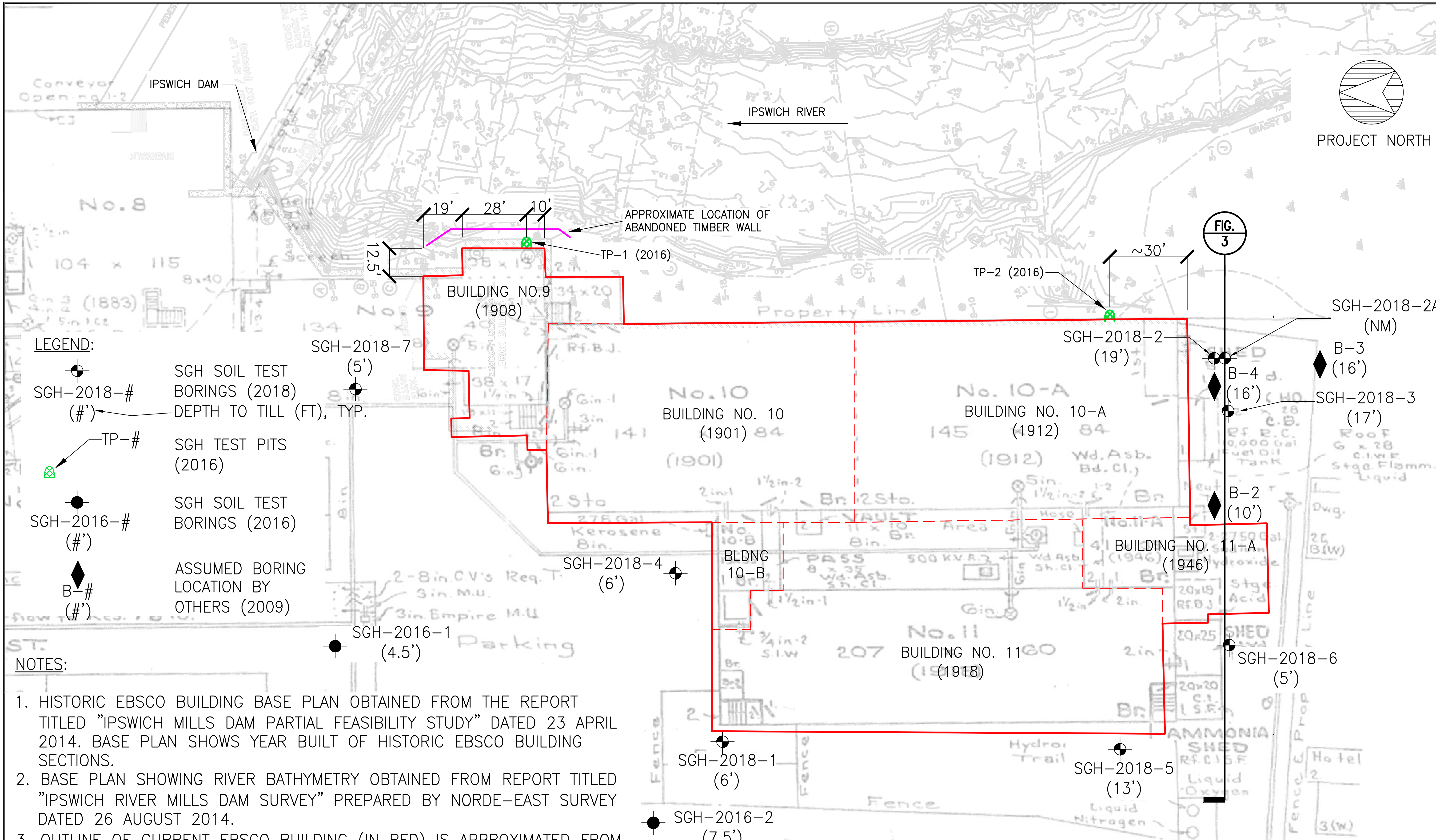
Project: FEASIBILITY STUDY FOR IPSWICH MILLS DAM REMOVAL IPSWICH, MA			
Title: SUBSURFACE INVESTIGATION LOCATION PLAN			
Drawn: ZKB	Checked: SFK	Approved:	Project No.: 160630.01

Drawing No.: FIG. 1
Scale: 1" = 200'
Date: 06/05/18



NOTES:

1. HISTORIC EBSCO BUILDING BASE PLAN OBTAINED FROM THE REPORT TITLED "IPSWICH MILLS DAM PARTIAL FEASIBILITY STUDY" DATED 23 APRIL 2014. BASE PLAN SHOWS YEAR BUILT OF HISTORIC EBSCO BUILDING SECTIONS.
2. BASE PLAN SHOWING RIVER BATHYMETRY OBTAINED FROM REPORT TITLED "IPSWICH RIVER MILLS DAM SURVEY" PREPARED BY NORDE-EAST SURVEY DATED 26 AUGUST 2014.
3. OUTLINE OF CURRENT EBSCO BUILDING (IN RED) IS APPROXIMATED FROM MASS GIS ONLINE VIEWER AND LIMITED FIELD MEASUREMENTS NEAR TEST PITS.
4. DASHED LINES INDICATING EXTENTS OF HISTORIC EBSCO BUILDING SECTIONS ARE APPROXIMATE.
5. NM = NOT MEASURED

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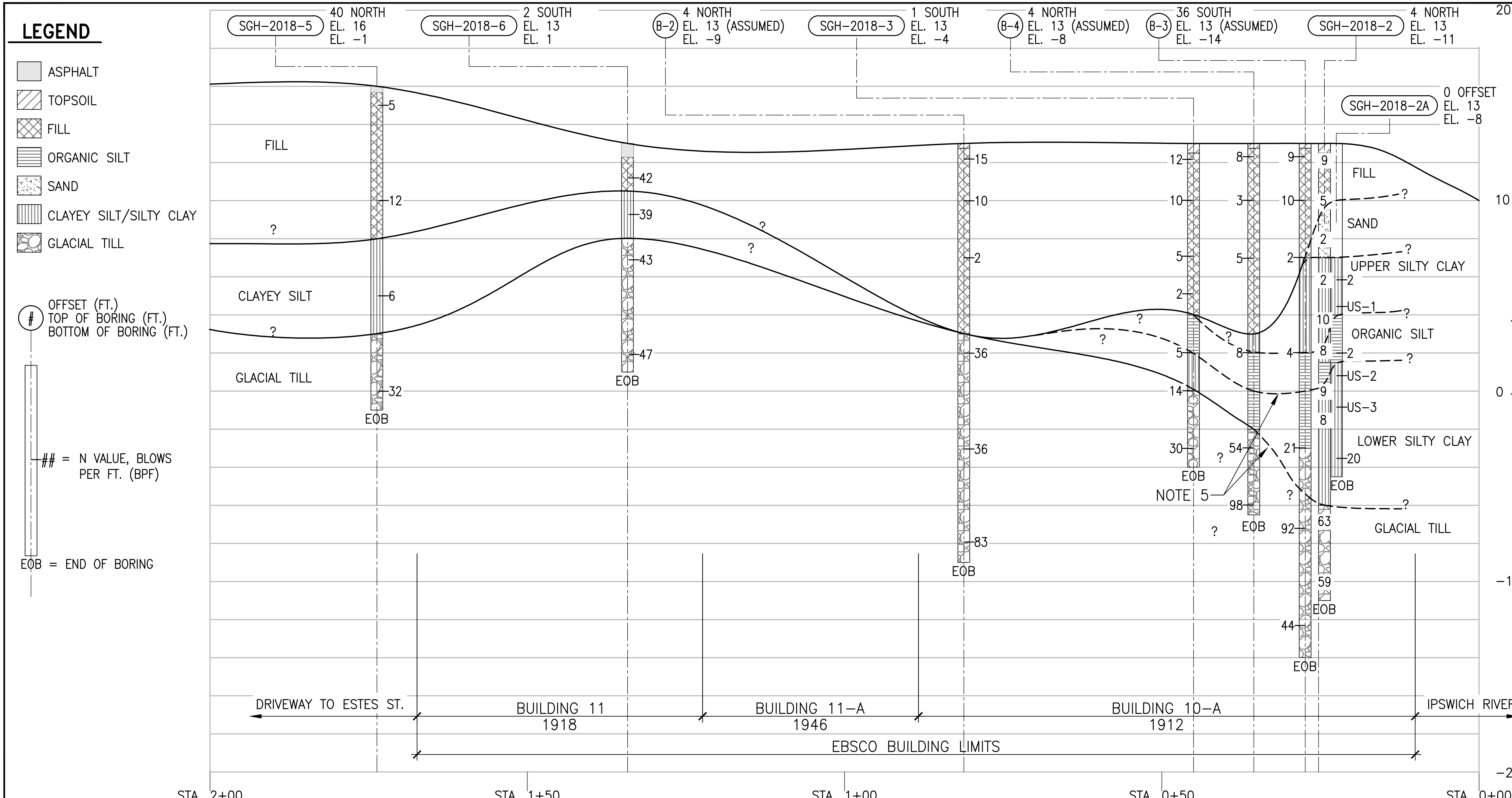
Project:	FEASIBILITY STUDY FOR IPSWICH MILLS DAM REMOVAL IPSWICH, MA
Title:	EBSCO FACILITY AND INVESTIGATION LOCATION PLAN
Drawn:	ZKB
Checked:	SFK
Approved:	GAZ
Project No.:	160630.01
Drawing No.:	FIG. 2
Scale:	1"=40'-0"
Date:	06/20/18



NOTES:
1. ALL ELEVATIONS ARE ESTIMATED AND REFERENCED TO THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88).
2. SOIL STRATA ARE GENERALIZED PROFILES INTERPRETED FROM BORING LOGS PREPARED BY SGH AND OTHERS. REFER TO SOURCE BORING LOGS FOR MORE DETAILED SOIL DESCRIPTIONS.
3. STRATIFICATION LINES REPRESENT APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES; TRANSITIONS MAY BE GRADUAL.
4. STATION NUMBERS FOR SOIL PROFILE ASSUME STATION 0+00 IS LOCATED AT THE EAST ELEVATION OF THE EBSCO FACILITY ADJACENT TO THE RIVER (I.E. THE RIVERFRONT FOUNDATION WALL). SEE FIGURE 2 FOR SECTION ALIGNMENT.
5. ORGANIC SILT AND CLAY STRATUM THICKNESS HAVE BEEN MODIFIED DUE TO THE LACK OF CONTINUOUS SAMPLING THROUGH THE ORGANIC SILT STRATUM PERFORMED BY OTHERS IN 2009, AT B-3 AND B-4.

SCALES
HORIZONTAL 1" = 15'
VERTICAL 1" = 5'

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Project:	FEASIBILITY STUDY FOR IPSWICH MILLS DAM REMOVAL IPSWICH, MA
Title:	SOIL PROFILE – EBSCO FACILITY
Drawn:	CRB
Checked:	ZKB/SFK
Approved:	GAZ
Project No.:	160630.01
Drawing No.:	FIG. 3
Scale:	AS NOTED
Date:	06/20/2018

APPENDICES

Appendix A

Report titled Feasibility Study and Conceptual Plan for Ipswich Mills Dam Removal: Evaluation of Potential Impacts, EBSCO Facility Building, Ipswich, MA” prepared by SGH dated 17 February 2017 and revised 20 February 2018

Feasibility Study and Conceptual Plan for Ipswich Mills Dam Removal: Evaluation of Potential Impacts

EBSCO Facility Building
Ipswich, Massachusetts

17 February 2017
(Revised 20 February 2018)

SGH Project 160630

COMMONWEALTH OF MASSACHUSETTS
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KONICKI
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PREPARED BY:

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SIMPSON GUMPERTZ & HEGER

Engineering of Structures
and Building Enclosures

17 February 2017
(Revised 20 February 2018)

Mr. Neal Price
Senior Hydrogeologist / Senior Project Manager
Horsley Witten Group
90 Route 6A
Sandwich, MA 02563

Project 160630 – Feasibility Study and Conceptual Plan for Ipswich Mills Dam Removal: Evaluation of Potential Impacts on the EBSCO Facility Building Foundations, Ipswich, MA

Dear Mr. Price:

Enclosed please find our report regarding the evaluation of the potential impact(s) of the proposed removal of Ipswich Mills Dam, and the subsequent lowering of water levels upstream of the dam, on the presumed timber pile foundations of the EBSCO Information Services facilities immediately upstream of the dam. This report summarizes our observations, findings, and conclusions.

If additional information becomes available, we reserve the right to supplement or modify the material presented herein.

Sincerely yours,

COMMONWEALTH OF MASSACHUSETTS
WILLIAM P. KONICKI
32170
REGISTERED PROFESSIONAL ENGINEER

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EXECUTIVE SUMMARY

The Ipswich Mills Dam is a run-of-the-river dam on the Ipswich River located in downtown Ipswich, Massachusetts. The dam is being considered for removal, and the purpose of the current study is to evaluate the impact the proposed dam removal could have on adjacent structures, specifically the EBSCO Information Services (EBSCO) facilities located on the west bank of the river immediately upstream of the dam.

A partial feasibility study for the Ipswich Mills Dam removal performed in 2014 opined that at least a portion of the EBSCO Facility may be supported by timber pile foundations. This opinion was based on borings performed in 2009, which encountered soft compressible soils, including peat, at the south end of the building, and the era of building construction, which was completed between 1901 and 1918. A foundation supported by untreated timber piles, if present, could be impacted by lowered water levels resulting from the removal of the Ipswich Mills Dam because the exposure of currently submerged piles could instigate fungal decay of the pile tops, resulting in settlement of the building. Additionally, lowered water levels could result in increased vertical stresses on the soil, leading to settlement of slabs-on-grade, shallow footings, buried utilities, or other buried structures, due to consolidation of soft compressible soils below the buildings, similar to the soils observed at the south end of the site.

The objective of SGH's scope of work for this project is to investigate the hypotheticals mentioned in the prior paragraph and thereby evaluate the potential impacts of the proposed dam removal on the EBSCO Facility. Our study includes investigation of the exterior foundation wall at the property line between the EBSCO Facility and the Ipswich River (referred to as the riverfront foundation wall in this report). SGH's scope of work is part of a larger feasibility study and concept plan for the dam removal, led by the Horsley Witten Group, Inc. (HW). The HW Team is performing a hydraulic analysis of the river and has provided an initial, planning-level estimate for the lowest water level likely to occur under a post–dam-removal scenario.

In August 2016, SGH performed a field investigation that consisted of two test pits located adjacent to the EBSCO Facility and excavated from the river side, and three soil test borings (two borings drilled uphill from the EBSCO Facility on the public street adjacent to the EBSCO property line, about 50 to 100 ft from the EBSCO Facility, and one boring drilled on the opposite side of the river on town property). A more comprehensive subsurface investigation was initially considered to better observe the different portions of the EBSCO Facility constructed in 1901, 1908, 1912, and 1918, as well as note any differences between interior and exterior foundations. However, due to funding and access constraints to the EBSCO facility, the current subsurface program was designed to avoid EBSCO Facility property areas in active use.

The following findings were developed from our subsurface investigation:

- Timber piles were not observed in our two test pits excavated in August 2016. The riverfront wall foundations of Building Nos. 9 and 10-A of the EBSCO Facility are bearing on rock and/or are bearing on competent soils or piled foundations at an elevation lower than the currently estimated low water level of the Ipswich River at the site after dam removal. Therefore, no indication was observed at our two test pit locations of the potential for fungal attack of timber piles in a post–dam-removal scenario.

- If timber piles exist at other locations supporting the EBSCO Facility, it is anticipated that the tops of the timber piles are at a low enough elevation to remain submerged after dam removal and, therefore, fungal deterioration of the tops of the timber piles would not occur.

- The three borings directed by SGH in August 2016 were located outside of the EBSCO site and did not encounter soft compressible soils. The 2009 borings performed by others at the southeast corner of the EBSCO Facility indicate the presence of localized soft compressible soils, including organics and clay, along the riverfront, which is common in riverfront settings. Where soft compressible soils are present, lowered groundwater levels could potentially result in settlement of pavement, slabs-on-grade, structures on shallow soil-bearing spread footings, or buried utilities supported above the soft compressible soils. We estimate a potential settlement of the soft compressible soils of approximately 1 in., 2.5 in., and 3.5 in. due to a water level drawdown of 1 ft, 2 ft, and 3 ft respectively (i.e. groundwater level at El. 5 ft, El. 4 ft, and El. 3 ft respectively). At this time it is uncertain to what extent, if any, compressible soils may or may not underlie the EBSCO Facility. We estimated the settlement assuming average soil properties from a range of published values for organic silt and clay.

Based on the results of the current investigation, we identify the following three options for the project team to determine next steps in the feasibility study for the Ipswich Dam removal:

- Option 1 – Maintain Current Groundwater Level During Post–Dam Removal. This option presents the least amount of risk for settlement due to timber pile deterioration or consolidation of compressible soils, if present, at the EBSCO Facility. Groundwater levels measured during our investigation were approximately El. 6 ft, therefore maintaining this groundwater elevation would likely not result in adverse impacts to the EBSCO Facility. Maintenance of current groundwater levels at approximately El. 6 ft would require evaluating appropriate approaches to dam removal or other engineered solutions such as groundwater recharge. Additional subsurface investigation would be required to evaluate the feasibility of applicable engineered solutions. This option also requires continuous monitoring of groundwater levels and structure movement to verify performance after the dam is removed, for the life of the structure.

If the project team anticipates that the post–dam-removal groundwater levels cannot be maintained at or above El. 6 ft, then one of the following two options may be implemented to determine risks to the EBSCO Facility and develop mitigation options if needed:

- Option 2 – Pre–Dam-Removal Supplemental Subsurface Investigation. This involves completing a supplemental foundation investigation in the building areas that were not accessible during the current investigation. Performing this investigation prior to completing the feasibility study for the dam removal would provide actionable information to perform a better assessment of the likelihood of the need for mitigation options, as it would allow the project team to identify whether timber piles are present in the remaining areas of the EBSCO Facility where test pits have not been performed, and would also allow to determine if soft compressible soils are present within the footprint of the EBSCO Facility. We consider that this option lowers the risk of adverse impacts from dam removal as it allows for timely planning and budgeting for mitigation, if needed, during the initial design phases of the project. The extent of post–dam-removal movement monitoring required to confirm adequate performance of the building would be determined based on the results of the supplemental subsurface investigation.

  An outline of the recommended supplemental investigation is included in Appendix A. We estimate that the order-of-magnitude cost for the supplemental investigation as outlined would be approximately $200,000, assuming adequate access for the investigation, minor dewatering required for test pits, and replacement of the concrete slab and asphalt pavement cut penetrations.

  If EBSCO does not provide access to the inside of its facility and access for test pit investigations on the exterior of the facility, then a limited soil test boring investigation could be performed on the building exterior. The limited investigation would include five to ten soil test borings drilled in the EBSCO Facility parking lot and other exterior areas near the building, such as the grassed area at the south end of the building. The soil test borings would provide some subsurface information for the EBSCO Facility site and allow the project team to further evaluate the potential risks due to the potential presence of compressible soils and timber piles, if any are deemed to be present. We estimate that the order-of-magnitude cost for the limited supplemental investigation would be approximately $50,000.

- Option 3 – Perform Pre– and Post–Dam-Removal Precision Movement Monitoring, No Supplemental Subsurface Investigation. We understand a staged drawdown test in combination with precision movement monitoring could be performed for an extended period of time prior to dam removal. The pre–dam-removal precision movement monitoring would help establish a baseline against which to compare post–dam-removal performance.

  Under this option, planning for the dam removal project would proceed without further information about the foundations in building areas outside the current study, and also without further information regarding the presence of soft compressible soils within the EBSCO Facility. The project team would rely solely on pre– and post–dam-removal precision movement monitoring to assess the building performance and determine if mitigation measures are required. Precision movement monitoring helps identify problem areas; however, limits to accuracy, access, and duration of monitoring make this a more reactive approach compared to the other options. We consider that this option results in a higher risk of potential unmitigated settlement of the building because some distress to the building utilities, adjacent structure and/or slab-on-grade may occur before the post–dam-removal precision monitoring program detects measurable movement. In addition, there is a higher risk of significantly underestimating or overestimating the costs of mitigation. We note that if post–dam-removal mitigation measures are required, the costs are more likely to be higher than had mitigation been performed pre–dam removal, as the costs of repairs of any building distress (cracks, unlevelness, etc.) would need to be included. This also requires access to the interior of the EBSO Buildings to install monitoring points and at each round of survey of the monitoring points over an extended period of time.

Table of Contents

Letter of Transmittal

EXECUTIVE SUMMARY

CONTENTS

PHOTOS

FIGURES

APPENDICES

1. INTRODUCTION

All elevations in this report are in feet referenced to the North American Vertical Datum of 1988 (ft NAVD 88) unless otherwise noted. Elevations in reports by others are reported in feet referenced to the National Geodetic Vertical Datum of 1929 (NGVD 29). We converted elevations referenced to NGVD 29 to elevations referenced to NAVD 88 using an offset of +0.8 ft.

1.1 Background

The Ipswich Mills Dam is a run-of-the-river dam on the Ipswich River located in downtown Ipswich, Massachusetts, approximately 4 mi from the river terminus at the Atlantic Ocean. The dam was originally constructed to provide power for local industry, but no longer serves this purpose. The dam is being considered for removal, and the purpose of the current study is to evaluate the impact the proposed dam removal could have on adjacent structures, specifically the EBSCO Information Services (EBSCO) facilities located on the west bank of the river immediately upstream of the dam.

On the EBSCO campus, the brick masonry building located at 10 Estes Street (referred to as the EBSCO Facility in this report) is located along the property line abutting the Ipswich River. The EBSCO Facility is a two-story structure with brick masonry exterior bearing walls and timber floor framing supported on interior timber columns. The first-floor level (i.e., top of slab) is located approximately 3 ft above the current impounded river levels (El. 12 ft +/-). Asphalt-paved parking and employee access areas are located to the west and north of the EBSCO Facility, and a brick pavement patio area is located adjacent to the north entrance.

A partial feasibility study for the Ipswich Mills Dam removal performed in 2014 concluded that at least a portion of the EBSCO Facility may be supported by timber pile foundations. This conclusion was based on borings performed in 2009, which encountered soft compressible soils, including peat, at the south end of the building, and the era of building construction, which was completed between 1901 and 1918.

A foundation supported by untreated timber piles could be impacted by lowered water levels resulting from the removal of the Ipswich Mills Dam – as the currently submerged piles became exposed, rapid fungal decay of the pile tops could ensue, resulting in settlement of the building. Additionally, lowered water levels could result in increased vertical stresses on the soil, leading to settlement of slabs-on-grade, shallow footings, buried utilities, or other buried structures and buried utilities due to consolidation of soft compressible soils below the buildings, similar to the soils observed at the south end of the site.

1.2 Objective

The objective of our work is to evaluate the potential impacts of the proposed dam removal on the EBSCO Facility located adjacent to the Ipswich Mills Dam. Our study includes investigation of the exterior foundation wall at the property line between the EBSCO Facility and the Ipswich River (referred to as the riverfront foundation wall in this report).

Simpson Gumpertz & Heger Inc. (SGH's) scope of work is part of a larger feasibility study and concept plan for the dam removal, led by Horsley Witten Group (HW).

1.3 Scope of Work

Our scope of work included the following:

- Review prior reports prepared by others related to the EBSCO Facility; see Appendix B and Appendix C.
- Perform a field investigation, which included the following:
  - Excavate two test pits in the Ipswich River immediately adjacent to the EBSCO Facility riverfront foundation wall.
  - Perform three soil test borings, located near the EBSCO campus on the west bank of the river and near the Ipswich Mills Dam on the east bank of the river.
  - Install two groundwater observation wells and temporarily monitor groundwater levels for a period of nearly two months.
- Prepare this report.

2. DOCUMENT REVIEW

2.1 Ipswich Mills Dam Partial Feasibility Study (2014)

We reviewed the report titled "Ipswich Mills Dam Partial Feasibility Study" prepared by Horsley Witten Group, GEI Consultants, and Clean Soils Environmental Ltd, dated 23 April 2014 (Appendix B). SGH assisted GEI in reviewing existing building foundations for the partial feasibility study. We note the following pertinent information related to the foundations of the EBSCO Facility:

- The dam is likely constructed on top of or at the toe of a rock ledge. At the time of the study the extent of the rock ledge was not well understood, but the study concludes that the height of the rock ledge will likely be a primary factor in determining the normal or low water surface elevation if the dam is removed.
- The elevation of the dam spillway crest and normal pool water surface is El. 8.9 ft.
- The elevation of the dam gated outlet and the average upstream river bed elevation at the upper falls is El. 6.7 ft.
- The average bed elevation at the dam toe is El. 2.9 ft. The Ipswich river is tidal downstream of the dam.
- The Federal Emergency Management Agency (FEMA) Flood Insurance Study (FIS) dated 1985 reports that the 500 yr recurrence interval flood results in a water surface at El. 13.9 ft near the Ipswich Mills Dam.
- The EBSCO Facility is composed of four buildings constructed at different time periods. Building Nos. 9, 10, 10-A, and 11 were constructed in 1908, 1901, 1912, and 1918 respectively. Building No. 9 was partially demolished. Building Nos. 9, 10, and 10-A abut the Ipswich River upstream of the dam.
- EBSCO commissioned a study to determine the feasibility of constructing a building addition in 2009. The study included three soil test borings reportedly performed at the south end of the building near the river. The 2009 soil test borings show the following:
    - At the two borings near the river (B-3 and B-4), there is 5 to 10 ft of very loose to loose fill, over 5 to 10 ft of soft to firm (medium stiff) clay and peat, over very dense glacial till. The top of the till is located approximately at 15 ft below ground surface (bgs) at both borings.
    - At the boring located further from the river (B-2), there is 10 ft of very loose to medium dense fill over very dense glacial till.
- GEI concluded that the soft compressible soils are inadequate to support the EBSCO Facility addition and that, taking into consideration the age of the buildings, the EBSCO Facility is likely supported by timber piles.

- GEI did not observe signs of distress in the EBSCO Facility facade based on limited visual observations.

- GEI noted the following: “We observed some dishing of the first floor slab indicating that the slab is a slab-on-grade and that the soils underlying the slab are compressible and had settled.”

- GEI recommended further investigation of the EBSCO Facility foundations, including but not limited to test pit investigation, probing along the foundation wall, lowering the impoundment to provide access for visual observation of the foundation wall, and performing core sampling of the foundation elements.

2.2 Bathymetric Survey (2014)

We reviewed the report titled “Surveyor’s Report for Ipswich Mills Dam Survey” prepared by Norde-East Survey, dated 26 August 2014 (Appendix C), which shows several transects that extend the full width of the river upstream of the dam. We note the following pertinent to the foundations of the EBSCO Facility:

- The results of the bathymetric survey performed approximately 15 ft upstream of the dam, apparently located on the rock ledge, show the following:

    - Soundings immediately adjacent to the dam show sediment thicknesses ranging between 0 and 1 ft thick.
    - Transect AI (approximately 170 ft long in a northwest to southeast direction) shows the river bed surface ranging between approximately El. 5.5 ft and El. 7.5 ft.

- The results of the bathymetric survey performed near the north (downstream) end of the EBSCO Facility show the following:

    - Soundings S-29 and S-30 show sediment ranging between 0.5 ft and 0.8 ft thick and consisting of silt to gravel and sand, respectively.
    - Transect D (approximately 150 ft long in a west to east direction) shows the river bed surface at approximately El. 6 ft near the EBSCO Facility and a low point at approximately El. 1 ft near the middle of the river.
    - Transect E (approximately 135 ft long in a west to east direction) shows the river bed surface at approximately El. 7 ft near the EBSCO Facility and a low point at approximately El. 0.5 ft near the middle of the river.

- The results of the bathymetric survey performed near the south (upstream) end of the EBSCO Facility show the following:

    - Sounding S-9, located near the EBSCO Facility shows sediment 1.0 ft thick, and consisting of silt.
    - Transect I (approximately 150 ft long in a west to east direction) shows the river bed surface at approximately El. 6 ft near the EBSCO Facility and a low point at approximately El. 2 ft near the middle of the river.

3. FIELD INVESTIGATION

SGH performed a field investigation between 22 and 24 August 2016. The field investigation consisted of two test pits adjacent to the EBSCO Facility and excavated from the river side, and three soil test borings (two borings drilled uphill from the EBSCO Facility and one boring drilled on the opposite side of the river on town property). Site access was coordinated with various personnel from EBSCO, HW, the Ipswich River Watershed Association, and the Ipswich Department of Public Works.

The EBSCO Facility incorporates four building structures (Buildings 9, 10, 10-A, and 11), each constructed at a different time, reportedly 1901, 1908, 1912 and 1918. As such, foundation conditions may vary from building to building. Due to access and budget constraints, we performed a limited investigation that included two test pit locations, both located at the riverfront foundation wall (Buildings 9 and 10-A), and three soil test borings drilled on public property. Two of the three soil test borings were located on Estes Street and Union Street outside the EBSCO property line, about 50 to 100 ft from the EBSCO Facility. In addition, access constraints also prevented SGH from obtaining sample cores of the EBSCO Facility foundation walls.

3.1 Test Pit Investigation

On 22 through 24 August 2016, Mr. Steven Keppel from SGH visited the site to observe and document the conditions at exterior test pits excavated by Pepperell Cove Marine of Portsmouth, New Hampshire (PCM). Ms. Mary Donlon from SGH also assisted with the field investigation on 24 August 2016.

The project team coordinated a drawdown of the dam impoundment during the test pit investigation. Drawdown started on the afternoon of 22 August. Prior to drawdown, the impoundment level was at approximately El. 8 ft (Photo 1). On 24 August around 3:00 p.m., the impoundment level was at approximately El. 6.2 ft (Photo 2). During the investigation, SGH measured impoundment level elevations with assistance from the IRWA using the on-site staff gauge located on the east bank upstream of the dam. We estimated test pit elevations based on the impoundment level measurements.

Both test pits were excavated in the Ipswich River adjacent to the EBSCO Facility. The test pit investigations were limited to observation of the riverfront foundation wall of the EBSCO Facility.

PCM performed underwater excavation, removing sediment with a handheld airlift or pressurized water jet and removing larger debris and small rip-rap by hand (Photos 3 and 4). Test pit locations are shown in Figs. 1 and 2. Test Pit No. 1 (TP-1) was located upstream of the dam at the north end of the EBSCO Facility (i.e., Building No. 9 constructed in 1908). Test Pit No. 2 (TP-2) was located upstream of the dam at the south end of the EBSCO Facility (i.e., Building No. 10-A constructed in 1912). Cross section sketches of each test pit are included in Figs. 3 and 4. Visibility underwater was limited and elevations of submerged soils and structures were estimated by PCM at the direction of SGH. We verified elevations with probe measurements where possible.

We summarize our field observations during test pit excavation as follows:

- Subsurface conditions in the river adjacent to the EBSCO Facility generally consist of very soft sediment overlying rip-rap. We observed the following at each test pit:

    - At TP-1, the sediment above the rip-rap is approximately 1.5 ft thick and consists of dark brown, very soft silt with trace debris (glass and brick). The rip-rap consists of subangular stones ranging between 3 in. and 12 in. diameter. PCM removed rip-rap between El. 5.2 ft and El. 3.2 ft and was unable to advance TP-1 beyond El. 3.2 ft (i.e., test pit refusal). PCM was unable to widen the test pit further by hand; however, we probed and determined that test pit refusal was likely due to a rock ledge at El. 3.2 ft.
    - At TP-2, the sediment above the rip-rap is approximately 4.5 ft thick and consists of dark brown, very soft silt with some organics, some clay, and trace debris (glass and brick). The rip-rap encountered at the bottom of the excavation consists of subangular stones generally larger than 12 in. diameter. PCM was unable to remove the rip-rap by hand at TP-2 and reached refusal at El. –0.5 ft.
    - We confirmed that the EBSCO Facility riverfront foundation wall extended to the bottom of each test pit (El. +3.2 ft and El. –0.5 ft at TP-1 and TP-2, respectively).

- Drawdown of the impoundment exposed a portion of the EBSCO Facility riverfront foundation wall that is typically submerged (Photos 5 and 6). We did not observe cracking or other indications of settlement of the concrete foundation wall. At approximately El. 10 ft and below, we observed staining and slight erosion of the concrete surface. At TP-1, the erosion of the concrete is most severe, up to 2 in. deep, at about El. 9.5 ft. The erosion of the concrete appears less severe below approximately El. 7 ft. We scanned the concrete foundation wall near TP-1 with a magnetic detection device (model Structure Scan Mini manufactured by GSSI). We were unable to detect steel reinforcement within 12 in. of the surface; however, our readings may have been impacted by latent moisture in the concrete.

- At both test pits, we observed a buried timber structure that appears to be the remnants of abandoned concrete formwork (Photo 7). The timber formwork consists of stacked horizontal planks supported by 3 in. by 3 in. vertical posts spaced at 2 ft o.c. At TP-1, the top of the remnant formwork was located at approximately El. 5.7 ft and it extended down to the bottom of the test pit (El. 3.2 ft). At Test Pit No. 2, the top of the remnant formwork was located at approximately El. 0.8 ft. The bottom of TP-2 was located at El. –0.5 ft, and we did not uncover the bottom of the remnant formwork.

- At TP-1, we observed a second timber structure, located parallel to and similar in construction to the abandoned formwork. The function of this abandoned timber wall structure is unknown. We understand from HW that the IRWA and EBSCO believe that it may be part of an abandoned dock. The abandoned timber wall is located in the river approximately 10 +/- ft from the EBSCO Facility and is shown on Fig. 2. We observed timber struts approximately 7 in. by 7 in. by 6 ft long and spaced approximately 6 ft on center connected to the abandoned wall with lapped joints. Based on probing, we estimate that the rip-rap was placed the full width between the formwork and the abandoned timber wall.

3.2 Exploratory Soil Test Borings

SGH performed a subsurface investigation at the project site on 24 August 2016. The subsurface investigation consisted of three soil test borings (Soil Test Borings SGH-2016-1 to SGH-2016-3) and installation of two groundwater observation wells (at Soil Test Borings SGH-2016-2 and SGH-2016-3). Drilling of the soil test borings and installation of observation wells were performed by Carr-Dee Corporation (Carr-Dee), of Medford, Massachusetts, under SGH supervision.

3.2.1 Soil Test Borings

Soil test boring locations are shown in Fig. 1. Two soil test borings were drilled on Saltonstall Street (Photo 8) and Estes Street, adjacent to the EBSCO campus; and one soil test boring was drilled in the gravel driveway located on the town right of way access to the dam adjacent to the private residence located at 69 South Main Street. A representative from SGH was present throughout the field work to observe drilling, assist in obtaining samples, and prepare a descriptive log of each test boring. The sampling intervals, soil descriptions, Standard Penetration Test (SPT) blowcounts, and other pertinent field data are summarized in the individual soil boring logs included in Appendix D.

All soil test borings were drilled using a truck-mounted drill rig. A 4 in. inside diameter hollow stem auger was used for drilling. Soil samples were obtained using 2 in. O.D. split spoon samplers driven using a 140 lb hammer falling 30 in. with a rope cathead.

Soil test borings were extended to refusal of the hollow stem auger to depths ranging between 10.5 ft bgs and 16.5 ft bgs. Typically, continuous SPT samples were obtained from ground surface to the end of the boring. We did not encounter groundwater in Soil Test Boring

SGH-2016-1 (Saltonstall Street) and SGH-2016-2 (Estes Street). All samples were secured, sealed, and transported to the SGH office.

3.2.2 Groundwater Observation Wells

Carr-Dee installed two groundwater observation wells at Soil Test Borings SGH-2016-2 (Estes Street) and SGH-2016-3 (S. Main Street). The observation wells extended approximately to the bottom of the borings, to a depth of 10.5 ft bgs and 16.5 ft bgs respectively. Observation wells consist of 2 in. nominal diameter PVC pipe with 10 ft of slotted screen. The slotted screen is surrounded by filter sand that extends to 4 ft above the top of the slotted screen, with the exception of SGH-2016-2, which was backfilled with cuttings at 4 ft bgs and above. The remainder of the annular space around the PVC pipe was backfilled with soil cuttings and a well cover was set in grout around the top of the PVC pipe, flush with the surrounding grade. Individual observation well installation logs are included in the soil test boring logs in Appendix D.

On 24 August 2016, SGH installed data loggers in Observation Wells SGH-2016-2 (Estes Street) and SGH-2016-3 (S. Main Street) to obtain a continuous record of groundwater levels over time. We installed a Levelogger in each observation well, SGH-2016-2 (Estes Street) and SGH-2016-3 (S. Main Street), to record groundwater levels. We installed a Barologger Edge in Observation Well SGH-2016-2 to record local atmospheric pressure. All devices are manufactured by Solinst Canada Ltd. We made an additional site visit on 11 October 2016 to collect groundwater data and remove the dataloggers.

During installation of the data loggers, SGH obtained initial groundwater depth measurements with a manual Solinst water level meter. The manual measurement at Observation Well SGH-2016-3 (S. Main Street), obtained a few hours after well installation, shows the groundwater level at about 11.75 ft bgs, i.e., at about El. 2.25 ft. We did not observe groundwater within SGH-2016-2 (Estes Street) during site visits on 24 August and 11 October 2016, nor did we detect groundwater with the Levelogger during the monitoring period at this well.

4. ANALYSIS

4.1 Subsurface Conditions

Site subsurface conditions vary depending on proximity to the Ipswich River. We prepared a subsurface profile along the EBSCO Facility riverfront foundation wall based on the results of the soil test borings performed by SGH and others, our test pit investigation, and the bathymetric survey performed by others (Fig. 5). We summarize the strata at the site as follows:

- **Stratum 1 – Fill:** This stratum consists of 3 to 10 ft of fill consisting of very loose to very dense, light to dark brown, dry to wet, sand and silt to sandy gravel, fine to coarse, well to poorly graded, subangular, with trace silt. SPT blow counts range from 2 to +50 blows per foot (bpf). This stratum was encountered at Soil Test Borings SGH-2016-1 through SGH-2016-3 and Soil Test Borings B-2, B-3, and B-4, which were performed by others in 2009 at the south end of the EBSCO Facility. Some organics within the fill were encountered at Soil Test Borings B-3 and B-4 located near the river.

- **Stratum 2 – Sand:** This stratum consists of 2 to 4.5 ft of medium dense to very dense, light orange brown, dry, silty sand, fine to coarse grained, uniform to well-graded, subangular. SPT blow counts ranged from 42 to 53 bpf. This stratum was encountered at Soil Test Borings SGH-2016-1 and SGH-2016-2, underlying the Fill stratum.

- **Stratum 3 – Clay and Organic Silt:** This stratum consists of 5 to 10 ft of very soft to firm, grey to black, wet, clay with trace sand to sand and silt with organics. SPT blow counts ranged from 2 to 8 bpf. This stratum was encountered at Soil Test Borings B-3 and B-4, underlying the Fill stratum.

- **Stratum 4 – Glacial Till:** This stratum consists of 3 to more than 11 ft of medium dense to very dense, light reddish brown to olive grey, dry to wet, silty clay to sandy gravel, fine to coarse, well to poorly graded, subangular, with trace silt. SPT blow counts ranged from 23 bpf to refusal. This stratum was encountered at Soil Test Borings SGH-2016-2 and SGH-2016-3, underlying the Sand stratum, at Soil Test Borings SGH-2016-1, underlying the Granular Fill stratum, at Soil Test Boring B-2, underlying the Fill stratum, and at Soil Test Borings B-3 and B-4, underlying the Clay and Organic Silt stratum.

- **Stratum 5 – Rock Ledge:** The elevation of the top of the rock ledge varies at the site and Fig. 5 is an initial estimate of rock elevations based on limited information. The test pit investigation and the bathymetric survey results indicate that rock ledge varies between approximately El. 3.0 ft and El. 7.5 ft near the dam. We did not perform rock cores for the current study.

4.2 Groundwater Conditions

We plotted the groundwater levels measured at Observation Well SGH-2016-3 and daily local precipitation data for the monitoring period between 24 August and 11 October 2016 (Fig. 6). We obtained the daily local precipitation data from the Plum Island Ecosystems Long Term Ecological Research Network (PIE LTER) for its field station in Rowley, Massachusetts.

At the time of drilling, we encountered groundwater only at Soil Test Boring SGH-2016-3, at a depth of 11.75 ft bgs. During the monitoring period, groundwater levels at Observation Well SGH-2016-3 varied between El. 5.9 ft and El. 7.0 ft (i.e., between 7 ft and 8.1 ft bgs). We did not observe groundwater within Observation Well SGH-2016-2 during site visits on 24 August and 11 October 2016, and we did not detect groundwater with the logger installed at this observation well (the logger was located near the bottom of the observation well at El. 8.5 ft).

4.3 Stability of Riverfront Foundation Wall

We performed calculations to evaluate the stability of the EBSCO Facility riverfront foundation wall against sliding and overturning under future conditions. We used the following parameters for the calculations:

- **Load Conditions**: We considered two load cases.
  - Load Case 1 – The first case considers the lowest estimated impoundment level post-dam removal (water level at El. 3 ft) and groundwater elevation behind the foundation wall also at El. 3 ft.
  - Load Case 2 – The second case considers a flood condition (water level on the building side at El. 11.4 ft and impoundment level at El. 3.0 ft). This represents an extreme condition that may occur immediately post-flood and assumes that the floodwater on the first floor drains out the doors, leaving the groundwater level close to the slab elevation on the building side of the foundation wall (which may occur short term with limited subsurface drainage), while the river level in front of the wall has receded to its lowest estimated normal water level post-dam removal.
  - For both load cases, we assumed a 100 psf uniform surcharge representing the combined load from the weight of the ground floor slab and the surface live load inside the building.

  We did not include live load from the first floor, second floor, and the roof. These live loads do not induce any lateral pressure on the foundation wall; they will only increase the vertical load acting on the foundation wall, thereby increasing the factor of safety for sliding and overturning.

  Wind loading in the east-to-west direction is assumed to be carried by the roof and floor diaphragms to the end walls. The main wind forces on the building do not load the riverfront foundation wall. Only the local wind suction will load the exposed face with wind in the west-to-east direction; this has not been included in our evaluation.

Given the age of the buildings, we anticipate that they were not originally designed for seismic loading; therefore, we did not evaluate this condition.

- **Soil Backfill:** We used a soil friction angle of 30° and a wet unit weight of 120 pcf. These parameters are consistent with loose to medium dense mixed-fill conditions. We evaluated both at-rest and active soil pressure coefficient based on Rankine theory for a conservative estimate of soil loading on the foundation wall.

- **Base of Foundation Wall to Rock Interface Friction:** We used an interface friction angle of 35° between the concrete foundation wall and rock and we assumed the interface to be horizontal.

- **Foundation Wall:** The wall section used in our analysis is shown in Figure 3, consistent with our field observations. We used a unit weight of 150 pcf for concrete material. We assumed that the wall is 1.5 ft thick at the top and about 3 ft wide at the bottom. We assumed that the retained-soil side of the wall is vertical. We estimated dead loads from the brick masonry bearing wall and floor framing above assuming that the interior columns are located approximately 28 ft from the wall. We do not have floor plans or interior measurements, and we estimated the building dead loads using building material densities from ASCE 7-10. Based on our calculations, we estimated a dead load of 3,115 lbs/ft for the building and 2,835 lbs/ft for the foundation wall.

- We assumed that the EBSCO facility floor slab provides no restraint at the top of the riverfront foundation wall (this is a conservative estimate that assumes that the slab does not contribute to the stability of the wall).

The following table summarizes the results of our stability analysis of the retaining wall:

Table 1 – Riverfront Foundation Wall – Stability Analysis Results

Condition	Minimum Factor of Safety(1)(2) against Overturning	Minimum Factor of Safety(1)(3) against Sliding
Lowest Estimated Impoundment Level Post-Dam Removal (WL = 3.0 ft)	2.5	2.5
Flood Condition (WL = El. 11.4 ft)	1.2	1.2

Notes:
(1) The factor of safety is a measure of how much capacity a system has beyond that needed to resist an applied load. The factor of safety is calculated by dividing the estimated system capacity (ability to resist loads) by the estimated applied load (demand).
(2) Typical design factors of safety for overturning of retaining walls are 2.0 for static normal operational loading conditions and 1.1 for extreme loading conditions.
(3) Typical design factors of safety for sliding of retaining walls are 1.5 for static normal operational loading conditions and 1.1 for extreme loading conditions.

4.4 Settlement of Compressible Soils

We did not encounter compressible soils within our borings performed in 2016. For our settlement analysis, we assumed a soil profile similar to the conditions encountered at Test Boring B-3 performed by others in 2009 and located on the south side of the EBSCO Facility. We assumed that the soil profile consists of a 5 ft fill stratum overlying a 6 ft thick clay stratum overlying a 4 ft thick organic silt stratum. The boring log prepared by others identifies the stratum underlying the clay as peat; however, we assumed the stratum is organic silt based on the reported soil description.

Since the boring log does not indicate that laboratory testing was performed (i.e., no consolidation testing, moisture content, or Atterberg limits), we estimated average soil properties for the clay and organic silt strata based on general published values.[1]

We estimated the potential settlement of compressible soils due to primary consolidation imposed by an increase in effective stress due to lowered groundwater levels. We did not include secondary compression of the organic soils in our calculations; if organic soils are indeed present under the EBSCO Facility, secondary compression is occurring and will continue to occur regardless of whether groundwater levels are lowered or not.

On 24 August 2016, we observed the impoundment level at El. 6.2 ft. The results of our groundwater monitoring in observation well SGH-2016-3 show that the groundwater level was approximately at El. 6 ft for the period of record (24 August 2016 to 11 October 2016). Based on our observations of the river staff gauge during our investigation in August 2016, and groundwater data collected at one observation well, we assumed for our settlement analysis that the groundwater levels near the impoundment are similar to the impoundment level. During our investigation, the impoundment was lowered to El. 6.5 +/- ft for a period of approximately one month. Therefore, assuming that the overburden soils under the EBSCO Facility have experienced groundwater levels as low as El. 6 ft, we calculated the range of potential settlement of the clay and organic soils resulting from a 1, 2, and 3 ft drop in groundwater levels. We understand that H&W has not yet completed the hydraulic study of post-dam-removal river levels; however, the preliminary estimated lower-bound water level after the proposed dam removal is at approximately El. 3 ft, and will likely be higher (between El. 3 ft and El. 6 ft). The water level is subject to change pending results from the hydraulic analysis performed by HW.

Table 2 summarizes the soil properties used in our analysis:

[1] We estimated soil properties from published values in *An Introduction to Geotechnical Engineering* by Holtz and Kovacs (1981) and *Geological Background and Engineering Parameters of Boston Blue Clay* by Connors of the University of Massachusetts at Lowell (1993).

Table 2 – Assumed Clay and Organic Silt Strata Consolidation Parameters[1]

Soil Stratum	Boring ID	Depth to Mid-Layer [ft, bgs]	σ'_{vo}[1] [psf]	σ'_{vf}[1] [psf]	Over Consolidation Ratio OCR	Initial Void Ratio e_0	Average C_c[2]
Clay	B-3	8	529	898	1.0	1	0.350
Organic Silt	B-3	13	797	1,166	1.0	4	2.750

1. σ'_{vo} is the estimated existing overburden or in situ vertical effective stress at midlayer (prior to dam removal). σ'_{vf} is the estimated vertical effective stress after dam removal (lowered groundwater level).
2. C_c is the primary consolidation index.

Based on the assumptions listed above, we estimate that the average settlement due to lowering groundwater levels by 1 ft, 2 ft, and 3 ft in the post-dam removal conditions is in the order of 1, 2.5, and 3.5 in., respectively, in those areas where compressible soils are present.

5. DISCUSSION

5.1 EBSCO Facility Riverfront Wall Foundation

As part of the feasibility study, HW will be performing a hydrologic and hydraulic (H&H) analysis of the river and at present has not yet determined anticipated water levels near the EBSCO Facility after the proposed dam removal. We understand that the post-dam-removal river level will vary substantially depending on the river bed elevations and the local climate. HW provided a preliminary estimate that the low river level may be in the range of El. 3 ft to El. 6 ft after dam removal (elevation is subject to change pending the results of the H&H analysis).

Table 3 below summarizes elevations pertinent to the EBSCO Facility:

Table 3 – Water Level and EBSCO Foundation Wall Elevations Along the Riverfront Foundation Wall

Description	Elevation [ft, NAVD 88]
Top of Foundation Wall at Building No. 10-A.	12.5
Maximum Elevation of Bottom of Foundation Wall at Building No. 10-A / Bottom of Test Pit No. 2 (TP-2)	–0.5
Approximate Top of Glacial Till Stratum at South End of Building No. 10A	-5.0
Top of Foundation Wall at Building No. 9.	11.4
Apparent Bottom of Foundation Wall at Building No. 9 / Bottom of Test Pit No. 1 (TP-1)	3.2
Estimated Low River Level Elevation After Dam Removal (Preliminary Estimate from HW)	3 to 6
Dam Crest	8.9
Average Elevation of Rock Ledge at Toe of Dam	2.9
Top of Abandoned Timber Formwork and Abandoned Timber Wall	5.7

Based on the information collected to date, we expect that the exterior concrete foundation wall of the EBSCO Facility, which abuts the Ipswich River (referred to as the riverfront foundation wall in this report), will not be significantly impacted by the expected water-level drawdown due to removal of the dam.

The riverfront foundation wall of the EBSCO Facility appears to be bearing on rock at the north end (Building No. 9). The elevation of rock ledge detected at TP-1 (El. 3.2 ft) is consistent with the average elevation reported at the toe of the dam (El. 2.9 ft), and the low points of the river transects near TP-1 (El. 0.5 ft to El. 1 ft).

At the south end of the EBSCO Facility (Building No. 10-A), the riverfront foundation concrete wall extends below El. –0.5 ft, which is about 8.4 ft below the current normal river pool elevation. We note that the top of the glacial till stratum was encountered at about El. -5 ft in the borings performed by others at the south end of the EBSCO Facility. That is, the bottom of the riverfront foundation wall may be as much as 4.5 ft above the top of the glacial till stratum. Since it is not possible to drive timber piles any significant distance into very dense glacial till, and given the depth to which excavation was performed to construct the wall (deeper than 10 ft), it is unlikely that the riverfront foundation wall is supported on timber piles. However, even if the riverfront foundation wall at Building No. 10-A was supported on timber piles, with pile tops below El. –0.5 ft it is unlikely that the tops of the timber piles will be adversely impacted by lowered water levels after dam removal as the anticipated minimum water level due to dam removal is currently estimated to be at El. 3 ft to El. 6 ft, which is at least 3.5 ft above where the tops of the timber piles could be, if present.

The current study was limited to test pit investigations adjacent to the EBSCO Facility riverfront foundation wall at the north end (Building No. 9 constructed in 1908) and the south end (Building No. 10-A constructed in 1912). Based on visual observations of the riverfront wall and assuming the rock ledge elevations are similar along the length of the riverfront wall, the foundation construction is likely to be similar at other locations along this wall of the EBSCO Facility. However, we did not observe the foundations supporting interior walls or columns of Buildings No. 9 and 10A or the other buildings on the EBSCO campus (Building Nos. 10, 11, and 11A constructed in 1901, 1918, and 1946 respectively). Any portion of the EBSCO Facility supported on timber piles with pile top cutoffs located above the currently anticipated lowered river level (El. 3 ft to El. 6 ft), should they exist, would be subject to timber pile deterioration.

5.1.1 Mitigation of Potential Deteriorating Timber Piles

Although we did not observe timber piles supporting the EBSCO Facility riverfront foundation during our investigation, if timber piles with high cutoff elevations (i.e., top of piles above estimated lowest groundwater level) were to be present supporting interior walls, interior columns, or foundation walls for the buildings not investigated during this study, mitigation options include:

- Replace existing timber pile foundations with new micropile foundations. Drilled-in micropile foundations would be installed around the bearing walls and/or columns, extending down into the glacial till/rock strata underlying the site. Special structural connection brackets or a system of needle beams would be required to transfer the load from the bearing walls and/or columns to the micropiles. Replacing existing timber piles with new deep foundation elements represents a significant disruption to the building facility's operations, has a long construction schedule, and is very expensive compared to the more typically used cut-and-post underpinning approach described below.

- Perform cut-and-post underpinning repairs of existing timber pile foundations. Cut-and-post underpinning involves removing the top portions of the timber piles that are exposed above groundwater and replacing them with new concrete-encased steel posts. The cost of this repair is typically driven by the labor for excavating pits to access the existing foundation elements. The labor required for excavation is in turn impacted by site access and dewatering operations, among other factors. The repair would involve removing and replacing finishes around the foundation elements, excavating soils and temporarily storing/stockpiling excavated material, dewatering the access pit, mining underneath the pile cap foundation to expose the timber piles, providing temporary shoring for the existing foundation, removing the tops of all timber piles, installing concrete-filled steel posts in place of the removed timber pile tops, placing the concrete encasement around the steel posts, and backfilling the excavation. Typically a portion of the soil spoils need to be hauled and disposed of off site. Cut-and-post underpinning is typically the most effective solution to repair deteriorating timber pile tops due to lowered groundwater levels in an existing occupied structure.

- Install a groundwater recharge system to artificially raise groundwater levels to preserve the timber piles. Groundwater recharge involves installing wells and trenches to inject water into the ground to artificially raise water levels locally, thus submerging the tops of the existing timber piles. A water treatment system is required to remove impurities and biological agents that may be present in the water that could clog the system filters and screens. At this site, installing a groundwater recharge system would also involve installing a below-grade cutoff wall (e.g., jet grout wall) along the riverfront property line, and potentially on each side of the EBSCO Facility perpendicular to the river, to prevent loss of injection water toward the river. Additional engineering studies (e.g., permeability tests, injection well tests) would be required to determine if the groundwater recharge option is feasible for the EBSCO Facility. Given the need for a cutoff wall and groundwater treatment system, this option is bound to be more expensive than cut-and-post underpinning.

If mitigation of deteriorating timber piles is required at the EBSCO Facility, we consider cut-and-post underpinning to be the least costly of the options discussed above. Based on our experience on prior projects involving deteriorating timber piles, we estimate that the direct cost for cut-and-post underpinning repairs, if needed, would be on the order of $700/sq ft of foundation repaired (or $2,900/lf of foundation wall). This order-of-magnitude direct cost may vary greatly depending upon project specifics, including, but not limited to, the existing structure and subsurface conditions, the extent of the area to be repaired, access to repair areas, finishes, and any staging required to maintain building occupancy during the repair work. Also, the order-of-magnitude costs above consider the subcontracted cost, not the burdened cost to the project owner. Additional costs for general conditions, general contractor markup, owner project management, design fees, and contingencies are not included. The total burdened cost to the owner could be on the order of $1,300/sq ft (or $5,200/lf of foundation wall) assuming the following: 10% general conditions, 10% general contractor markup, 10% design fees, 50% contingency.

It is possible to perform movement monitoring of the EBSCO Facility to check the performance of the structure upon dam removal to try to detect the onset of settlement, and thus use the movement monitoring data as an indicator of the presence of timber piles. However, the rate of timber pile deterioration can be highly variable. In our experience, due to the accuracy of conventional survey methods, by the time that movement is detected, building distress, such as cracks, has already developed.

5.2 EBSCO Facility Riverfront Foundation Wall Stability

We analyzed the stability of the EBSCO Facility riverfront foundation wall and determined that the wall has enough reserve capacity to resist unbalanced loading under the proposed dam removal conditions (Table 1) and assumed water levels; our analysis indicates that the calculated factor of safety for both post-dam removal static normal operating conditions and extreme loading conditions is larger than the minimum factor of safety required for design.

Based on the preliminary lower-bound estimate for the low river level (El. 3 to 6 ft post–dam removal), erosion of the river bed could result in undermining of the rip-rap scour protection for any foundations bearing on soil (if any exist) along the EBSCO riverfront foundation wall. Pending further hydraulic analysis, it may be prudent to evaluate the need for replacement of the existing stone rip-rap after removal of the dam to prevent scour and subsequent foundation undermining. We understand that erosion of the rip-rap will be analyzed in future hydraulic studies.

5.3 EBSCO Facility Riverfront Foundation Wall Aesthetic Considerations

The estimated lowered water levels will not affect the structural stability of the foundation wall, but they could impact the visual appearance of the concrete surface.

During the investigation we observed erosion of the concrete foundation wall, likely due to freezing and thawing near the normal water levels (e.g., El. 8 ft to El. 10 ft). We observed less surface erosion and staining below the apparent current normal water levels, indicating that the concrete below the water line (and subjected to fewer freeze-thaw cycles) was somewhat better preserved than the exposed concrete subjected to more freeze-thaw cycles.

An investigation into the condition of the riverfront foundation wall concrete was not included in the scope of work for the current study. Based on the age of the concrete, we anticipate that it is not purposely air entrained, and due to the likely low air void ratio, we expect that previously submerged concrete that becomes exposed to weather due to lowered water levels under post-dam-removal conditions will be subject to some freeze-thaw deterioration, and will undergo erosion similar to the concrete that has already been exposed.

5.4 Settlement of Compressible Organic Soils

Available logs of borings performed by others at the south end of the building, within 50 ft from the Ipswich River, indicate the presence of a 5 ft to 10 ft thick clay and peat stratum. We did not encounter soft compressible soils in the three soil test borings we performed, located 50 ft or more from the river. The thickness of the clay and peat stratum appears to be largest near the river and very likely decreases with distance away from the river. Therefore, we anticipate that these problem soils are present in localized areas primarily adjacent to the river. At this time it is uncertain to what extent, if any, compressible soils may or may not underlie the EBSCO Facility itself. Slabs supported on grade, shallow spread footings bearing above the soft compressible soils, and other shallow structures such as buried utilities overlying soft compressible soils could be at risk of some settlement due to consolidation of the soft soils initiated by lower groundwater levels resulting from the potential lower river level. We expect that where soft compressible soils are present along the river, average settlements in the order of 1 in. to 3.5 in. can be anticipated due to lowering of groundwater levels by 1 ft to 3 ft, respectively.

We observed no signs of settlement of the EBSCO Facility exterior wall along the river; therefore, we anticipate that this foundation wall is bearing on competent soils (likely glacial till) or rock. We also have not observed signs of significant settlement of the slab inside the EBSCO Facility, although some dishing of the slab has been noted, which suggests that some settlement of the slab has taken place in the past. To our knowledge, EBSCO has not reported any issues with settlement of the slab.

It is possible that settlement of soft compressible soils underlying the slab-on-grade, if they exist, could have occurred in the past and been repaired before EBSCO moved into the Facility. Based on the depth of the foundation walls near the south end of the building (TP-2), extending below El. -0.5 ft, it is also possible that a portion or all of the compressible soils that may have been present were removed as part of the excavation to construct the foundation walls. To the extent that soft compressible soils are present within the footprint of the EBSCO Facility, some settlement of the slab-on-grade may occur.

Mitigation measures for settlement of existing slabs-on-grade supported on soft compressible soils fall under two categories:

- Structural Remediation Approach – the goal is to replace the existing slab-on-grade either by 1) removing the soft compressible soils, replacing them with engineered compacted fill, and installing a new slab-on-grade, or 2) removing the existing slab-on-grade and replacing it with a structural slab supported on deep foundations (micropiles) bearing on suitable soils. Both options represent a significant disruption to a building facility's operations, have long construction schedules, and are expensive.

- Ground Improvement Approach – the goal is to reduce the compressibility of the underlying soft soils; usually maintaining the use of the existing slab-on-grade. Options for ground improvement of soft compressible soils for in-service facilities include compaction grouting or jet grouting. Both methods would involve drilling holes through the existing slab at regular intervals, for example on a 5 ft by 5 ft grid, to provide access for a drill rig to install grout columns below grade to provide supplemental support to the slab.

    - Compaction Grouting involves installing low-slump grout in lifts below grade to displace the soil and create columns of grout. More specifically, compaction grouting is performed by inserting 2 in. diameter grout injection pipes through the target weak soil stratum, then pumping low-slump grout under pressure, which forms a bulb of grout and pushes the surrounding soil, thus densifying the soil. After achieving a target pressure or volume of grout, the grout injection pipe is raised to a higher elevation, and another bulb of grout is injected. The process is repeated, extending the grout vertically through the entire weak soil stratum. The spacing of the compaction grout locations is designed by an engineer and typically depends on the subsurface conditions, the use of the building space (loads), and the capacity of the existing or new slab.

    - Jet Grouting involves installing grout (with water and/or air) at a high velocity to erode and mix with the soil to create columns of soil-crete. More specifically, jet grouting is performed by drilling a 6 in. diameter hole into the soil through the target weak soil stratum, then a specialty drill rod with a nozzle is lowered into the hole. The contractor pumps fluid (a mixture of grout, air and/or water) through the nozzle and spins the drill rod to erode a circular area. The drill rod is retracted up through the soil to create a column of soil-crete. Test borings are drilled after jet grouting to confirm strength of as-built soil-crete columns. The spacing of jet grout locations typically depends on the subsurface conditions and the ability for the soil to be eroded by the jet grout process.

Typically the soil-grout or grout columns created during compaction or jet grouting are spaced at about 5 ft on center for the typical 4 to 5 in. thick slab-on-grade, as this is typically the limit of the capacity of the slab-on-grade to resist concentrated loads acting on it. Grouting operations inside in-service facilities require significant coordination to control soil/grout spoils and maintain areas clean for use. Although grouting involves shorter construction schedules and is typically more cost-effective than the structural remediation approach, construction costs can still be high depending on the extent of the area to be treated.

All mitigation measures need to consider the presence of buried utilities under the existing slab-on-grade. Under the structural remediation approach, the buried utilities can be supported from the new structural slab. If soft compressible soils extend outside the building area to be remediated, flexible utility connections would need to be considered at the interface where existing utilities extend beyond the new structural slab or ground improvement area.

If mitigation of settling organic soils is required at the EBSCO Facility, we estimate the direct cost for repairs could be on the order of $750/sq ft, $350/sq ft, and $450/sq ft for the three repair options discussed above (structural slab with micropiles, compaction grouting, and jet grouting respectively). These order-of-magnitude costs may vary greatly depending upon project specifics, including, but not limited to, the existing structure and subsurface conditions, the depth of the micropiles or grout columns, the extent of the area to be repaired, access to repair areas, finishes, and any staging required to maintain the building occupancy during the repair work. We did not consider costs from other trades, such as plumbing for buried subsurface utilities or replacement of interior finishes. Also, the order-of-magnitude costs above consider the subcontracted cost, not the burdened cost to the project owner. Additional costs for general conditions, general contractor markup, owner project management, design fees, and contingencies are not included. The total burdened cost to the project owner could be on the order of $1,500/sq ft, $700/sq ft, and $900/sq ft for the three options above, respectively, assuming the following: 10% general conditions, 10% general contractor markup, 10% design fees, and 50% contingency.

Movement monitoring of the EBSCO Facility slab-on-grade can be performed to detect settlements and thus help identify if there is ongoing settlement of the slab-on-grade due to the presence of soft compressible soils. However, it is possible that some distress to the slab (e.g., cracks) may occur during the movement monitoring program.

6. CONCLUSIONS

We conclude the following regarding the potential impacts of the dam removal on the adjacent EBSCO Facility:

- The riverfront wall foundations of Building Nos. 9 and 10-A of the EBSCO Facility are bearing on rock and/or are bearing on soils or piled foundations at an elevation lower than the currently estimated low water level of the Ipswich River at the site after dam removal (El. 3 ft to El. 6 ft). We did not observe timber piles supporting the EBSCO Facility at these locations and, even if timber piles are present, it is anticipated that the tops of the timber piles are low enough to remain submerged in a post-dam removal scenario, and therefore, fungal deterioration of the tops of the timber piles would not occur.

- Soil test borings performed by others in 2009 indicate the presence of localized soft compressible soils, including organics, along the riverfront. Where organics are present, lowered groundwater levels could result in settlement of pavement, slabs-on-grade, and structures on spread footings or buried utilities supported above the soft compressible soils. We estimate a potential settlement of the soft compressible soils of approximately 1 in., 2.5 in., and 3.5 in. due to a water level drawdown of 1 ft, 2 ft, and 3 ft, respectively (i.e., groundwater level at El. 5 ft, El. 4 ft, and El. 3 ft, respectively). At this time it is uncertain to what extent, if any, compressible soils may or may not underlie the EBSCO Facility. We estimated the settlement assuming average soil properties from a range of published values for organic silt and clay.

- Based on the results of the current investigation, we identify the following three options for the project team to determine next steps in the feasibility study for the Ipswich Dam removal:

  - Option 1 – Maintain Current Groundwater Level During Post-Dam Removal. This option presents the least amount of risk for settlement due to timber pile deterioration or consolidation of compressible soils, if present, at the EBSCO Facility. Groundwater levels measured during our investigation were approximately El. 6 ft, therefore maintaining this groundwater elevation would likely not result in adverse impacts to the EBSCO Facility. Maintenance of current groundwater levels at approximately El. 6 ft would require evaluating appropriate approaches to dam removal or other engineered solutions such as groundwater recharge. Additional subsurface investigation would be required to evaluate the feasibility of applicable engineered solutions. This option also requires continuous monitoring of groundwater levels and structure movement to verify performance after the dam is removed, for the life of the structure.

    If the project team anticipates that the post-dam removal groundwater levels cannot be maintained at or above El. 6 ft, then one of the following two options may be implemented to determine risks to the EBSCO Facility and develop mitigation options if needed.

  - Option 2 – Pre–Dam-Removal Supplemental Subsurface Investigation. This involves completing a supplemental foundation investigation in the building areas that were not accessible during the current investigation. Performing this investigation prior to completing the feasibility study for the dam removal would provide actionable information to perform a better assessment of the likelihood of the need for mitigation options, as it would allow the project team to identify whether timber piles are present in the remaining areas of the EBSCO Facility where test pits have not been performed, and would also allow us to determine if soft compressible soils are present within the footprint of the EBSCO Facility. We consider that this option lowers the risk of adverse impacts from dam removal as it allows for timely planning and budgeting for mitigation, if needed, during the initial design phases of the project. The extent of post–dam-removal movement monitoring required to confirm adequate performance of the building would be determined based on the results of the supplemental subsurface investigation.

An outline of the recommended supplemental investigation is included in Appendix A. We estimate that the order-of-magnitude cost for the supplemental investigation as outlined would be approximately $200,000, assuming adequate access for the investigation, minor dewatering required for test pits, and replacement of the concrete slab and asphalt pavement cut penetrations.

- If EBSCO does not provide access to the inside of its facility and access for test pit investigations on the exterior of the facility, then a limited soil test boring investigation could be performed on the building exterior. The limited investigation would include five to ten soil test borings drilled in the EBSCO Facility parking lot and other exterior areas near the building, such as the grassed area at the south end of the building. The soil test borings would provide some subsurface information for the EBSCO Facility site and allow the project team to further evaluate the potential risks due to the potential presence of compressible soils and timber piles, if any are deemed to be present. We estimate that the order-of-magnitude cost for the limited supplemental investigation would be approximately $50,000.

- Option 3 – Perform Pre– and Post–Dam-Removal Precision Movement Monitoring, No Supplemental Subsurface Investigation. We understand a staged drawdown test in combination with precision movement monitoring could be performed for an extended period of time prior to dam removal. The pre–dam-removal precision movement monitoring would help establish a baseline against which to compare post–dam-removal performance.

  Under this option, planning for the dam removal project would proceed without further information about the foundations in building areas outside the current study, and also without further information regarding the presence of soft compressible soils within the EBSCO Facility. The project team would rely solely on pre– and post–dam-removal precision movement monitoring to assess the building performance and determine if mitigation measures are required. Precision movement monitoring helps identify problem areas; however, limits to accuracy, access, and duration of monitoring make this a more reactive approach compared to the other options. We consider that this option results in a higher risk of potential unmitigated settlement of the building because some distress to the building utilities, adjacent structure, and/or slab-on-grade may occur before the post–dam-removal precision monitoring program detects measurable movement. In addition, there is a higher risk of significantly underestimating or overestimating the costs of mitigation. We note that if post–dam-removal mitigation measures are required, the costs are more likely to be higher than had mitigation been performed pre–dam-removal, as the costs of repairs of any building distress (cracks, unlevelness, etc.) would need to be included. This also requires access to the interior of the EBSCO Facility buildings to install monitoring points and during each round of survey of the monitoring points over an extended period of time.

Limitations of Current Investigation

The information presented herein is based on the geotechnical information collected to date. The boring logs and geotechnical investigation records depict subsurface conditions only at the specific soil sampling locations. Subsurface conditions at other locations may differ from conditions observed at specific sample depths and exploration locations. There is no warranty or guarantee, either expressed or implied, that the conditions indicated by such investigations or records thereof are representative of those existing throughout such areas, or any part thereof, or that unexpected developments may not occur, or that materials other than, or in proportions different from, those indicated may not be encountered.

I:\BOS\Projects\2016\160630.00-DAMR\WP\001rWPKonicki-R-160630.00.ras.docx

Photo 1

View of EBSCO building on 22 August prior to the temporary impoundment drawdown.

Photo 2

View of EBSCO building on 24 August after the temporary impoundment drawdown.

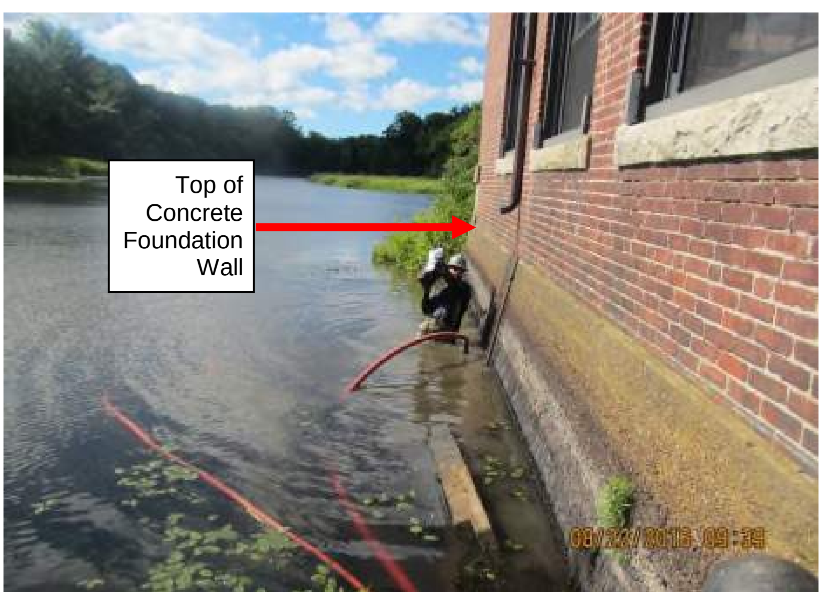


Photo 3

Diving contractor removing rip-rap by hand at TP-1, adjacent to Building No. 9.

Photo 4

Typical rip rap and debris removed from TP-1.

Photo 5

View of riverfront foundation wall at TP-1 (Building No. 9) on 24 August, looking south. Staining and concrete erosion is apparent below-normal water level.

Photo 6

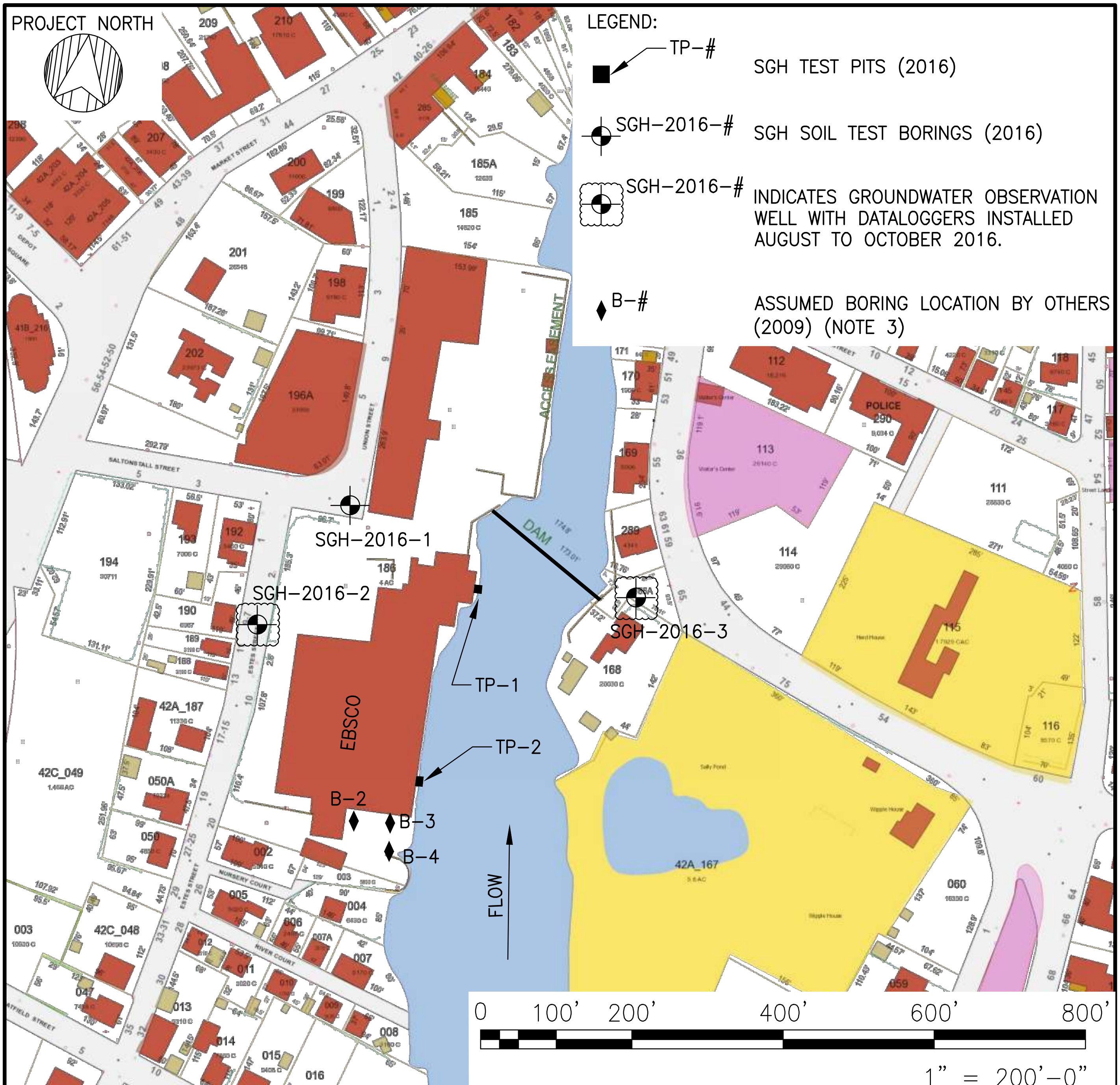
View of riverfront foundation wall at TP-2 (Building No. 10-A) on 23 August, looking north. Staining and concrete erosion is apparent below-normal water level.

Photo 7

Abandoned timber formwork removed from TP-1.

Photo 8

Drill rig set up at Soil Test Boring SGH-2016-1, located approximately 100 ft from EBSCO Building No. 9. EBSCO Building No. 3 on the right hand side.



NOTES:

1. BASE PLAN OBTAINED FROM MASS GIS ONLINE VIEWER.
2. SGH TEST PIT AND BORING LOCATIONS ARE APPROXIMATE.
3. LOCATIONS OF THE BORINGS PERFORMED BY OTHERS IN 2009 ARE ESTIMATED FROM THE MEMORANDUM FROM GEI TO THE TOWN OF IPSWICH RE: EVALUATION OF POTENTIAL IMPACTS ON EBSCO BUILDINGS FROM PROPOSED REMOVAL OF IPSWICH MILLS DAM DATED 13 DECEMBER 2013 REVISED 14 FEBRUARY 2014.

SIMPSON GUMPERTZ & HEGER
Engineering of Structures and Building Enclosures
Simpson Gumpertz & Heger Inc.
41 Seyon Street, Building 1, Suite 500
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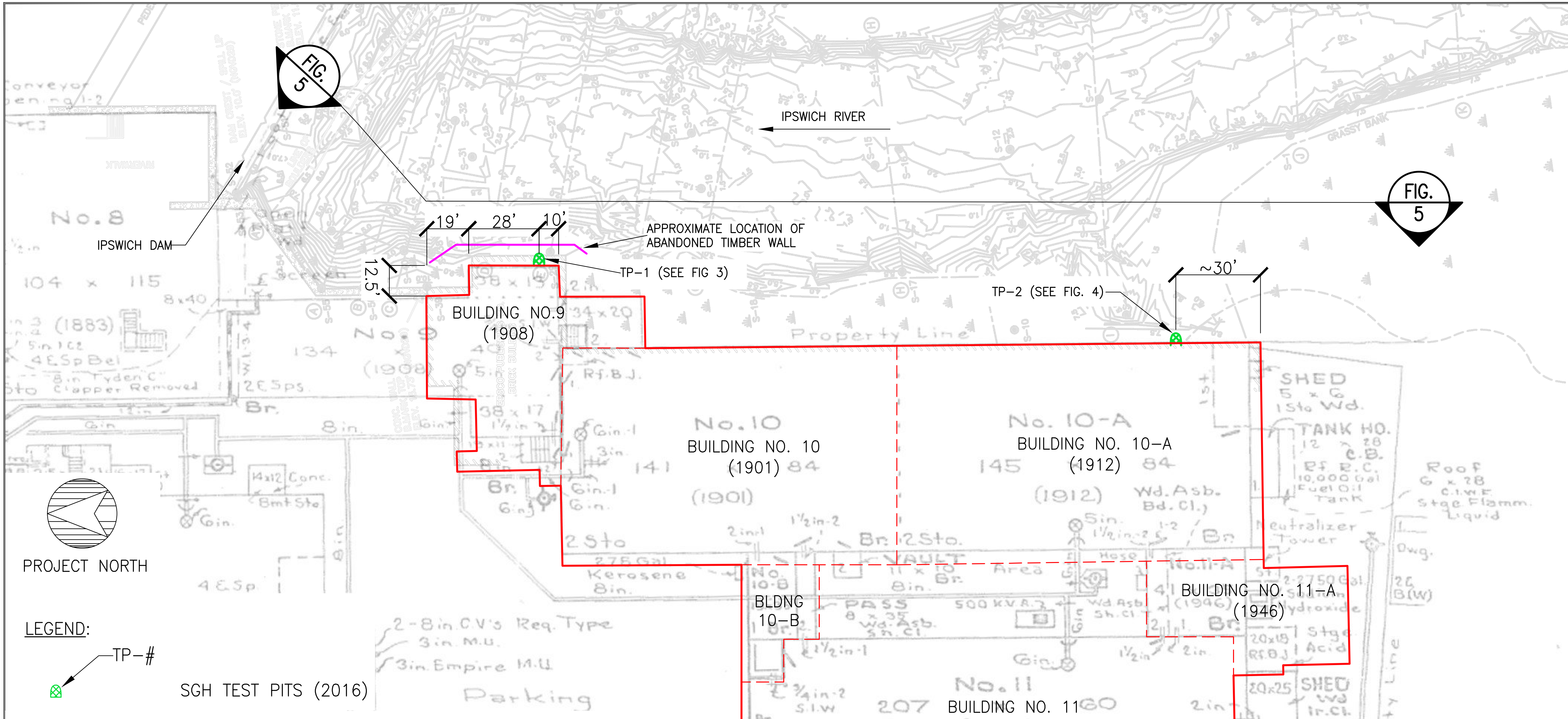
781.907.9000
fax: 781.907.9009
www.sgh.com

Project: FEASIBILITY STUDY FOR IPSWICH MILLS DAM REMOVAL IPSWICH, MA			
Title: SUBSURFACE INVESTIGATION LOCATION PLAN			
Drawn: NDL	Checked: SFK	Approved:	Project No.: 160630.00

Drawing No.:	FIG. 1
Scale:	1" = 200'
Date:	10/14/16



LEGEND:

TP-#

SGH TEST PITS (2016)

NOTES:

1. HISTORIC EBSCO BUILDING BASE PLAN OBTAINED FROM THE REPORT TITLED "IPSWICH MILLS DAM PARTIAL FEASIBILITY STUDY" DATED 23 APRIL 2014. BASE PLAN SHOWS YEAR BUILT OF HISTORIC EBSCO BUILDING SECTIONS.
2. BASE PLAN SHOWING RIVER BATHYMETRY OBTAINED FROM REPORT TITLED "IPSWICH RIVER MILLS DAM SURVEY" PREPARED BY NORDE-EAST SURVEY DATED 26 AUGUST 2014.
3. OUTLINE OF CURRENT EBSCO BUILDING (IN RED) IS APPROXIMATED FROM MASS GIS ONLINE VIEWER AND LIMITED FIELD MEASUREMENTS NEAR TEST PITS.
4. DASHED LINES INDICATING EXTENTS OF HISTORIC EBSCO BUILDING SECTIONS ARE APPROXIMATE.

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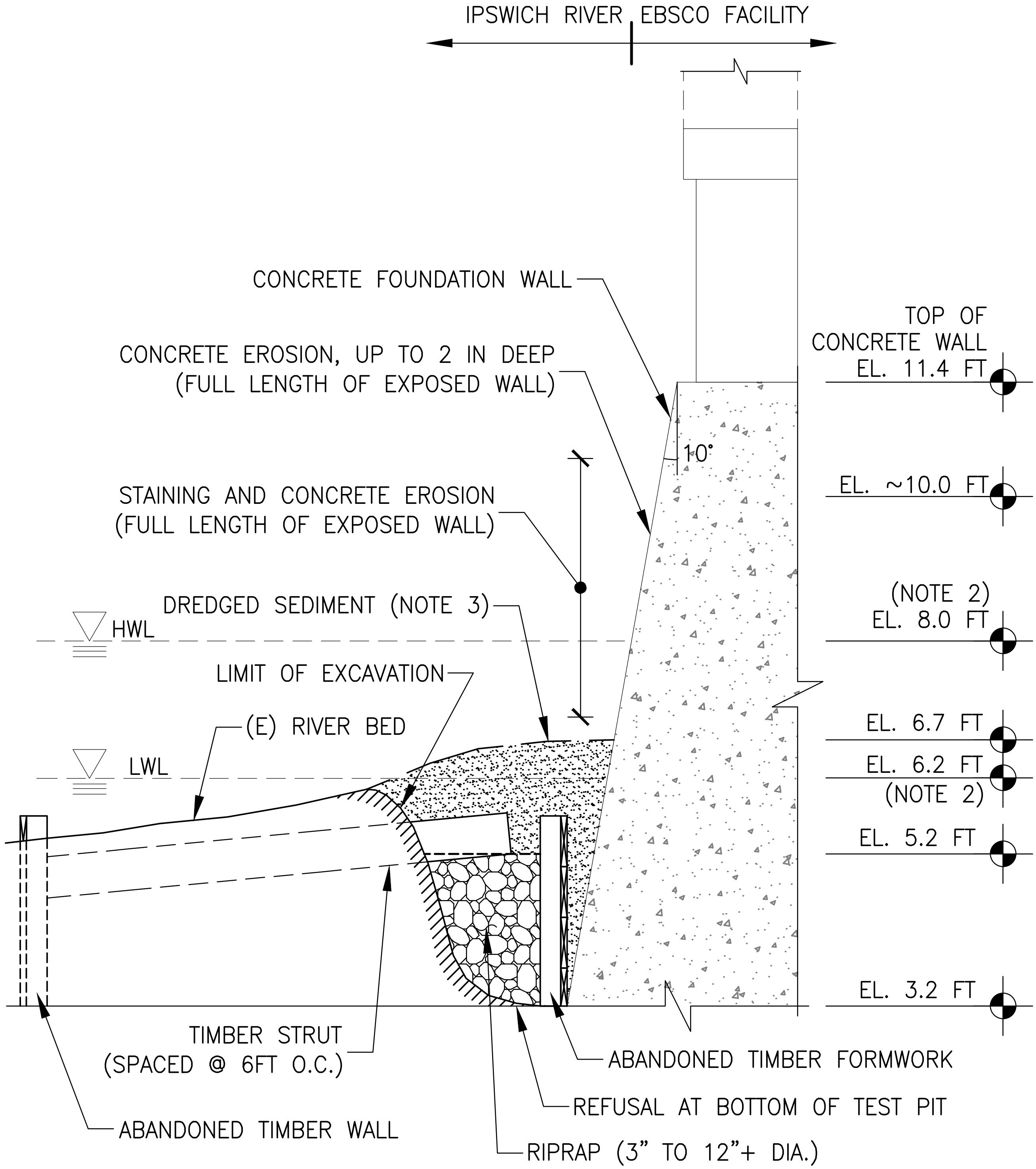
Project: FEASIBILITY STUDY FOR IPSWICH MILLS DAM REMOVAL IPSWICH, MA				Drawing No.: FIG. 2
Title: TEST PIT LOCATION PLAN				Scale: 1"=40'-0"
Drawn: NDL	Checked: SFK	Approved:	Project No.: 160630.00	Date: 02/10/17

NOTES:

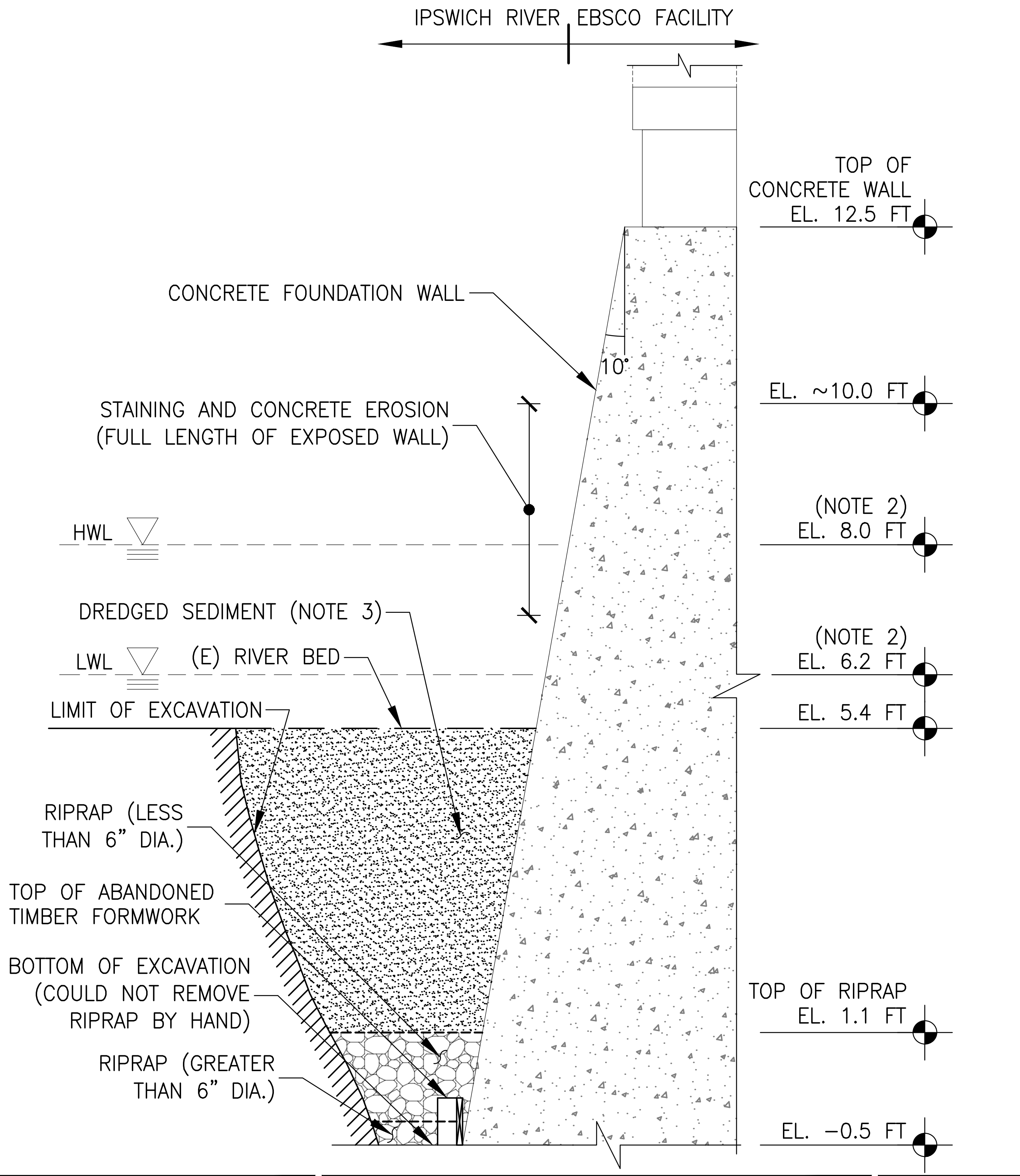
1. ALL ELEVATIONS ARE APPROXIMATE AND REPORTED IN FEET REFERENCED TO NAVD88.
2. IMPOUNDMENT ELEVATIONS ESTIMATED FROM STAFF GAUGE NEAR DAM. HIGH WATER LEVEL (HWL) MEASURED ON 22 AUGUST 2016 PRIOR TO BEGINNING TEST DRAWDOWN. LOW WATER LEVEL (LWL) MEASURED ON 24 AUGUST 2016 AT 3PM.
3. SEDIMENT CONSISTS OF SILT; DARK BROWN; VERY SOFT; TRACE DEBRIS.



SIMPSON GUMPERTZ & HEGER Engineering of Structures and Building Enclosures Simpson Gumpertz & Heger Inc. 781.907.9000 41 Seyon Street, Building 1, Suite 500 fax: 781.907.9009 Waltham, Massachusetts 02453 www.sgh.com	Project: FEASIBILITY STUDY FOR IPSWICH MILLS DAM REMOVAL IPSWICH, MA	Drawing No.: FIG. 3
	Title: SECTION VIEW, TP-1	Scale: 1/2"=1'-0"
	Drawn: NDL · Checked: SFK · Approved: · Project No.: 160630.00	Date: 02/10/17

NOTES:

1. ALL ELEVATIONS ARE APPROXIMATE AND REPORTED IN FEET REFERENCED TO NAVD88.
2. IMPOUNDMENT ELEVATIONS ESTIMATED FROM STAFF GAUGE NEAR DAM. HIGH WATER LEVEL (HWL) MEASURED ON 22 AUGUST 2016 PRIOR TO BEGINNING TEST DRAWDOWN. LOW WATER LEVEL (LWL) MEASURED ON 24 AUGUST 2016 AT 3PM.
3. SEDIMENT CONSISTS OF SILT; DARK BROWN; VERY SOFT; SOME ORGANICS; SOME CLAY; TRACE DEBRIS.



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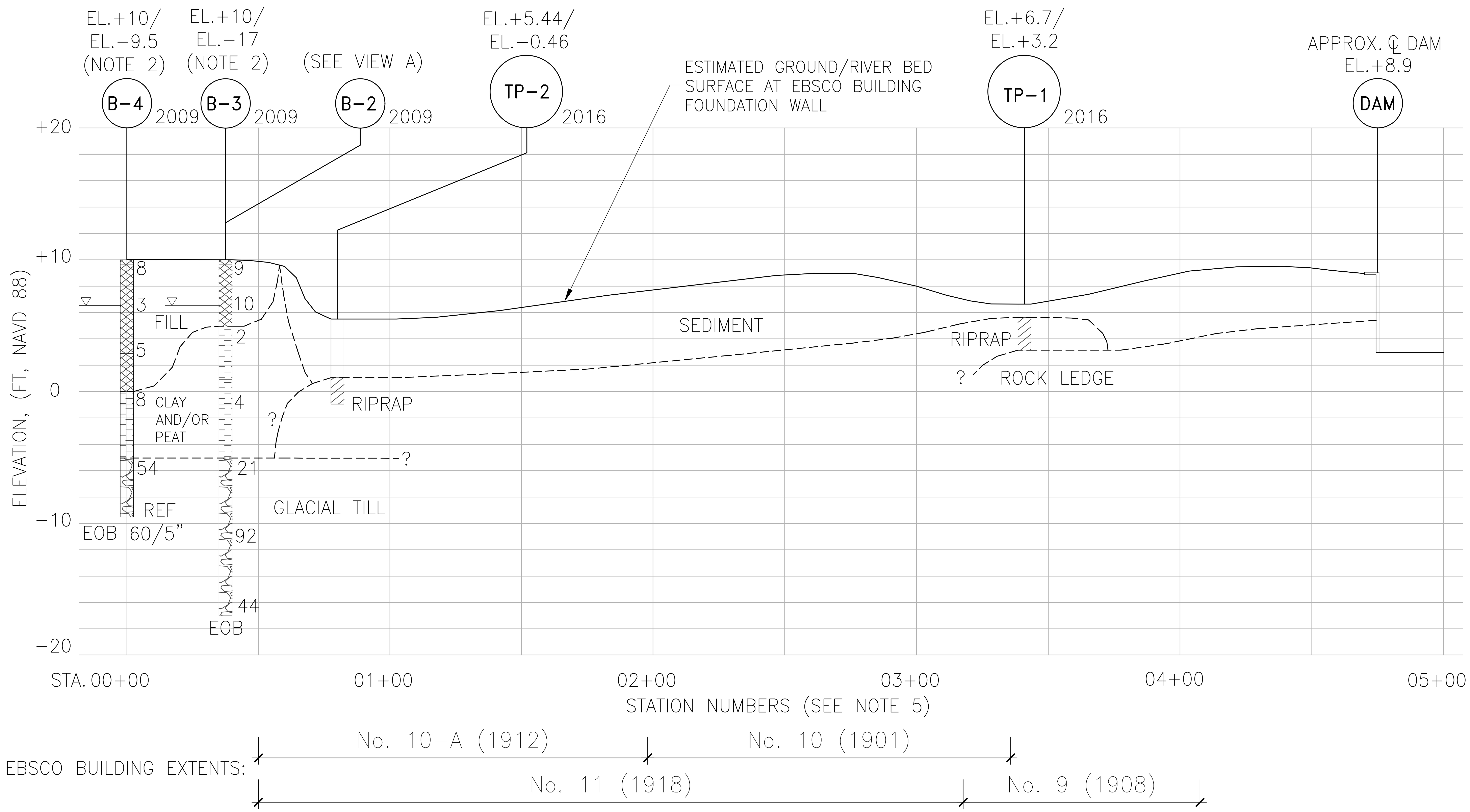
Project: FEASIBILITY STUDY FOR IPSWICH MILLS DAM REMOVAL IPSWICH, MA

Drawing No.: FIG. 4

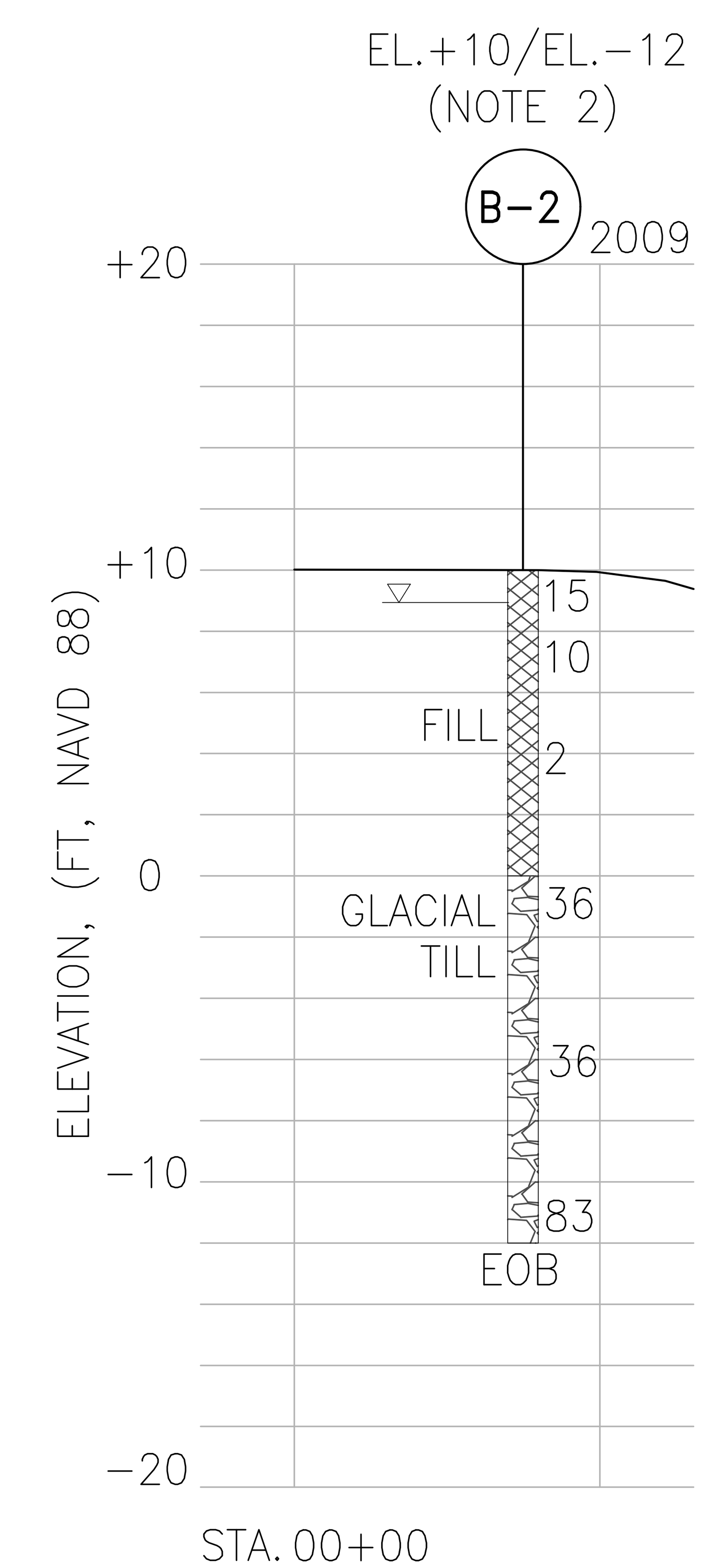
Title: SECTION VIEW, TP-2

Scale: 1/2"=1'-0"

Drawn:	Checked:	Approved:	Project No.:	Date:
NDL	SFK		160630.00	11/22/16



SECTION AT EBSCO BUILDING RIVERFRONT FOUNDATION WALL



VIEW A

NOTES:

1. ALL ELEVATIONS ARE REFERENCED TO THE NAVD88 DATUM U.O.N.
2. THE GROUND SURFACE ELEVATIONS FOR 2009 BORINGS (B2, B3, B4) ARE NOT REPORTED. WE ASSUMED THE GROUND SURFACE IS EL.+10 FT NAVD88, WHICH WE ESTIMATED BASED ON THE BATHYMETRIC SURVEY PERFORMED BY NORDE-EAST ON 26 AUGUST 2014 AND VISUAL OBSERVATIONS NEAR TP-2 IN AUGUST 2016.
3. SOIL STRATA SHOWN ARE GENERALIZED PROFILES INTERPRETED FROM BORING LOGS PREPARED BY SGH AND OTHERS. REFER TO SOURCE BORING LOGS FOR MORE DETAILED SOIL DESCRIPTIONS.
4. STRATIFICATION LINES REPRESENT APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES; TRANSITIONS MAY BE GRADUAL.
5. STATION NUMBERS ALONG THE RIVERFRONT WALL OF THE EBSCO BUILDING ASSUME STATION 00+00 IS LOCATED AT BORING B-4. SEE FIGURE 2 FOR SECTION ALIGNMENT.

LEGEND

#
-OFFSET (FT)
-TOP OF BORING OR TEST PIT ELEVATION (FT)/
-BOTTOM OF BORING OR TEST PIT ELEVATION (FT)

WATER LEVEL DURING BORING ▽

#'=ELEVATION OF STRATUM CHANGE
##=N VALUE, BLOWS PER FT (BPF)
REF 100/2" = REFUSAL, BLOWS/PEN.
EOB=END OF BORING

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Boston
Chicago
Los Angeles
New York
San Francisco
Washington, DC

Consultant

No.	Date	Description	By
1	6/26/17	REPORT REVISION	SFK

FEASIBILITY STUDY FOR IPSWICH MILLS DAM REMOVAL
IPSWICH, MA

Project

SUBSURFACE PROFILE

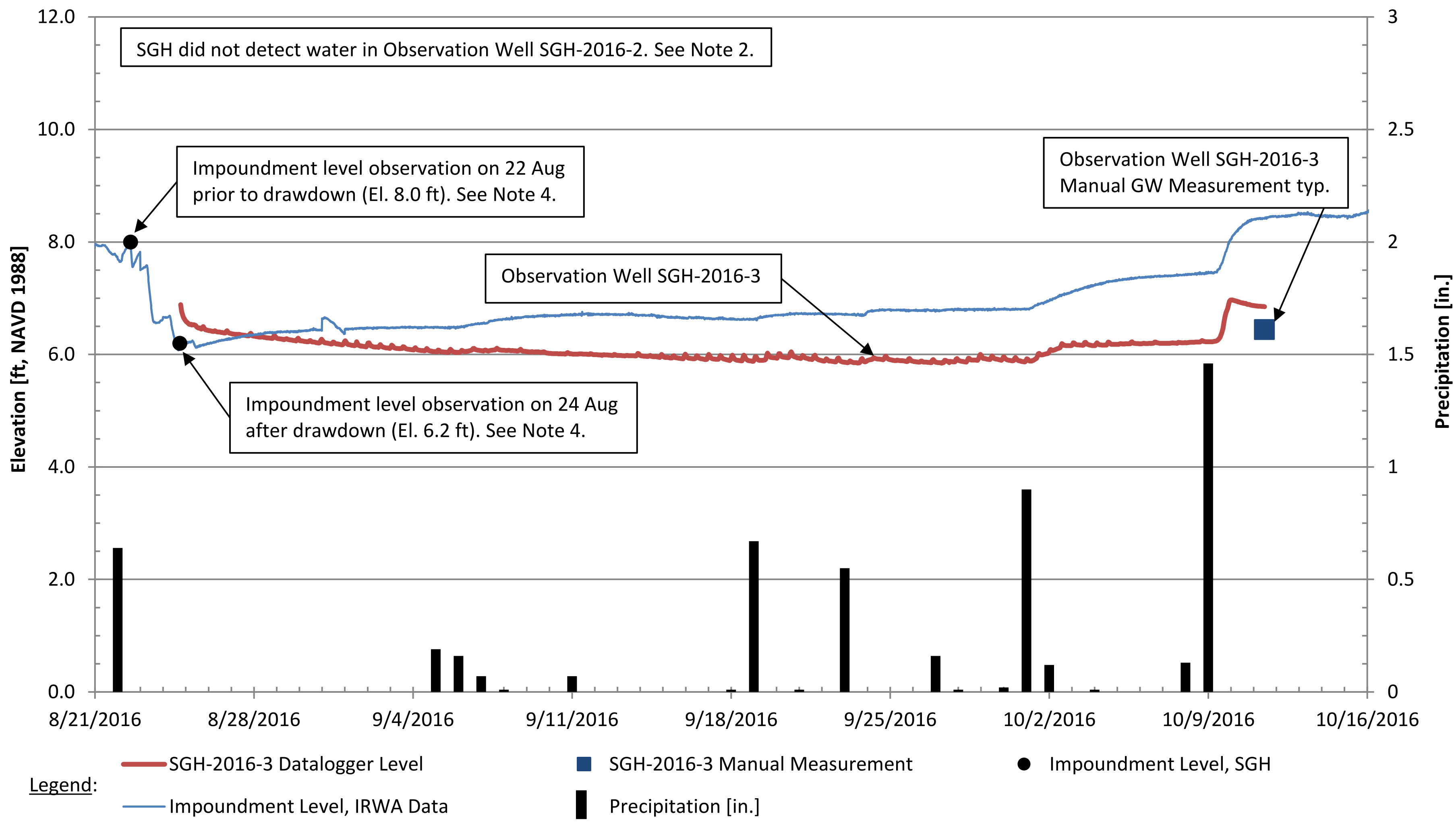
Drawing Title

Project No.	Checked	Date
160630.00	SFK	10/28/2016
Drawn	Approved	Scale
NDL	-	1V:5H

Drawing No.

FIG. 5

Seal



Notes:

1. Groundwater levels are data recorded every 1 hr at the site with data loggers (model Levelogger Gold by Solinst) installed in groundwater observation wells.
2. We observed that Observation Well SGH-2016-2 was dry on 24 August 2016 during installation and on 11 October 2016 when we removed the data logger. The logger did not detect water during the period of record. We installed the datalogger in SGH-2016-2 at El. 8.5 ft, near the bottom of the observation well (located at El. 7.5 ft).
3. Daily precipitation values are from 15 minute unverified data obtained from the Plum Island Ecosystems LTER Field Station in Rowley, MA (http://www.pielter.org/)
4. Impoundment Level, SGH data are based on SGH field observations of the IRWA staff gauge located on the upstream side of the Ipswich Mills Dam. Continuous impoundment level data provided by the IRWA and HW.
5. Groundwater observation well locations are shown on Fig. 1.

APPENDIX A
Recommended Supplemental Foundation Investigation

Based on the results of the current investigation, we identified options for the project team to determine next steps in the feasibility study for the Ipswich Dam removal. Option 2 is to perform a supplemental foundation investigation in the building areas that were not accessible during the current investigation. Performing this investigation prior to completing the feasibility study for the dam removal would provide actionable information to perform a better assessment of the likelihood of the need for mitigation options, as it would allow the project team to identify whether timber piles are present in the remaining areas of the EBSCO Facility where test pits have not been performed and would also allow identification of the presence of soft compressible soils within the footprint of the EBSCO Facility.

An outline of the recommended supplemental investigation is included below. We estimate that the order-of-magnitude cost for the supplemental investigation as outlined would be approximately $200,000, assuming adequate access for the investigation, minor dewatering required for test pits, and replacement of basic finishes only (concrete slab and asphalt pavement). The recommended supplemental foundation investigation includes the following tasks:

- Perform additional subsurface investigation to aid in determining the presence and extent of soft compressible soils within and around the EBSCO Facility. We recommend that at least five soil test borings be located around the exterior of the EBSCO Facility and that at least two soil test borings be located inside the EBSCO Facility. We recommend performing the soil test borings prior to the test pit investigation, as the results of the soil test borings can assist in selecting test pit investigation locations.

  - If soft compressible soils are present, obtain undisturbed soil samples for consolidation tests in a soil testing laboratory to determine soil compressibility properties.

- Perform additional test pit investigations at portions of the buildings not included in the current study, (e.g., Building Nos. 10, 11, and 11-A, which were constructed in 1901, 1918, and 1946 respectively). Additional test pits should include the following:

  - At least one test pit located inside the EBSCO Facility near a column and/or interior bearing wall. Three test pits located outside the EBSCO Facility at each of Building Nos. 10, 11, and 11-A.
  - The test pits could also be used to observe subsurface soil conditions below the first-floor slab. Dewatering is likely to be needed to confirm the depth to the bottom of the concrete foundations and determine if timber piles are present.

- Obtain three concrete core samples of the riverfront foundation wall to confirm the thickness and to obtain samples for laboratory analysis. Perform a petrographic analysis and testing to estimate long-term durability impacts due to a lowered impoundment level.

Alternatively, if EBSCO does not provide access to the inside of its facility or access for test pit investigations on the exterior of the facility, then a limited soil test boring investigation could be performed on the building exterior. The limited investigation would include five to ten soil test borings drilled in the EBSCO Facility parking lot and other exterior areas near the building, such as the grassed area at the south end of the building. The soil test borings would provide some subsurface information for the EBSCO Facility site and allow the project team to further evaluate the potential risks due to compressible soils and timber piles, if any are deemed to present. We estimate that the order-of-magnitude cost for the limited supplemental investigation would be approximately $50,000.

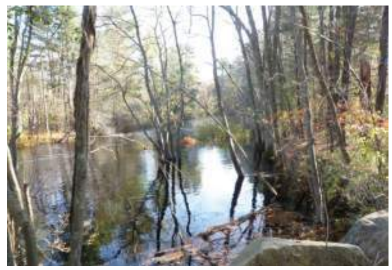
APPENDIX B
Excerpts from Report titled “Ipswich Mills Dam Partial Feasibility Study” prepared by Horsley Witten Group dated 23 April 2014

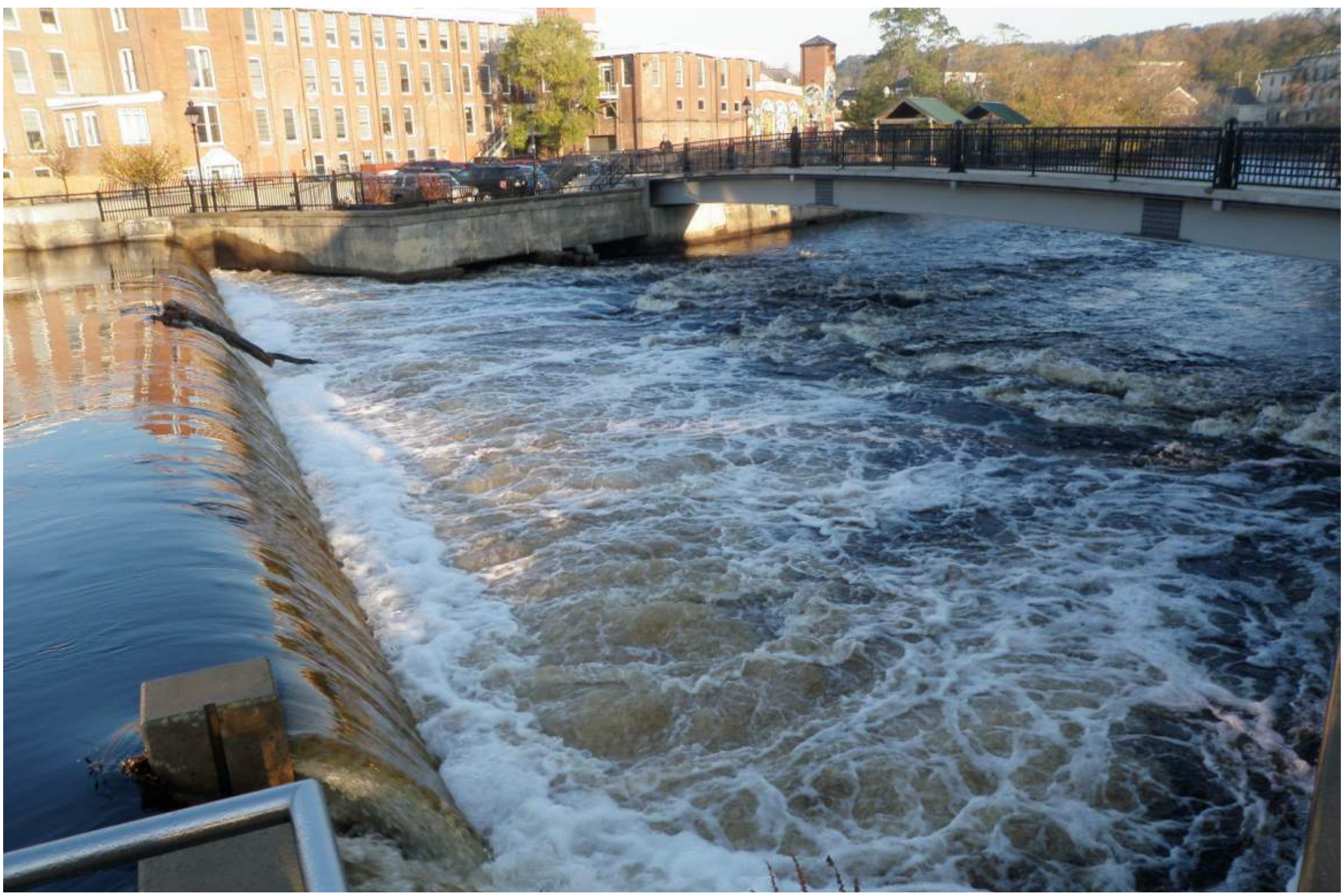
Ipswich Mills Dam Partial Feasibility Study

Preliminary analysis of three primary factors that may influence the cost and feasibility of the removal of the Ipswich Mills Dam, Ipswich, MA

April 23, 2014







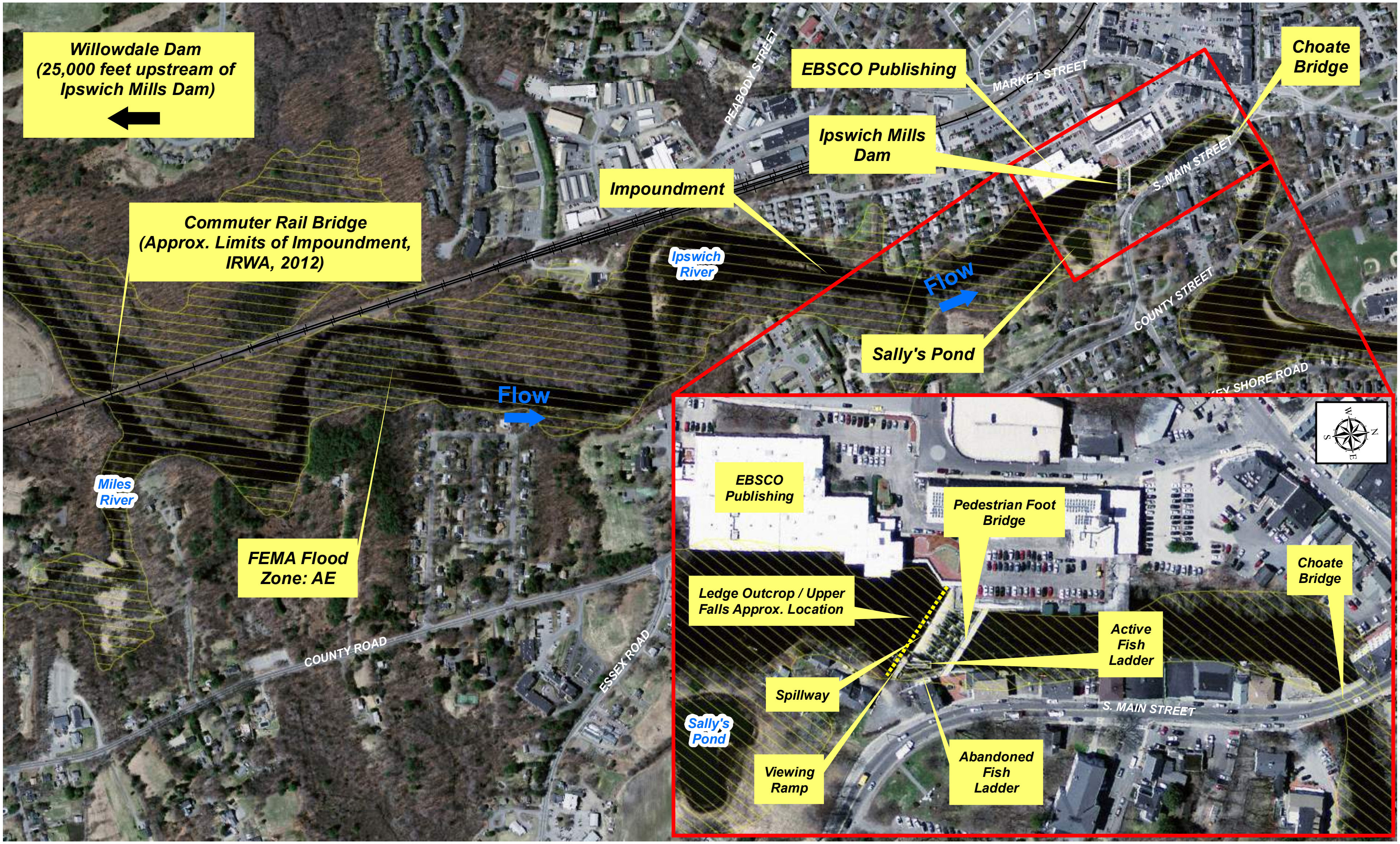
clf conservation law foundation

Division of Ecological Restoration

GEI Consultants

CLEAN SOILS ENVIRONMENTAL LTD

Horsley Witten Group Sustainable Environmental Solutions



Path: H:\Projects\2011\11101 Ipswich Mills Dam Feas.Study\GIS\Maps\Figure_1.mxd

Legend

Commuter Rail Service

FEMA AE Zone: The base floodplain (100-yr flood zone), or areas with a 1% annual chance of flooding and a 26% chance of flooding over the life of 30-year mortgage. Base flood elevations have been provided.

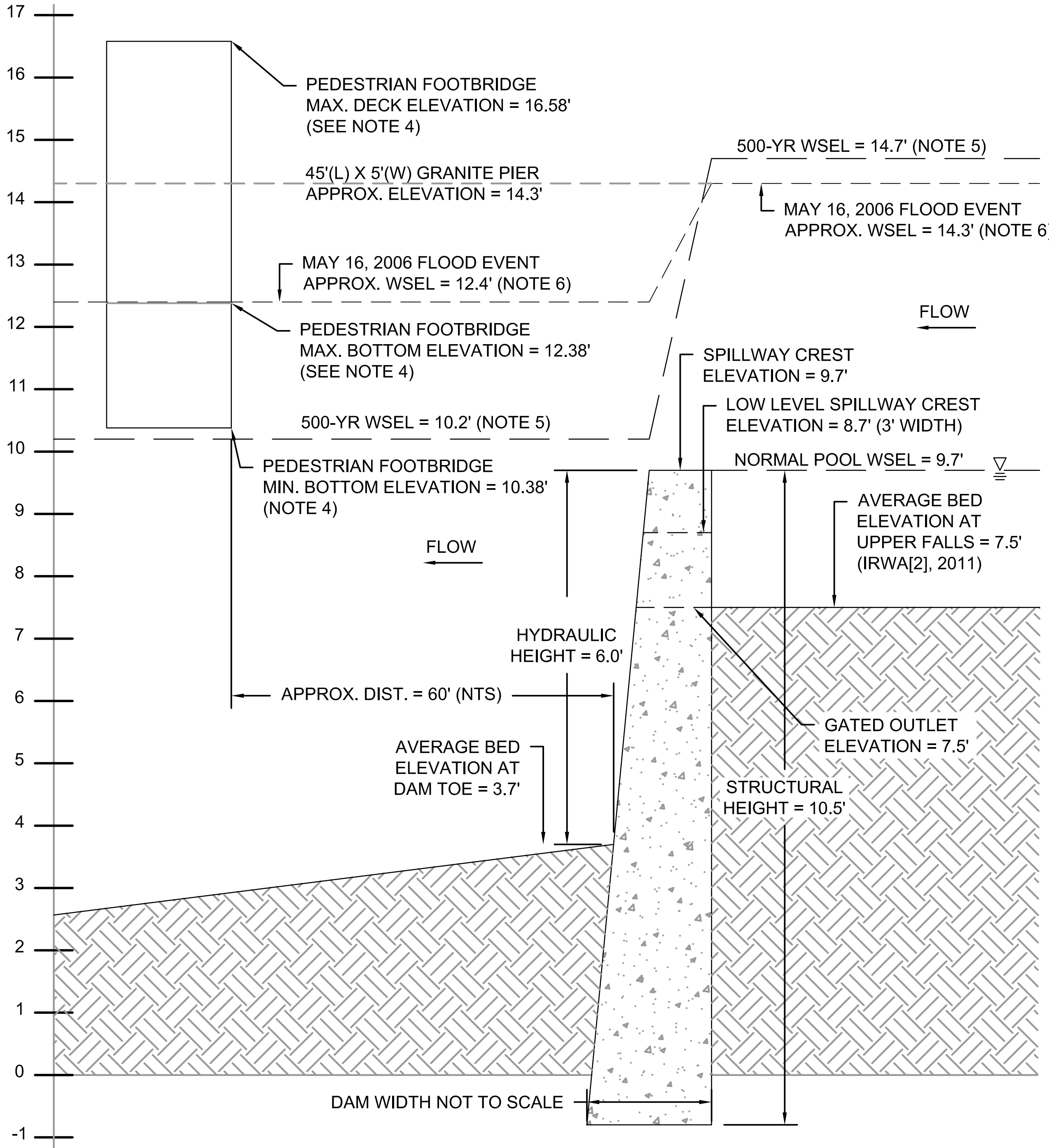
Feet
750

Horsley Witten Group
Sustainable Environmental Solutions

Ipswich Mills Dam and
Surrounding Area
Ipswich, MA

Date: 3/15/2012

Figure 1



NOTES:

1. ALL ELEVATIONS REPORTED ARE IN FEET AND ARE BASED ON THE NATIONAL GEODETIC VERTICAL DATUM OF 1929 (NGVD).
2. WSEL = WATER SURFACE ELEVATION
3. UNLESS OTHERWISE NOTED, REPORTED ELEVATIONS AND DIMENSIONS IN THIS FIGURE WERE OBTAINED FROM THE IPSWICH MILLS DAM PHASE INSPECTION / EVALUATION REPORT (HALEY & ALDRICH, 2009).
4. ELEVATIONS WERE OBTAINED FROM THE *PEDESTRIAN BRIDGE OVER THE IPSWICH RIVER CONSTRUCTION PLANS* (BETA GROUP, INC. & MASSHIGHWAY, 2001).
5. ELEVATIONS WERE OBTAINED FROM THE IPSWICH, MA FLOOD INSURANCE STUDY (FEMA, 1985).
6. ELEVATIONS WERE APPROXIMATED BY IRWA FROM PHOTO DOCUMENTATION ON MAY 16, 2006 (SEE FIGURES 2A & 2B) IN CONJUNCTION WITH THE REFERENCES LISTED IN NOTES 3 AND 4.

Project Number: *11101*

Sheet Number: *1 of 1*

Registration:

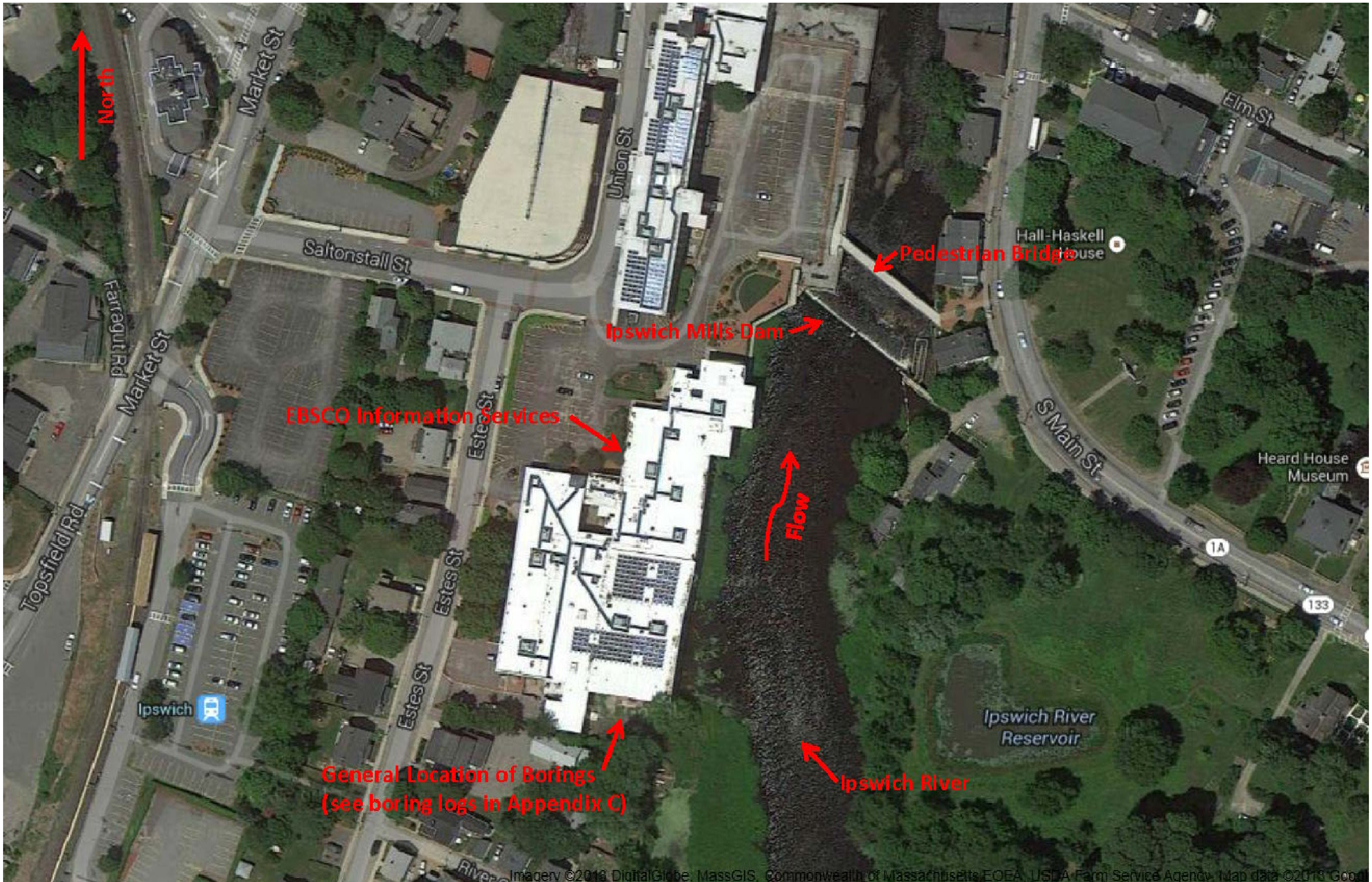
Prepared For: Town of Ipswich, 25 Green Street, Ipswich, MA 01938

Plan Set: *IPSWICH MILLS DAM CROSS-SECTION*
IPSWICH, MASSACHUSETTS

Plan Title: *FIGURE 3*

Horsley Witten Group, Inc.
Sustainable Environmental Solutions
90 Route 6A
Sandwich, MA 02563
508-833-6600 voice
508-833-3150 fax

Date:	Designed By:	Drawn By:	Checked By:
06/04/2012	--	KH	EB



Source: **Google Maps**

0 150 300

APPROXIMATE SCALE, FEET

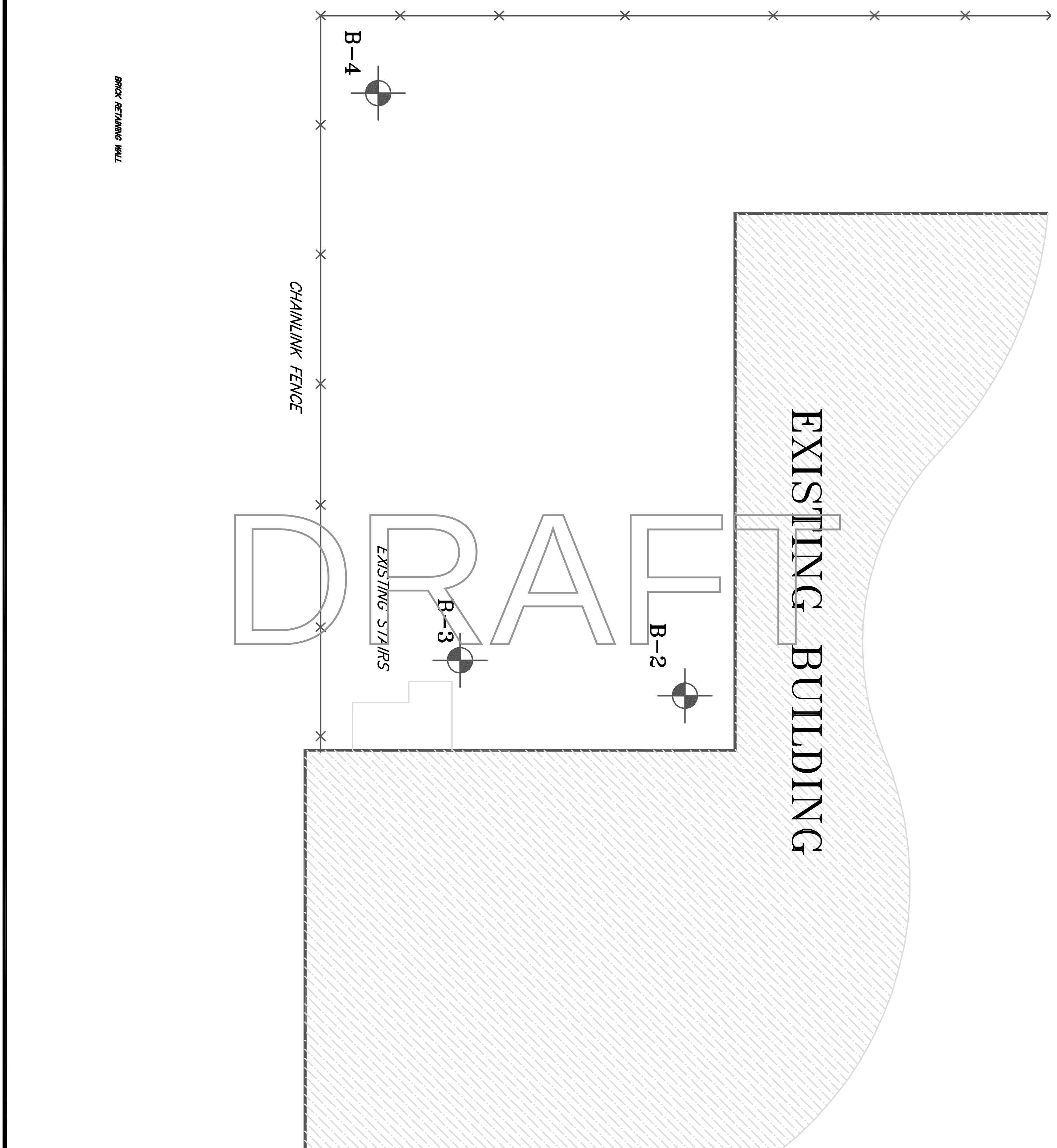
Evaluation of Ipswich Mills Dam Removal on EBSCO Buildings Ipswich, Massachusetts	GEI Consultants	AERIAL VIEW OF PROJECT AREA	
Town of Ipswich Ipswich, Massachusetts	Project 1325760-0	December 2013	Fig. 1

M:\DRAFTING\2013\1325760 Ipswich Mills Dam\1325760-01

Source: **Bing Maps**

Evaluation of Ipswich Mills Dam Removal on EBSCO Buildings Ipswich, Massachusetts	GEI Consultants	EBSCO BUILDING Nos. 9, 10, AND 10-A
Town of Ipswich Ipswich, Massachusetts	Project 1325760-0	December 2013 Fig. 2

M:\DRAFTING\2013\1325760 Ipswich Mills Dam\1325760-01



TITLE:

BORING LOCATION PLAN

PROJECT: **EBSCO PUBLISHING WAREHOUSE EXPANSION**
IPSWICH, MASSACHUSETTS

CLIENT: **PARK CONSTRUCTION CORPORATION**
FITZWILLIAM, New Hampshire

GEOTECHNICAL SERVICES INC.
18 COTE AVENUE, UNIT #11, GOFFSTOWN, NH 03045
TEL. (603) 624-2722 FAX. (603) 624-3733

DATE: JUNE, 22 2009

DESIGN BY:	H. WETHERBEE, P.E.	PROJECT No.	???
DRAWN BY:	D. HAYNER	SHEET No.	1 of 1
CHECKED BY:	H. WETHERBEE, P.E.	SCALE:	NONE

Geotechnical Services, Inc. ◆ 18 Cote Avenue, Goffstown, NH 03045 Tel. 603.624.2722 Fax. 603.624.3733 ◆ 12 Rogers Road, Haverhill, MA 01835 Tel. 978.374.7744 Fax. 978.374.7799

TEST BORING LOG

Boring No. B - 2

Project	Ebsco Publishing Warehouse Add	GSI Project No.		Elevation	n/a
Location	Ipswich, MA	Project Mgr.	Glenn Zoladz	Datum	
Client	Ebsco Publishing	Inspector	Denis Hayner	Date Started	6/19/2009
Contractor	New Hampshire Boring	Checked By		Date Finished	6/19/2009
Driller	Gregg-Mike	Rig Make & Model	Scout Rig		

Item:	Auger	Casing	Sampler	Core Barrel		Truck		Skid		Hammer Type:
Type	HS		SS			Track	X	ATV		Safety Hammer
Inside Diameter (in).	2.25		1-3/8			Bomb		Geoprobe	X	Doughnut
Hammer Weight (lb)			140			Tripod		Other		Automatic
Hammer Fall (in.)			30			Winch		Cat Head	X	Roller Bit / Cutting Head

Depth (ft)	Casing (Blows/ft)	Sample Data No.	Depth (ft)	Rec (in.)	SPT (Blows/ 6-in.)	Rock RQD (%)	PID Rdg. (ppm)	SOIL AND ROCK CLASSIFICATION-DESCRIPTION BURMISTER SYSTEM (SOIL) U.S. CORPS OF ENGINEERS SYSTEM (ROCK)
0		S1	0-2	5	5-8			Top 3" loose Dark Brown fine to medium Sand, little Silt, trace to little Organics (TOPSOIL)
1					7-5			
2		S2	2-4	7	1-4			Very Loose to loose wet Brown/Black fine Sand and Silt, trace to little Organics (FILL)
3					6-6			
4								
5								
6		S3	5-7	12	2-1			Very Loose, Moist, Brown/Black fine Sand and Silt, trace organics
7					1-1			
8								
9								
10		S4	10-12	16	10-17			Light Brown, medium dense, wet fine to coarse Sand and Gravel, trace to little Silt (TILL)
11					19-15			
12								
13								
14								
15								
16		S5	15-17	9	15-15			Light Brown, medium dense, wet fine to coarse Sand and Gravel, trace to little Silt
17					21-16			
18								
19								
20								
21		S-6	20-22	18	44-45			Light Brown, dense, wet fine to coarse Sand and Gravel, trace to little Silt
22					38-45			
23								Boring terminated at 22.0 feet without refusal
24								
25								
26								
27								
28								
29								
30								

DRAFT

Water Level Data

Date	Time	Depth (ft) to: Bott. of Casing	Bott. of Hole	Water
6/19	11:30	n/a	n/a	1.0 ft

Sample Identification
O = Open Ended
U = Undisturbed
S = Split Spoon
C = Rock Core
G = Geoprobe

Cohesive Soils N-Value
0 to 2: Very Soft
2 to 4: Soft
4 to 8: Medium Stiff
8 to 15: Stiff
15 to 30: Very Stiff
Over 30: Hard

Granular Soils N-Value
0 to 4: Very Loose
4 to 10: Loose
11 to 30: Medium Dense
31 to 50: Dense
Over 50: Very Dense

Trace (0 to 5%) Little (10 to 20%) Some (20 to 35%) And (35 to 50%)

Standard Penetration Test (SPT) = 140# hammer falling 30", Blows are per 6" taken with an 18" long x 1.5" I.D. split spoon sampler in accordance with ASTM D 1586, unless otherwise noted.

REMARKS: The stratification lines represent the approximate boundary between soil types and the transition may be gradual. Water level readings have been made in the test borings at times and under conditions stated on the test boring logs. Fluctuations in the level of the groundwater may occur due to other factors than those present at the time measurements were made.

Notes:

B-2

Geotechnical Services, Inc. ◆ 18 Cote Avenue, Goffstown, NH 03045 Tel. 603.624.2722 Fax. 603.624.3733 ◆ 12 Rogers Road, Haverhill, MA 01835 Tel. 978.374.7744 Fax. 978.374.7799

GSI

TEST BORING LOG

Boring No. B - 3

Project	Ebsco Publishing Warehouse Add	GSI Project No.		Elevation	n/a
Location	Ipswich, MA	Project Mgr.	Glenn Zoladz	Datum	
Client	Ebsco Publishing	Inspector	Denis Hayner	Date Started	6/19/2009
Contractor	New Hampshire Boring	Checked By		Date Finished	6/19/2009
Driller	Gregg-Mike	Rig Make & Model	Scout Rig		

Item:	Auger	Casing	Sampler	Core Barrel		Truck		Skid		Hammer Type:
Type	HS		SS			Track	X	ATV		Safety Hammer
Inside Diameter (in).	2.25		1-3/8			Bomb		Geoprobe	X	Doughnut
Hammer Weight (lb)			140			Tripod		Other		Automatic
Hammer Fall (in.)			30			Winch		Cat Head	X	Roller Bit / Cutting Head

Depth (ft)	Casing (Blows/ft)	Sample Data No.	Depth (ft)	Rec (in.)	SPT (Blows/ 6-in.)	Rock RQD (%)	PID Rdg. (ppm)	SOIL AND ROCK CLASSIFICATION-DESCRIPTION BURMISTER SYSTEM (SOIL) U.S. CORPS OF ENGINEERS SYSTEM (ROCK)
0		S1	0-2	5	2-5			Top 3" Very loose to loose Dark Brown fine to medium Sand, little Silt, trace to little Organics (TOPSOIL)
1					4-3			
2								
3		S2	2-4	18	7-5			Top 6" Loose Dark Brown fine to medium Sand and gravel, little Silt, trace to little Organics (FILL)
4					5-5			Loose, Moist, Light Brown fine Sand and Silt (FILL)
5								
6		S3	5-7	18	1-1			Grey, wet, very soft Clay ,trace black fine sand (in seams) (CLAY)
7					1-1			--- q_u = 1.0 tsf using a pocket penetrometer
8								
9								
10		S4	10-12	18	1-2			Grey, wet, very soft Clay ,trace black fine sand (in seams) --- q_u = 1.0 tsf using a pocket penetrometer
11					2-2			Bottom 4" Black, Wet, fine Sand and Silt with some organics (PEAT)
12								
13								
14								
15								
16		S5	15-17	16	2-7			Light Brown, wet, loose to medium dense fine to medium Sand and Silt with some Clay (TILL)
17					14-17			Bottom 4", Light Brown, medium dense, Wet fine to coarse Sand and Gravel, trace to little Silt (TILL)
18								
19								
20								
21		S-6	20-22	14	33-44			Light Brown, dense, wet, fine to coarse Sand and Gravel, trace to little Silt
22					48-35			
23								
24								
25		S-7	25-27	14	5-19			Light Brown, medium dense, wet, fine to coarse Sand and Gravel, trace to little Silt
26					25-28			
27								
28								Boring terminated at 27 feet without refusal
29								
30								

Water Level Data

Date	Time	Depth (ft) to: Bott. of Casing	Bott. of Hole	Water
6/19	9:30	n/a	n/a	3.5 ft

Sample Identification	Cohesive Soils N-Value	Granular Soils N-Value
O = Open Ended	0 to 2: Very Soft	0 to 4: Very Loose
U = Undisturbed	2 to 4: Soft	4 to 10: Loose
S = Split Spoon	4 to 8: Medium Stiff	11 to 30: Medium Dense
C = Rock Core	8 to 15: Stiff	31 to 50: Dense
G = Geoprobe	15 to 30: Very Stiff	Over 50: Very Dense
	Over 30: Hard	

Trace (0 to 5%) Little (10 to 20%) Some (20 to 35%) And (35 to 50%)

Standard Penetration Test (SPT) = 140# hammer falling 30", Blows are per 6" taken with an 18" long x 1.5" I.D. split spoon sampler in accordance with ASTM D 1586, unless otherwise noted.

REMARKS: The stratification lines represent the approximate boundary between soil types and the transition may be gradual. Water level readings have been made in the test borings at times and under conditions stated on the test boring logs. Fluctuations in the level of the groundwater may occur due to other factors than those present at the time measurements were made.

Geotechnical Services, Inc. ◆ 18 Cote Avenue, Goffstown, NH 03045 Tel. 603.624.2722 Fax. 603.624.3733 ◆ 12 Rogers Road, Haverhill, MA 01835 Tel. 978.374.7744 Fax. 978.374.7799

TEST BORING LOG

Boring No. B - 4

Project	Ebsco Publishing Warehouse Add	GSI Project No.		Elevation	n/a
Location	Ipswich, MA	Project Mgr.	Glenn Zoladz	Datum	
Client	Ebsco Publishing	Inspector	Denis Hayner	Date Started	6/19/2009
Contractor	New Hampshire Boring	Checked By		Date Finished	6/19/2009
Driller	Gregg-Mike	Rig Make & Model	Scout Rig		

Item:	Auger	Casing	Sampler	Core Barrel		Truck		Skid		Hammer Type:
Type	HS		SS			Track	X	ATV		Safety Hammer
Inside Diameter (in).	2.25		1-3/8			Bomb		Geoprobe	X	Doughnut
Hammer Weight (lb)			140			Tripod		Other		Automatic
Hammer Fall (in.)			30			Winch		Cat Head	X	Roller Bit	Cutting Head

Depth (ft)	Casing (Blows/ft)	Sample Data No.	Depth (ft)	Rec (in.)	SPT (Blows/ 6-in.)	Rock RQD (%)	PID Rdg. (ppm)	SOIL AND ROCK CLASSIFICATION-DESCRIPTION BURMISTER SYSTEM (SOIL) U.S. CORPS OF ENGINEERS SYSTEM (ROCK)
0		S1	0-2	4	4-4			Top 3" Very loose to loose Dark Brown fine to medium Sand, little Silt, trace to little Organics (TOPSOIL)
1					4-3			
2		S2	2-4	18	2-1			Very Loose Dark Brown fine to medium Sand and gravel, little Silt, trace to little Organics (FILL)
3					2-3			
4								
5								
6		S3	5-7	18	3-3			Very Loose, Moist, Brown/Black fine Sand and Silt, trace organics
7					2-3			
8								
9								
10		S4	10-12	18	1-5			Grey, Wet, soft Clay ,trace black fine sand (in seams) --- q_u = 1.0 tsf using a pocket penetrometer
11					3-3			Bottom 5" Black, Wet, fine Sand and Silt with some organics (PEAT)
12								
13								
14								
15								
16		S5	15-17	9	16-21			Light Brown, medium dense to dense, wet fine to coarse Sand and Gravel, trace to little Silt (TILL)
17					33-22			
18								
19		S-6	18-20	5	67-38			Light Brown, dense to very dense, wet fine to coarse Sand and Gravel, trace to little Silt
20					60-5"			
21								Refusal at 19.5 feet
22								Boring terminated at 19.5 feet
23								
24								
25								
26								
27								
28								
29								
30								

Water Level Data

Date	Time	Depth (ft) to: Bott. of Casing	Bott. of Hole	Water
6/19	1:30	n/a	n/a	3.5 ft

Sample Identification
O = Open Ended
U = Undisturbed
S = Split Spoon
C = Rock Core
G = Geoprobe

Cohesive Soils N-Value
0 to 2: Very Soft
2 to 4: Soft
4 to 8: Medium Stiff
8 to 15: Stiff
15 to 30: Very Stiff
Over 30: Hard

Granular Soils N-Value
0 to 4: Very Loose
4 to 10: Loose
11 to 30: Medium Dense
31 to 50: Dense
Over 50: Very Dense

Trace (0 to 5%) Little (10 to 20%) Some (20 to 35%) And (35 to 50%)

Standard Penetration Test (SPT) = 140# hammer falling 30", Blows are per 6" taken with an 18" long x 1.5" I.D. split spoon sampler in accordance with ASTM D 1586, unless otherwise noted.

REMARKS: The stratification lines represent the approximate boundary between soil types and the transition may be gradual. Water level readings have been made in the test borings at times and under conditions stated on the test boring logs. Fluctuations in the level of the groundwater may occur due to other factors than those present at the time measurements were made.

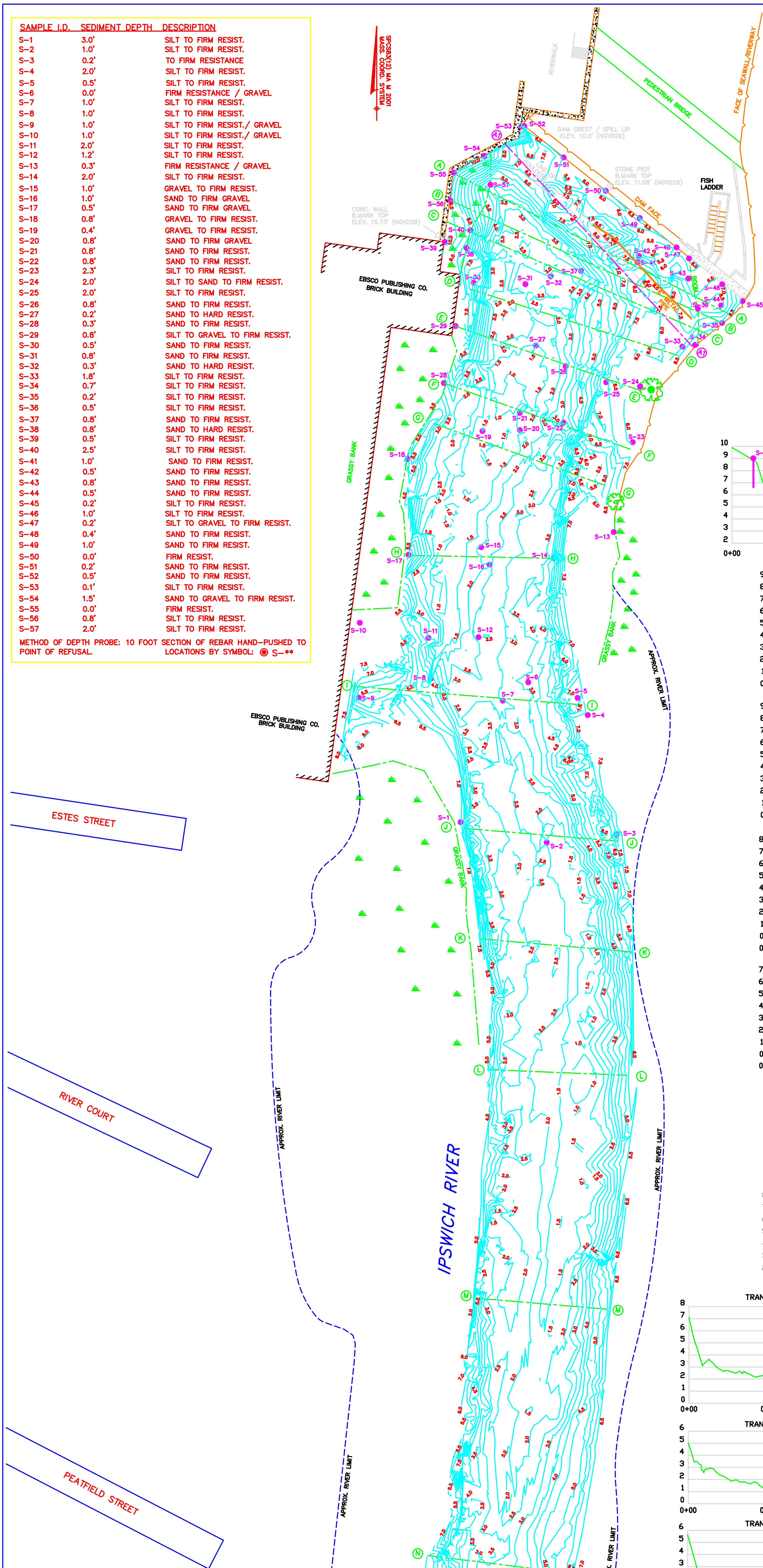
Notes:

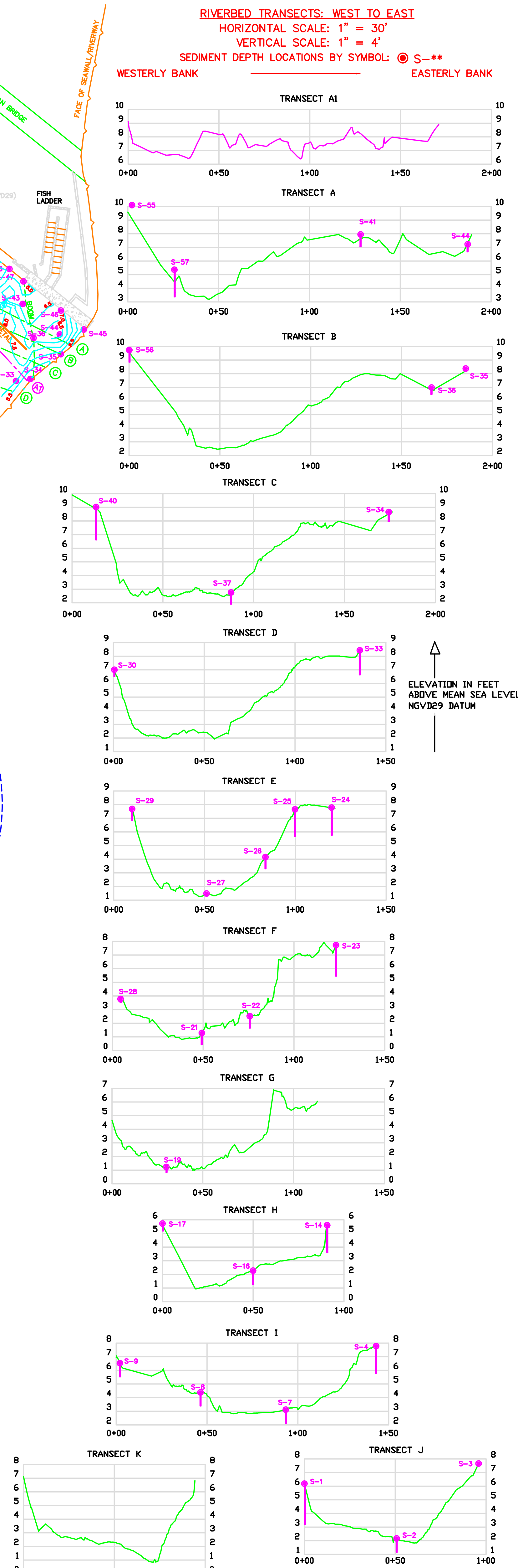
B-4

APPENDIX C
Excerpts from the Report titled "Ipswich River Mills Dam survey" prepared by Norde-East Survey dated 26 August 2014

SAMPLE I.D.	SEDIMENT DEPTH	DESCRIPTION
S-1	3.0'	SILT TO FIRM RESIST.
S-2	1.0'	SILT TO FIRM RESIST.
S-3	0.2'	TO FIRM RESISTANCE
S-4	2.0'	SILT TO FIRM RESIST.
S-5	0.5'	SILT TO FIRM RESIST.
S-6	0.0'	FIRM RESISTANCE / GRAVEL
S-7	1.0'	SILT TO FIRM RESIST.
S-8	1.0'	SILT TO FIRM RESIST.
S-9	1.0'	SILT TO FIRM RESIST./ GRAVEL
S-10	1.0'	SILT TO FIRM RESIST./ GRAVEL
S-11	2.0'	SILT TO FIRM RESIST.
S-12	1.2'	SILT TO FIRM RESIST.
S-13	0.3'	FIRM RESISTANCE / GRAVEL
S-14	2.0'	SILT TO FIRM RESIST.
S-15	1.0'	GRAVEL TO FIRM RESIST.
S-16	1.0'	SAND TO FIRM GRAVEL
S-17	0.5'	SAND TO FIRM GRAVEL
S-18	0.8'	GRAVEL TO FIRM RESIST.
S-19	0.4'	GRAVEL TO FIRM RESIST.
S-20	0.8'	SAND TO FIRM GRAVEL
S-21	0.8'	SAND TO FIRM RESIST.
S-22	0.8'	SAND TO FIRM RESIST.
S-23	2.3'	SILT TO FIRM RESIST.
S-24	2.0'	SILT TO SAND TO FIRM RESIST.
S-25	2.0'	SILT TO FIRM RESIST.
S-26	0.8'	SAND TO FIRM RESIST.
S-27	0.2'	SAND TO HARD RESIST.
S-28	0.3'	SAND TO FIRM RESIST.
S-29	0.8'	SILT TO GRAVEL TO FIRM RESIST.
S-30	0.5'	SAND TO FIRM RESIST.
S-31	0.8'	SAND TO FIRM RESIST.
S-32	0.3'	SAND TO HARD RESIST.
S-33	1.8'	SILT TO FIRM RESIST.
S-34	0.7'	SILT TO FIRM RESIST.
S-35	0.2'	SILT TO FIRM RESIST.
S-36	0.5'	SILT TO FIRM RESIST.
S-37	0.8'	SAND TO FIRM RESIST.
S-38	0.8'	SAND TO HARD RESIST.
S-39	0.5'	SILT TO FIRM RESIST.
S-40	2.5'	SILT TO FIRM RESIST.
S-41	1.0'	SAND TO FIRM RESIST.
S-42	0.5'	SAND TO FIRM RESIST.
S-43	0.8'	SAND TO FIRM RESIST.
S-44	0.5'	SAND TO FIRM RESIST.
S-45	0.2'	SILT TO FIRM RESIST.
S-46	1.0'	SILT TO FIRM RESIST.
S-47	0.2'	SILT TO GRAVEL TO FIRM RESIST.
S-48	0.4'	SAND TO FIRM RESIST.
S-49	1.0'	SAND TO FIRM RESIST.
S-50	0.0'	FIRM RESIST.
S-51	0.2'	SAND TO FIRM RESIST.
S-52	0.5'	SAND TO FIRM RESIST.
S-53	0.1'	SILT TO FIRM RESIST.
S-54	1.5'	SAND TO GRAVEL TO FIRM RESIST.
S-55	0.0'	FIRM RESIST.
S-56	0.8'	SILT TO FIRM RESIST.
S-57	2.0'	SILT TO FIRM RESIST.

METHOD OF DEPTH PROBE: 10 FOOT SECTION OF REBAR HAND-PUSHED TO POINT OF REFUSAL. LOCATIONS BY SYMBOL: ◉ S-**





RIVERBED SURVEY
IPSWICH RIVER MILLS DAM
TOWN OF IPSWICH, MA

SCALE 1" = 30' AUGUST 9, 2014

SURVEYED BY

NORDE-EAST SURVEY

27 CONGRESS STREET, Suite 205-8, SALEM, MA 01970.

Tel.(978) 5421920 E-mail: norde-east@verizon.net

NO.	DATE	DESCRIPTION (REVISIONS)	BY
1.	10/3/14	REVISE CONTOURS BY DAM	P.McC
2.	11/6/14	ADD TRANSECT A1	P.McC

GENERAL NOTES

1.) RIVERBED SOUNDINGS OBSERVED USING AN ODEC BATHY MF500 ECHO SOUNDER WITH REAL-TIME GPS SURFACE NAVIGATION INTERFACED TO HYPACK INC. SOFTWARE.
2.) RIVERBED SEDIMENT DEPTH SAMPLES BY DIRECT PROBE MEASURE & GPS LOCATION.
3.) RIVERBED CONTOURS LABELED AT 0.5 FOOT INTERVALS.
4.) ALL ELEVATION AND CONTOUR DATA RELATE TO THE NGVD29 VERTICAL DATUM.

0' 30' 60' 90'

PATRICK J. McCORMACK – PROFESSIONAL LAND SURVEYOR

DATE

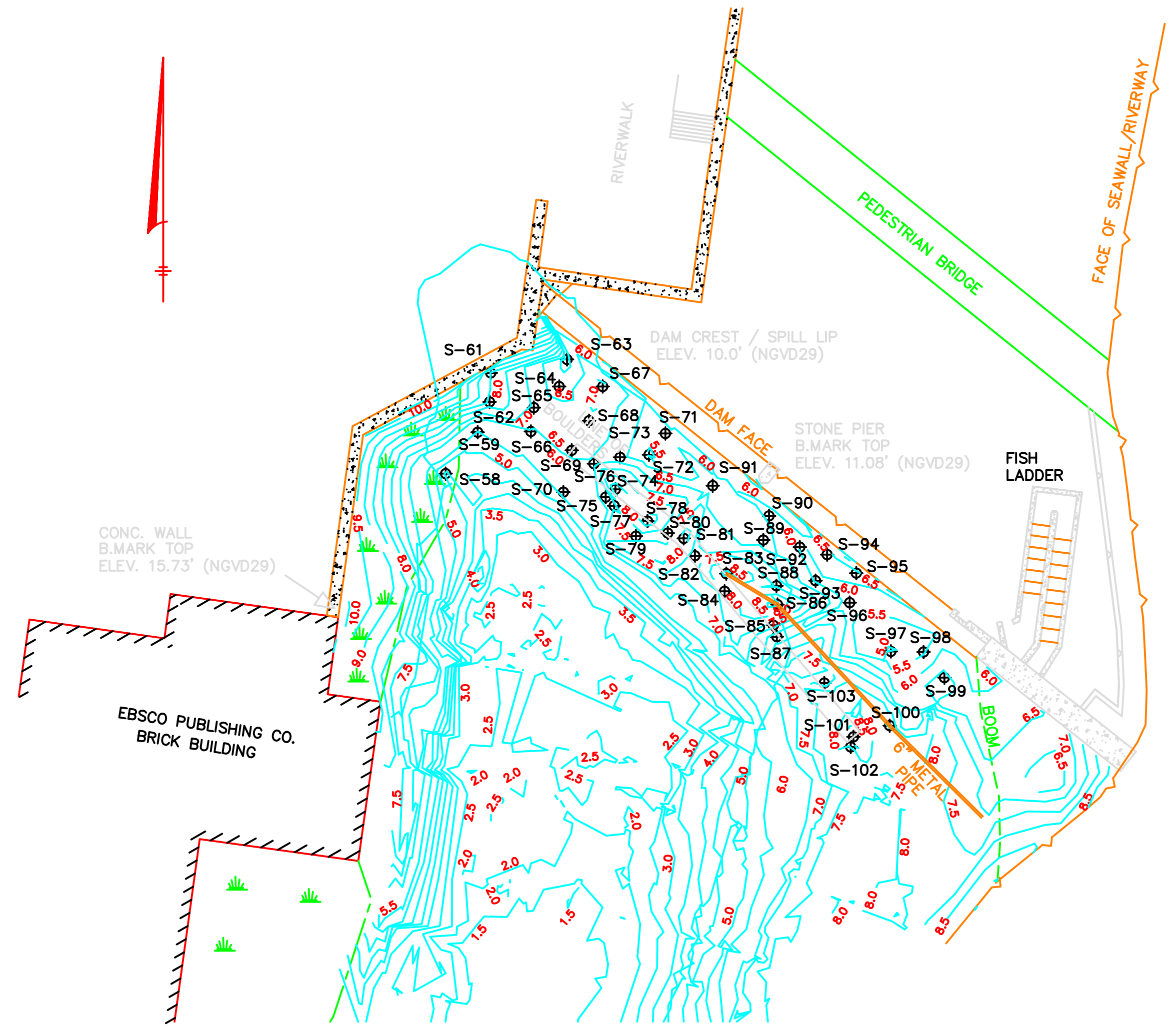
DRAWN BY: P.McCORMACK	DATE: 08/09/14
FIELD CREW: P.McCORMACK D.JONES R.GUILIZIA	DRAWING NO.: 777HYDRO.dwg
JOB NO.: 777	SHEET 1 OF 1

ADDITIONAL SAMPLES MEASURED 9/7/14

SAMPLE I.D.	SEDIMENT DEPTH	DESCRIPTION
S-58	1.2'	COARSE/SAND TO FIRM RESIST.
S-59	1.5'	SAND/COARSE TO FIRM RESIST.
S-61	1.7'	SILT TO FIRM RESIST.
S-62	0.3'	SAND TO FIRM RESIST.
S-63	0.1'	HARD PACK STONE.
S-64	0.8'	HARD PACK STONE.
S-65	0.8'	SILT/GRAVEL TO FIRM RESIST.
S-66	0.0'	HARD PACK GRAVEL.
S-67	0.0'	SOLID ROCK.
S-68	2.5'	LOOSE GRAVEL TO FIRM RESIST.
S-69	1.0'	SAND/GRAVEL TO FIRM RESIST.
S-70	1.5'	SILT/COARSE SAND TO FIRM RESIST.
S-71	0.5'	SILT/SAND TO FIRM RESIST.
S-72	0.3'	COBBLE TYPE SCOUR AREA.
S-73	1.3'	COBBLE TYPE SCOUR AREA.
S-74	0.0'	TOP OF BOULDER LINE.
S-75	1.2'	SILT/SAND TO FIRM RESIST.
S-76	0.5'	SILT/SAND TO FIRM RESIST.
S-77	0.1'	SILT TO FIRM RESIST.
S-78	0.0'	TOP OF BOULDER LINE.
S-79	0.2'	SILT/SAND TO FIRM RESIST.
S-80	0.8'	COARSE SAND TO FIRM RESIST.
S-81	0.0'	TOP OF BOULDER LINE.
S-82	0.3'	SAND TO FIRM RESIST.
S-83	___	TOP OF METAL PIPE.
S-84	0.5'	SILT/SAND TO FIRM RESIST.
S-85	0.0'	HARD PACK STONE.
S-86	___	TOP OF METAL PIPE.
S-87	0.0'	HARD PACK GRAVEL.
S-88	2.5'	SILT/SAND TO FIRM RESIST.
S-89	0.5'	SILYT/SAND TO FIRM RESIST.
S-90	0.0'	LARGE ROCK.
S-91	0.8'	COARSE SAND TO FRIM RESIST.
S-93	0.8'	SILT/SAND TO FIRM RESIST.
S-94	0.3'	SILT/SAND TO FIRM RESIST.
S-95	0.3'	SILT/SAND TO FIRM RESIST.
S-96	0.3'	SILT TO HARD RESIST.
S-97	0.8'	SILT/SAND TO FIRM RESIST.
S-98	0.3'	SILT TO HARD RESIST.
S-99	0.8'	SILT/GRAVEL TO FIRM RESIST.
S-100	___	TOP OF METAL PIPE.
S-101	0.0'	TOP OF BOULDER LINE.
S-102	0.5'	SILT/SAND TO FIRM RESIST.
S-103	0.3'	SILT/SAND TO FIRM RESIST.

S-60 SAMPLE SERIES NUMBER NOT ASSIGNED, FIELD NOTEBOOK OMISSION.

METHOD OF DEPTH PROBE: 10 FOOT SECTION OF REBAR HAND-PUSHED TO POINT OF REFUSAL. LOCATIONS BY SYMBOL: ⊕ S-**



GENERAL NOTES

1.) RIVERBED SOUNDINGS OBSERVED USING AN ODEC BATHY MF500 ECHO SOUNDER WITH REAL-TIME GPS SURFACE NAVIGATION INTERFACED TO HYPACK INC. SOFTWARE.
2.) RIVERBED SEDIMENT DEPTH SAMPLES BY DIRECT PROBE MEASURE & GPS LOCATION.
3.) RIVERBED CONTOURS LABELED AT 0.5 FOOT INTERVALS.
4.) ALL ELEVATION AND CONTOUR DATA RELATE TO THE NGVD29 VERTICAL DATUM.

PATRICK J. McCORMACK – PROFESSIONAL LAND SURVEYOR

DATE

0' 30' 60' 90'

RIVERBED SURVEY
IPSWICH RIVER MILLS DAM
TOWN OF IPSWICH, MA

SCALE 1" = 30' AUGUST 9, 2014

SURVEYED BY

NORDE-EAST SURVEY

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REVISIONS			
1.	10/3/14	REVISE CONTOURS BY DAM	P.McC
2.	11/6/14	ADDITIONAL TRANSECT A1	P.McC
3.	11/6/14	ADDITIONAL SEDIMENT DEPTHS	P.McC
NO.	DATE	DESCRIPTION	BY

DRAWN BY: P.McCORMACK	DATE: 08/09/14
FIELD CREW: P.McCORMACK, D.JONES, R.GUILIZIA	DRAWING NO.: 777HYDRO.dwg
JOB NO.: 777	EXHIBIT ATTACHMENT TO SHEET 1

APPENDIX D
SGH Soil Test Borehole Logs and Observation Well Installation Details

Simpson Gumpertz & Heger, Inc.
41 Seyon St, Building 1 Suite 500
Waltham, MA 02453
Telephone: 781-907-9000
Fax: 781-907-9009

BORING NUMBER SGH-2016-1

CLIENT Horsley & Witten Group
PROJECT NAME Ipswich Mills Dam Removal Feasibility Study
PROJECT NUMBER 160630.00
PROJECT LOCATION Ipswich MA
DATE STARTED 8/24/16 **COMPLETED** 8/24/16
GROUND ELEVATION 17 ft **HOLE SIZE** 4 inches
DRILLING CONTRACTOR Carr-Dee Corp. of MA
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger (HSA)
AT TIME OF DRILLING --- Not Encountered
LOGGED BY SFKeppel **CHECKED BY**
AT END OF DRILLING ---
NOTES Saltonsall St. and Union St. in front of EBSCO parking entrance
AFTER DRILLING ---

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)	GRAPHIC LOG	MATERIAL DESCRIPTION
0					0.3 — 4 in asphalt paving top coarse — 16.7
	SS S-1	89	13-12-7 (19)		FILL: sand and gravel; light brown; dense; dry; fine to coarse grained; poorly graded; sub-angular; some silt; some asphalt fragments; trace brick
					Note: Borehole uncased from 0 ft to 4 ft. Borehole collapsed at an approximate depth of 2 ft.
	SS S-2	38	9-11-21-25 (32)		
					4.5 — 12.5
5	SS S-3	75	9-18-20-35 (38)		GLACIAL TILL: silty sand and gravel; light reddish brown; very dense; dry; fine to coarse grained; poorly graded; angular
	SS S-4	74	19-22-29-60/2"		Note: difficult to advance HSA at 7 ft to 7.5 ft.
	SS S-5	28	82-100/1"		Note: recovered granite fragments at 8 ft.
10					Grading: gravel and silt; wet; some sand
	SS S-6	98	51-99-50/1"		Note: HSA refusal at 11.25 ft (EOB).
					11.3 — 5.8

Bottom of borehole at 11.3 feet.

Note: no indication of seasonal high water levels were observed within the borehole.

Appendix C

Simpson Gumpertz & Heger, Inc.
41 Seyon St, Building 1 Suite 500
Waltham, MA 02453
Telephone: 781-907-9000
Fax: 781-907-9009

WELL NUMBER SGH-2016-2

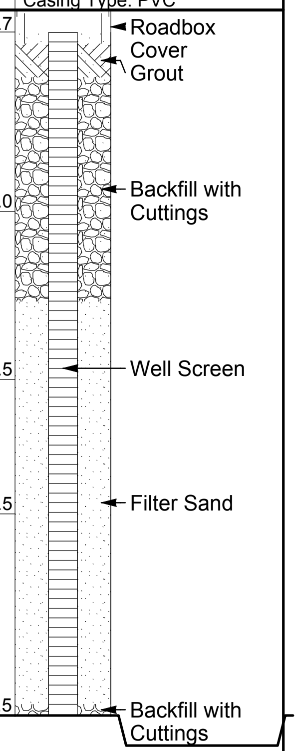
CLIENT Horsley & Witten Group
PROJECT NAME Ipswich Mills Dam Removal Feasibility Study
PROJECT NUMBER 160630.00
PROJECT LOCATION Ipswich MA
DATE STARTED 8/24/16 **COMPLETED** 8/24/16
GROUND ELEVATION 18 ft **HOLE SIZE** 4 inches
DRILLING CONTRACTOR Carr-Dee Corp. of MA
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger (HSA)
AT TIME OF DRILLING --- Not Encountered
LOGGED BY SFKeppel **CHECKED BY**
AT END OF DRILLING ---
NOTES Estes St. 15' from Sidewalk (between No. 7 and No. 10)
AFTER DRILLING ---

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)	DEPTH	MATERIAL DESCRIPTION	ELEV.
0				0.3	4" of Asphalt	17.7
	SS S-1	17	20-54-19 (73)		FILL: sand; dark brown; very dense; dry; fine to medium grained; poorly graded; sub-angular	
	SS S-2	63	5-4-6-10 (10)	3.0	SILTY SAND TO SANDY SILT: light orange brown; medium dense; dry; fine grained; uniform; sub-rounded	15.0
5	SS S-3	75	3-4-12-9 (16)	5.5	SILT: light orange brown; very stiff; moist; low plasticity; some sand; trace clay.	12.5
	SS S-4	92	10-10-17-36 (27)	7.5	Note: observed gravel at tip of sampler	10.5
	SS S-5	29	19-23-47-41 (70)		GLACIAL TILL: sandy gravel; light reddish brown; very dense; moist; fine to coarse grained; poorly graded sub-angular; some silt	
10	SS S-6		50/0"	10.5	Note: HSA refusal at 10.5 ft (EOB).	7.5

Bottom of borehole at 10.5 feet.

WELL DIAGRAM

Casing Top Elev: 18.2 (ft)
Casing Type: PVC



Note: no indication of seasonal high water levels were observed within the borehole.

Appendix C

Simpson Gumpertz & Heger, Inc.
41 Seyon St, Building 1 Suite 500
Waltham, MA 02453
Telephone: 781-907-9000
Fax: 781-907-9009

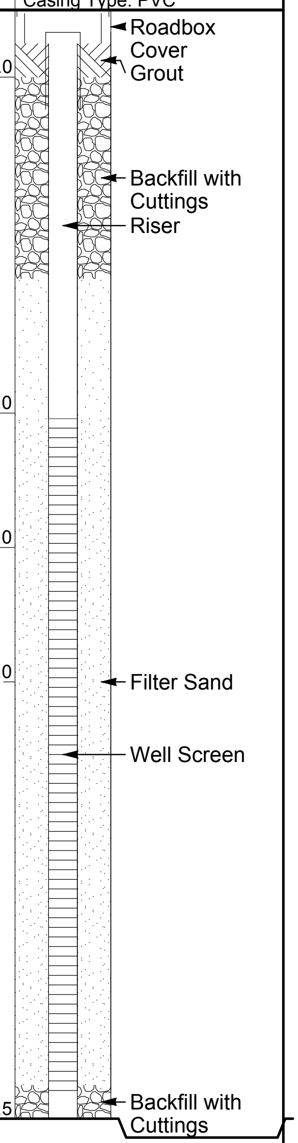
WELL NUMBER SGH-2016-3

CLIENT Horsley & Witten Group
PROJECT NAME Ipswich Mills Dam Removal Feasibility Study
PROJECT NUMBER 160630.00
PROJECT LOCATION Ipswich MA
DATE STARTED 8/24/16 **COMPLETED** 8/24/16
GROUND ELEVATION 14 ft **HOLE SIZE** 4 inches
DRILLING CONTRACTOR Carr-Dee Corp. of MA
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger (HSA)
▽ **AT TIME OF DRILLING** 11.75 ft / Elev 2.25 ft
LOGGED BY SFKeppel **CHECKED BY**
AT END OF DRILLING ---
NOTES Driveway between No. 63 and No. 69 S. Main St.
AFTER DRILLING ---

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)	MATERIAL DESCRIPTION
0	SS S-1	67	32-38-33-30 (71)	Gravel Base Pavement, fine to coarse, dense
1.0				13.0 — FILL: sandy gravel; light brown; very dense; dry; fine to coarse grained; well graded; sub-angular; trace silt
	SS S-2	50	9-22-19-16 (41)	
5	SS S-3	29	9-13-17-24 (30)	
6.0	SS S-4	38	70-57-42-25 (99)	8.0 — SAND: gravely sand; light orange brown; dense; dry; medium grained; well graded; sub-angular (outwash)
8.0				6.0 — Grading: moist
	SS S-5	83	9-10-13-16 (23)	GLACIAL TILL: silty clay; olive; stiff; moist; low plasticity Qp=4.25 tsf, Sv=0.9 tsf
10 (10.0)	SS S-6	25	21-31-28-20 (59)	4.0 — Grading: sandy gravel; olive grey; very dense; moist; medium to coarse grained; poorly graded; sub-angular; some silty clay
				▽ Grading wet at 12 ft.
	SS S-7	50	8-13-15-30 (28)	
				Grading: fine to medium grained at 14 ft.
15	SS S-8	38	36-37-100/4"	
16.5				-2.5 — Note: HSA refusal at 16.5 ft (EOB).

Bottom of borehole at 16.5 feet.

WELL DIAGRAM
Casing Top Elev: 14.2 (ft)
Casing Type: PVC



Note: no indication of seasonal high water levels were observed within the borehole.

Appendix C

Appendix B
2018 Soil Test Boring Logs

Simpson Gumpertz & Heger, Inc.
41 Seyon St, Building 1 Suite 500
Waltham, MA 02453
Telephone: 781-907-9000
Fax: 781-907-9009

BORING NUMBER SGH-2018-1

CLIENT Horsley & Witten Group
PROJECT NAME Ipswich Mills Dam Removal, Supplemental Borings
PROJECT NUMBER 160630.01
PROJECT LOCATION Ipswich MA
DATE STARTED 6/1/18 **COMPLETED** 6/1/18
GROUND ELEVATION 17 ft NAVD88 **HOLE SIZE** 4 ID/4.5 OD inches
DRILLING CONTRACTOR Carr-Dee Corp. of MA
GROUND WATER LEVELS:
DRILLING METHOD Case and Wash Boring
AT TIME OF DRILLING ---
LOGGED BY ZKBoswell **CHECKED BY**
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)	GRAPHIC LOG	MATERIAL DESCRIPTION
0				0.3	ASPHALT — 16.8
	SS S-1	29	7-3-3-5 (6)		FILL: sand; brown; loose; dry; fine to coarse grained; poorly graded; sub-angular; trace to some gravel; trace to little brick; trace silt
5	SS S-2	75	8-11-30-25 (41)		grading dense at 4 ft bgs Observed casing blows increase at approximately 6 feet bgs. Possible stratum change.
				6.0	11.0
					GLACIAL TILL: gravel; gray and brown; very dense; poorly graded; rounded; little fine to coarse sand; trace clayey silt
	SS S-3	100	100/4"		Observed very hard drilling from approximately 7.5 to 8 ft bgs.
	SS S-4	0	100/4"	8.4	8.6
					Bottom of borehole at 8.4 feet.

Note 1: Ground surface elevation estimated from a draft plan entitled "Transects-Plan View" prepared by Horsley Witten Group, Inc., and dated September 23, 2016 and field measurements taken in August 2016 and June 2018. The elevation datum is North American Vertical Datum 1988 (NAVD 88).

Note 2: Drilled borehole with casing to approximately 7.5 ft bgs.

Note 3: Borehole backfilled with drill cuttings and an asphalt cold patch was placed at the ground surface.

Simpson Gumpertz & Heger, Inc.
41 Seyon St, Building 1 Suite 500
Waltham, MA 02453
Telephone: 781-907-9000
Fax: 781-907-9009

BORING NUMBER SGH-2018-2

CLIENT Horsley & Witten Group

PROJECT NAME Ipswich Mills Dam Removal, Supplemental Borings

PROJECT NUMBER 160630.01

PROJECT LOCATION Ipswich MA

DATE STARTED 6/1/18 **COMPLETED** 6/1/18

GROUND ELEVATION 13 ft NAVD88 **HOLE SIZE** 4 ID/4.5 OD inches

DRILLING CONTRACTOR Carr-Dee Corp. of MA

GROUND WATER LEVELS:

DRILLING METHOD Case and Wash Boring

AT TIME OF DRILLING ---

LOGGED BY SFKeppel **CHECKED BY**

AT END OF DRILLING ---

NOTES Qp=Pocket Penetrometer (tsf), Sv=Pocket Torvane (tsf)

AFTER DRILLING ---

SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)
SS S-1	50	3-3-6-7 (9)
SS S-2	67	4-4-1-2 (5)
SS S-3	0	1-1-1-1 (2)
SS S-4	33	27-1-1-1 (2)
SS S-5	0	20-7-3 (10)
SS S-6	8	7-4-4-6 (8)
SS S-7	33	6-4-5-4 (9)
SS S-8	83	5-4-4-7 (8)
SS S-9	67	77-26-37-27 (63)
SS S-10	100	26-39-20-12 (59)

DEPTH (ft)	MATERIAL DESCRIPTION	ELEV.
0.5	TOPSOIL: silty loam; brown; dry	12.5
3.0	FILL: silty sand; brown; loose; moist; fine to coarse grained; poorly graded; sub-angular; trace gravel; trace brick	10.0
6.0	SAND and SILT: olive to brown; very loose; fine grained;	7.0
11.5	Silty CLAY: olive gray; very soft; low plasticity Solid wood observed in the wash water from a depth of 6 ft to 16 ft bgs. grading medium stiff to stiff at 8.5 ft bgs	1.5
13.0	Organic SILT: dark brown; medium stiff; low plasticity; some fine sand	0.0
19.0	Silty CLAY: olive gray; medium stiff; low plasticity Qp = 0.75 tsf at 15 ft bgs Sv = 0.6 tsf at 15 ft bgs	-6.0
24.0	Glacial TILL: gravelly sand; brown; very dense; fine to coarse grained; poorly graded; sub-angular; trace silty clay	-11.0

Bottom of borehole at 24.0 feet.

Note 1: Ground surface elevation estimated from a draft plan entitled "Transects-Plan View" prepared by Horsley Witten Group, Inc., and dated September 23, 2016 and field measurements taken in August 2016 and June 2018. The elevation datum is North American Vertical Datum 1988 (NAVD 88).

Note 2: Borehole backfilled with drill cuttings.

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Fax: 781-907-9009

BORING NUMBER SGH-2018-2A

CLIENT Horsley & Witten Group
PROJECT NAME Ipswich Mills Dam Removal, Supplemental Borings
PROJECT NUMBER 160630.01
PROJECT LOCATION Ipswich MA
DATE STARTED 6/1/18 **COMPLETED** 6/2/18
GROUND ELEVATION 13 ft NAVD88 **HOLE SIZE** 4 ID/4.5 OD inches
DRILLING CONTRACTOR Carr-Dee Corp. of MA
GROUND WATER LEVELS:
DRILLING METHOD Case and Wash Boring
AT TIME OF DRILLING ---
LOGGED BY SFKeppel **CHECKED BY**
AT END OF DRILLING ---
NOTES Qp=Pocket Penetrometer (tsf), Sv=Pocket Torvane (tsf)
AFTER DRILLING ---

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)	GRAPHIC LOG	MATERIAL DESCRIPTION	Elev.
0					Began sampling at 6 feet below ground surface (bgs).	
5						
6.0	SS S-1	8	1-1-1-1 (2)		SILTY CLAY: olive gray; very soft; low plasticity; trace coarse sand	7.0
8.7	ST US-1	92			ORGANIC SILT: dark brown; trace sand; trace gravel; grading soft at 10 ft bgs	4.3
10	SS S-2	0	1-1-1-2 (2)			
11.5	ST US-2	100			Silty CLAY: olive gray; stiff; low plasticity; fine sand seams 1/8 to 2 inches thick throughout sample. Observed a nail embedded in the bottom of shelby tube sample "US-2".	1.5
15	ST US-3	100				
	SS S-3	100	7-9-11-13 (20)		Qp (tsf) = 0.75, 1.5, 2, 2.25 from 15.5 to 17.5 ft bgs Sv (tsf) = 0.3, 0.5, 0.55 from 15.5 to 17.5 ft bgs	
17.5					Bottom of borehole at 17.5 feet.	-4.5

Note 1: Ground surface elevation estimated from a draft plan entitled "Transects-Plan View" prepared by Horsley Witten Group, Inc., and dated September 23, 2016 and field measurements taken in August 2016 and June 2018. The elevation datum is North American Vertical Datum 1988 (NAVD 88).

Note 2: Borehole backfilled with drill cuttings.

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BORING NUMBER SGH-2018-3

CLIENT Horsley & Witten Group
PROJECT NAME Ipswich Mills Dam Removal, Supplemental Borings
PROJECT NUMBER 160630.01
PROJECT LOCATION Ipswich MA
DATE STARTED 6/2/18 **COMPLETED** 6/2/18
GROUND ELEVATION 13 ft NAVD88 **HOLE SIZE** 2.5 ID/4.5 OD inches
DRILLING CONTRACTOR Carr-Dee Corp. of MA
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger
AT TIME OF DRILLING ---
LOGGED BY SFKeppel **CHECKED BY**
AT END OF DRILLING ---
NOTES Qp=Pocket Penetrometer (tsf), Sv=Pocket Torvane (tsf)
AFTER DRILLING ---

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)	MATERIAL DESCRIPTION	
0				0.5 TOPSOIL: silty loam; brown; dry	12.5
	SS S-1	42	8-4-8-6 (12)	FILL: silty sand; brown; medium dense; moist; fine to coarse grained; poorly graded; sub-angular; some gravel; trace brick; trace organics; slight chemical odor	
	SS S-2	33	5-6-4-2 (10)		
5	SS S-3	33	3-3-2-2 (5)	grading very loose to loose at 5 ft below ground surface (bgs)	
	SS S-4	75	2-1-1-1 (2)	9.0	4.0
10				Organic SILT: brown; soft; moist; trace wood fibers	
	SS S-5	58	2-2-3-4 (5)	11.0 Silty CLAY: olive gray; wet; low plasticity; trace mottled fine sand Qp (tsf) = <0.25, 0.75, 1.5 from 11 to 12 ft bgs Sv (tsf) = 0.25, 0.35, 0.75 from 11 to 12 ft bgs	2.0
	SS S-6	75	5-7-7-8 (14)	13.0 GLACIAL TILL: sand; brown; dense; fine to medium grained; poorly graded; sub-angular; some gravel; trace silty clay	0.0
15	SS S-7	88	8-13-17-18 (30)	17.0	-4.0

Bottom of borehole at 17.0 feet.

Note 1: Ground surface elevation estimated from a draft plan entitled "Transects-Plan View" prepared by Horsley Witten Group, Inc., and dated September 23, 2016 and field measurements taken in August 2016 and June 2018. The elevation datum is North American Vertical Datum 1988 (NAVD 88).

Note 2: Borehole backfilled with drill cuttings.

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BORING NUMBER SGH-2018-4

CLIENT Horsley & Witten Group
PROJECT NAME Ipswich Mills Dam Removal, Supplemental Borings
PROJECT NUMBER 160630.01
PROJECT LOCATION Ipswich MA
DATE STARTED 6/1/18 **COMPLETED** 6/1/18
GROUND ELEVATION 16 ft NAVD88 **HOLE SIZE** 4 ID/4.5 OD inches
DRILLING CONTRACTOR Carr-Dee Corp. of MA
GROUND WATER LEVELS:
DRILLING METHOD Case and Wash Boring
AT TIME OF DRILLING ---
LOGGED BY ZKBoswell **CHECKED BY**
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)	GRAPHIC LOG	MATERIAL DESCRIPTION
0	SS S-1	63	1-2-2-4 (4)		FILL: silty sand and gravel; brown; loose; fine to coarse grained Hard drilling observed from approximately 4 to 7 feet bgs. Observed a change of color in the wash water from brown to gray at approximately 6 feet bgs.
5	SS S-2	75	15-17-28-28 (45)		grading medium dense at 5 ft bgs
6.0					10.0
					GLACIAL TILL: fine to coarse sand and gravel; brown/gray; dense; poorly graded; sub-angular; trace to little silt Drill chatter observed at approximately 7 ft bgs. Observed drilling fluid loss at approximately 7.5 ft bgs. Observed hard drilling from approximately 7.5 to 8 ft bgs. Cuttings indiciate possible boulder.
10	SS S-3	46	15-18-20-29 (38)		
12.0					4.0

Bottom of borehole at 12.0 feet.

Note 1: Ground surface elevation estimated from a draft plan entitled "Transects-Plan View" prepared by Horsley Witten Group, Inc., and dated September 23, 2016 and field measurements taken in August 2016 and June 2018. The elevation datum is North American Vertical Datum 1988 (NAVD 88).

Note 2: Drilled borehole with casing to approximately 7.5 ft bgs and open hole to termination.

Note 3: Borehole backfilled with drill cuttings.

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BORING NUMBER SGH-2018-5

CLIENT Horsley & Witten Group
PROJECT NAME Ipswich Mills Dam Removal, Supplemental Borings
PROJECT NUMBER 160630.01
PROJECT LOCATION Ipswich MA
DATE STARTED 6/1/18 **COMPLETED** 6/1/18
GROUND ELEVATION 16 ft NAVD88 **HOLE SIZE** 4 ID/4.5 OD inches
DRILLING CONTRACTOR Carr-Dee Corp. of MA
GROUND WATER LEVELS:
DRILLING METHOD Case and Wash Boring
AT TIME OF DRILLING ---
LOGGED BY ZKBoswell **CHECKED BY**
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEV.
0 – 0.3					ASPHALT	15.7
0.3	SS S-1	21	3-3-2-2 (5)		FILL: silty sand; brown; loose; fine to coarse grained; little silt; trace gravel	
5	SS S-2	17	2-6-6-6 (12)		grading medium dense at 5 ft bgs	
8.0					Clayey SILT: brownish gray; medium stiff; low plasticity; trace fine sand	8.0
10	SS S-3	50	3-3-3-2 (6)			
13.0					GLACIAL TILL: silty sand; grayish brown; dense; fine to coarse grained; sub-angular; some clayey silt; trace gravel	3.0
15	SS S-4	67	15-16-16-13 (32)			
17.0						-1.0

Bottom of borehole at 17.0 feet.

Note 1: Ground surface elevation estimated from a draft plan entitled "Transects-Plan View" prepared by Horsley Witten Group, Inc., and dated September 23, 2016 and field measurements taken in August 2016 and June 2018. The elevation datum is North American Vertical Datum 1988 (NAVD 88).

Note 2: Borehole backfilled with drill cuttings and an asphalt cold patch was placed at the ground surface.

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BORING NUMBER SGH-2018-6

CLIENT Horsley & Witten Group

PROJECT NAME Ipswich Mills Dam Removal, Supplemental Borings

PROJECT NUMBER 160630.01

PROJECT LOCATION Ipswich MA

DATE STARTED 6/2/18 **COMPLETED** 6/2/18

GROUND ELEVATION 13 ft NAVD88 **HOLE SIZE** 4 ID/4.5 OD inches

DRILLING CONTRACTOR Carr-Dee Corp. of MA

GROUND WATER LEVELS:

DRILLING METHOD Case and Wash Boring

AT TIME OF DRILLING ---

LOGGED BY ZKBoswell **CHECKED BY**

AT END OF DRILLING ---

NOTES

AFTER DRILLING ---

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)	GRAPHIC LOG		MATERIAL DESCRIPTION	
0					0.8	ASPHALT	12.3
	SS S-1	67	70-32-10-16 (42)		2.5	FILL:dessicated concrete; gray; dense; angular; trace gravel; trace silt	10.5
	SS S-2	67	16-20-19-17 (39)		5.0	Clayey SILT: olive; hard; dry; low plasticity	8.0
5	SS S-3	67	20-22-21-21 (43)			GLACIAL TILL: sandy gravel; brown; dense; fine to coarse grained sand; poorly graded; rounded; trace to little clayey silt	
10	SS S-4	58	15-19-18-16 (37)		12.0		1.0

Bottom of borehole at 12.0 feet.

Note 1: Ground surface elevation estimated from a draft plan entitled "Transects-Plan View" prepared by Horsley Witten Group, Inc., and dated September 23, 2016 and field measurements taken in August 2016 and June 2018. The elevation datum is North American Vertical Datum 1988 (NAVD 88).

Note 2: Drilled borehole with casing to approximately 10 ft bgs.

Note 3: Borehole backfilled with drill cuttings and an asphalt cold patch was placed at the ground surface.

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BORING NUMBER SGH-2018-7

CLIENT Horsley & Witten Group
PROJECT NAME Ipswich Mills Dam Removal, Supplemental Borings
PROJECT NUMBER 160630.01
PROJECT LOCATION Ipswich MA
DATE STARTED 6/2/18 **COMPLETED** 6/2/18
GROUND ELEVATION 12 ft NAVD88 **HOLE SIZE** 4 ID/4.5 OD inches
DRILLING CONTRACTOR Carr-Dee Corp. of MA
GROUND WATER LEVELS:
DRILLING METHOD Case and Wash Boring
AT TIME OF DRILLING ---
LOGGED BY ZKBoswell **CHECKED BY**
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)	GRAPHIC LOG	MATERIAL DESCRIPTION
0	SS S-1	25	1-3-4-5 (7)		FILL: silty sand; brown; loose; dry; fine to coarse grained; poorly graded; sub-angular; some silt; trace gravel; trace brick; trace cinders; trace ash
	SS S-2	33	7-5-6-6 (11)		
5				5.0	7.0
	SS S-3	46	9-20-30-100 (50)		GLACIAL TILL: clay silt; brown; hard; some gravel; little fine to coarse sand; Observed very hard drilling at approximately 7.5 ft bgs. Rig was lifting off the ground. Split spoon refusal at approximately 7.5 ft bgs.
				7.5	4.5
	SS S-4		100/0"		Bottom of borehole at 7.5 feet.

Note 1: Ground surface elevation estimated from a draft plan entitled "Transects-Plan View" prepared by Horsley Witten Group, Inc., and dated September 23, 2016 and field measurements taken in August 2016 and June 2018. The elevation datum is North American Vertical Datum 1988 (NAVD 88).

Note 2: Drilled borehole with casing to approximately 7.5 ft bgs.

Note 3: Borehole backfilled with drill cuttings.

Appendix C

2018 Laboratory Test Results

THIELSCH ENGINEERING	195 Frances Avenue Cranston RI, 02910 Phone: (401)-467-6454 Fax: (401)-467-2398 http://www.thielsch.com *Let's Build a Solid Foundation*	Client Information: Simpson Gumpertz & Heger, Inc Waltham, MA PM: Steve Keppel Assigned By: Steve Keppel Collected By: SK and ZB	Project Information: Ipswitch- Supplemental Limited Subsurface Investigation Ipswitch, MA SGH Project Number: 160630.01 Summary Page: 1 of 3 Report Date: 06.13.18

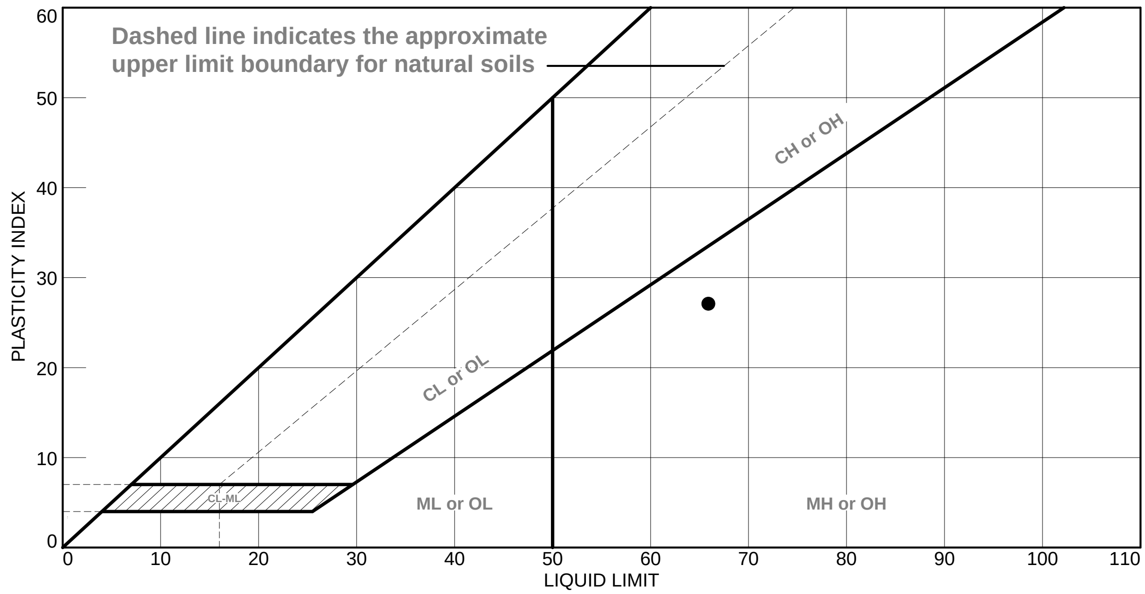
LABORATORY TESTING DATA SHEET

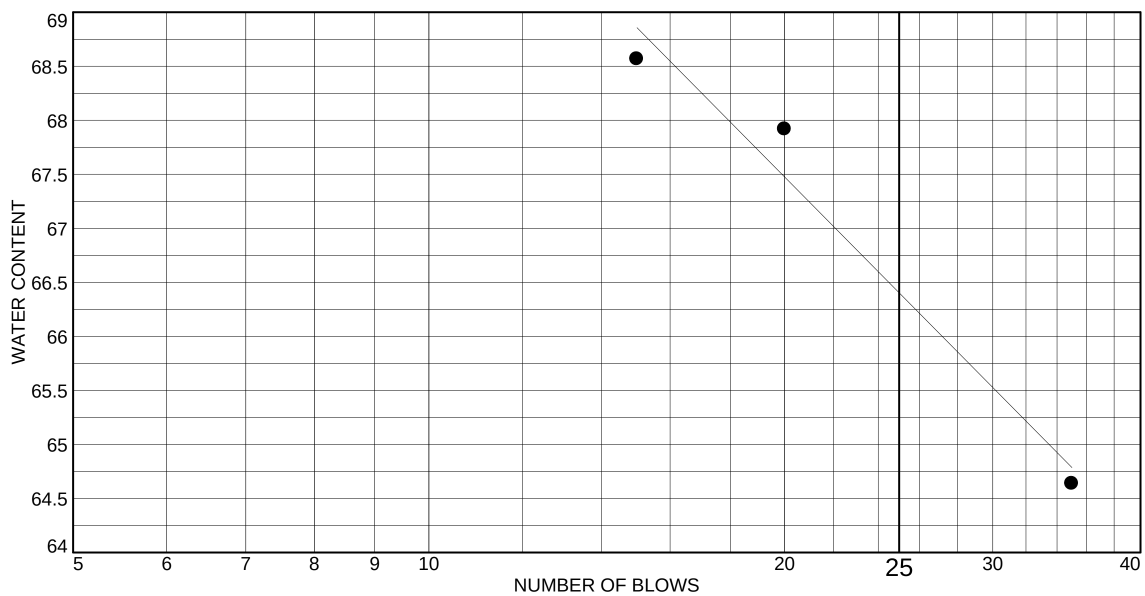
				Identification Tests								Proctor / CBR / Permeability Tests								
Boring ID	Sample No.	Depth (ft)	Laboratory No.	Water Content %	LL %	PL %	Gravel %	Sand %	Fines %	Org. %	G_s	Dry unit wt. pcf	Test Water Content %	γ_d MAX (pcf) W_{opt} (%)	γ_d MAX (pcf) W_{opt} (%) (Corr.)	Test Setup as % of Proctor	CBR @ 0.1"	CBR @ 0.2"	Perme-ability cm/sec	Laboratory Log and Soil Description
SGH-2018-3	S-5	10-11	18-S-737	68.3	66	39				10.3										Dark Brown Organic Silt (OH)
SGH-2018-3	S-6	12-13	18-S-738	33.1	42	21														Light Brown Lean Clay (CL)

Reviewed By ______________________

Date Reviewed 06.13.2018

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Dark Brown Organic Silt (OH)	66	39	27			

Project No. 160630.01 **Client:** Simpson Gumpertz & Heger

Project: Ipswitch - Suppplemental Limitied Subsurface Investigation

10 Estes Street

Source of Sample: SGH-2018-3 **Depth:** 10-11'

Sample Number: S-5

Remarks:

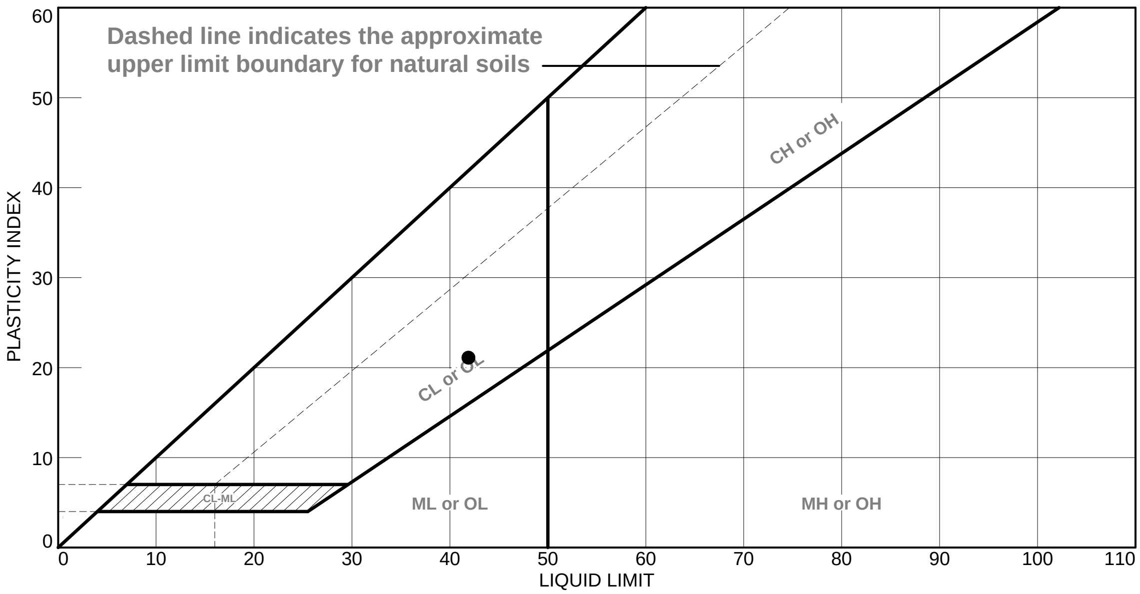
Thielsch Engineering Inc.

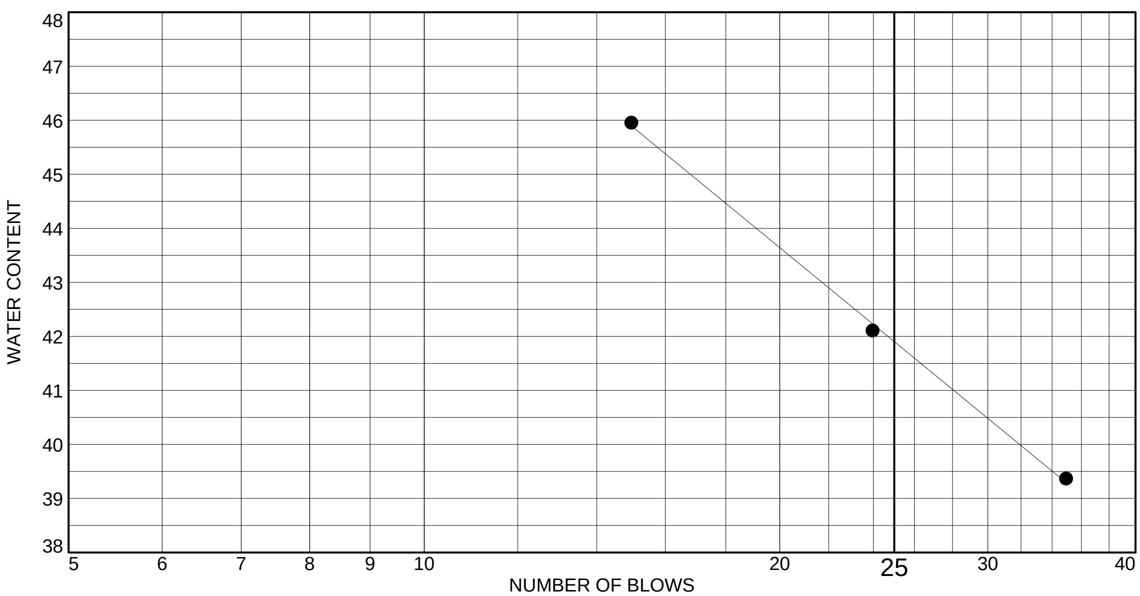
Cranston, RI

Figure 18-L-737

Tested By: MN **Checked By:** RR

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Light Brown Lean Clay (ML)	42	21	21			

Project No. 160630.01 **Client:** Simpson Gumpertz & Heger

Project: Ipswitch - Suppplemental Limitied Subsurface Investigation

10 Estes Street

Source of Sample: SGH-2018-3 **Depth:** 12-13'

Sample Number: S-6

Thielsch Engineering Inc.

Cranston, RI

Remarks:

Figure 18-L-738

Tested By: MN **Checked By:** RR

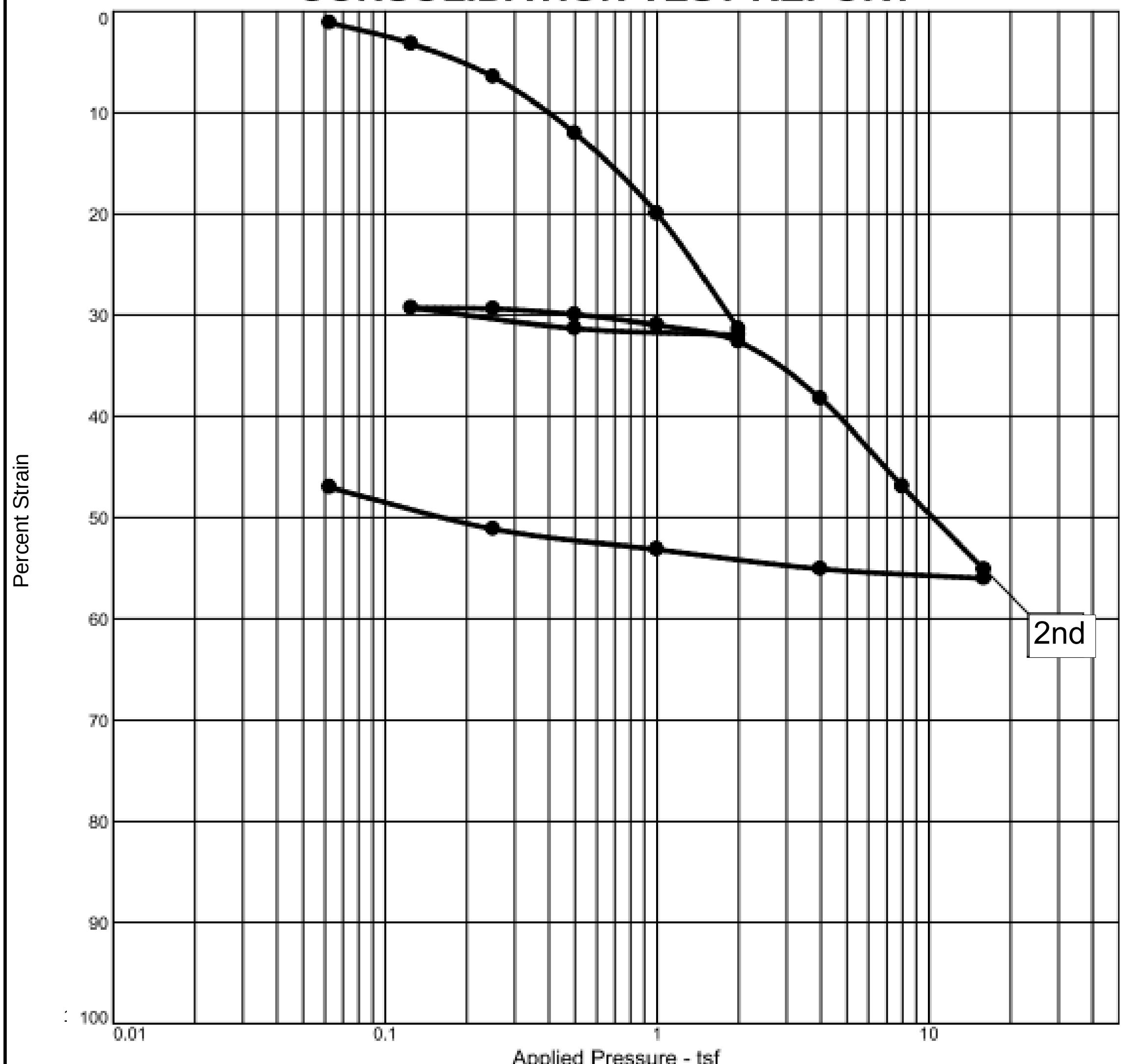
THIELSCH ENGINEERING	195 Frances Avenue Cranston RI, 02910 Phone: (401)-467-6454 Fax: (401)-467-2398 http://www.thielsch.com *Let's Build a Solid Foundation*	Client Information: Simpson Gumpertz & Heger, Inc Waltham, MA PM: Steve Keppel Assigned By: Steve Keppel Collected By: SK and ZB	Project Information: Ipswitch- Supplemental Limited Subsurface Investigation Ipswitch, MA SGH Project Number: 160630.01 Summary Page: 2 of 3 Report Date: 06.14.18

LABORATORY TESTING DATA SHEET

				Identification Tests								Shear / Consolidation Tests								
Boring ID	Sample No.	Depth (ft)	Laboratory No.	Water Content %	LL %	PL %	Gravel %	Sand %	Fines %	Org. %	G_s	Dry unit wt. pcf	Torvane or Type Test	$\bar{\sigma}_c$ psf	Failure Criteria	$\sigma_1 - \sigma_3$ or τ psf	Strain %	EST. Internal Friction Angle	CR / RR	Laboratory Log and Soil Description
SGH-2018-2A	US-1	8-10'	18-T-735		Average Total Unit Weight (8-10') = 102.8 pcf															
		8.0-8.25																		Loose #4 gravel
		8.25-8.7																		Grey lean clay with gravel
		8.7-9.2		23.0																Dark brown organic silt with sand and gravel
		9.2-9.65																		Dark brown organic silt with sand and gravel * Photographed
		9.3-9.4		84.8						11.0										Dark brown organic silt and sand
		9.4-9.6		142.8								32.3	Cons						0.29 / 0.044	Dark brown organic silt and sand

Reviewed By ____________________ Date Reviewed *06.15.2018*

CONSOLIDATION TEST REPORT



MATERIAL DESCRIPTION	USCS	AASHTO
Dark Brown organic silt		

LL	PI	Sp. Gr.	Overburden (tsf)	Dry Dens. (pcf)		Moisture		Saturation		Void Ratio		P_c (tsf)	C_c
				Init.	Final	Init.	Final	Init.	Final	Init.	Final		
NV	NP	2.0		32.3	60.9	142.8	69.8	99.6 %	100.0	2.870	1.050	0.6	1.18

Preparation Process: Trimmed using cutting ring

Condition of Test: Saturated at 2 tsf

D2435 Method	C_r	Swell Press. (tsf)	%
B	0.17		

Project No. 160630.01 **Client:** Simpson Gumpertz & Heger

Project: Ipswitch - Suppplemental Limitied Subsurface Investigation
10 Estes Street

Source: SGH-2018-2A **Depth:** 8-10' **Sample No.:** US-1

Thielsch Engineering Inc.

Cranston, RI

Remarks:
End of Primary Test at 9.45-9.55'.
Assumed specific gravity to be 2.0.

Checked By: sa

Title: Laboratory Manager

Figure C-735-1

Tested By: RR

CONSOLIDATION TEST DATA

6/14/2018

Client: Simpson Gumpertz & Heger
Project: Ipswitch - Suppplemental Limitied Subsurface Investigation
10 Estes Street
Ipswich, MA
Project Number: 160630.01
Location: SGH-2018-2A
Depth: 8-10' **Sample Number:** US-1
Material Description: Dark Brown organic silt
Liquid Limit: NV **Plasticity Index:** NP
Preparation Process: Trimmed using cutting ring
Condition of Test: Saturated at 2 tsf
Test Method: B **Final Density:** 60.9 **Figure No.:** C-735-1
Testing Remarks: End of Primary Test at 9.45-9.55'. Assumed specific gravity to be 2.0.
Tested By: RR **Checked by:** sa **Title:** Laboratory Manager

Test Specimen Data

NATURAL MOISTURE		VOID RATIO		AFTER TEST	
Wet w+t	= 205.48 g.	Spec. Gr.	= 2.0	Wet w+t	= 117.77 g.
Dry w+t	= 114.26 g.	Est. Ht. Solids	= 0.258 in.	Dry w+t	= 90.24 g.
Tare Wt.	= 50.40 g.	Init. V.R.	= 2.870	Tare Wt.	= 50.80 g.
Moisture	= 142.8 %	Init. Sat.	= 99.6 %	Moisture	= 69.8 %
UNIT WEIGHT		**TEST START**		Dry Wt.	= 39.44 g.
Height	= 1.000 in.	Height	= 1.000 in.		
Diameter	= 2.500 in.	Diameter	= 2.500 in.		
Weight	= 100.96 g.				
Dry Dens.	= 32.3 pcf				

End-Of-Load Summary

Pressure (tsf)	Final Dial (in.)	Deformation (in.)	C_v (cm.^{2}/sec.)	C_α	Void Ratio	% Strain
start	0.00394	0.00000			2.870	
0.06	0.01516	0.01122	0.0110		2.826	1.1 Comprs.
0.13	0.03592	0.03198	0.0124		2.746	3.2 Comprs.
0.25	0.06866	0.06472	0.0037		2.619	6.5 Comprs.
0.50	0.12440	0.12046	0.0031		2.404	12.0 Comprs.
1.00	0.20370	0.19976	0.0024		2.097	20.0 Comprs.
2.00	0.32350	0.31956	0.0016		1.633	32.0 Comprs.
0.50	0.31710	0.31316			1.658	31.3 Comprs.
0.13	0.29690	0.29296			1.736	29.3 Comprs.
0.25	0.29780	0.29386	0.0044		1.733	29.4 Comprs.
0.50	0.30360	0.29966	0.0068		1.710	30.0 Comprs.
1.00	0.31380	0.30986	0.0063		1.671	31.0 Comprs.
2.00	0.33020	0.32626	0.0030		1.607	32.6 Comprs.
4.00	0.38630	0.38236	0.0018		1.390	38.2 Comprs.
8.00	0.47310	0.46916	0.0014		1.054	46.9 Comprs.
16.00	0.56420	0.56026	0.0009		0.702	56.0 Comprs.
4.00	0.55470	0.55076			0.738	55.1 Comprs.
1.00	0.53550	0.53156			0.813	53.2 Comprs.
0.25	0.51540	0.51146			0.891	51.1 Comprs.
0.06	0.47410	0.47016			1.050	47.0 Comprs.

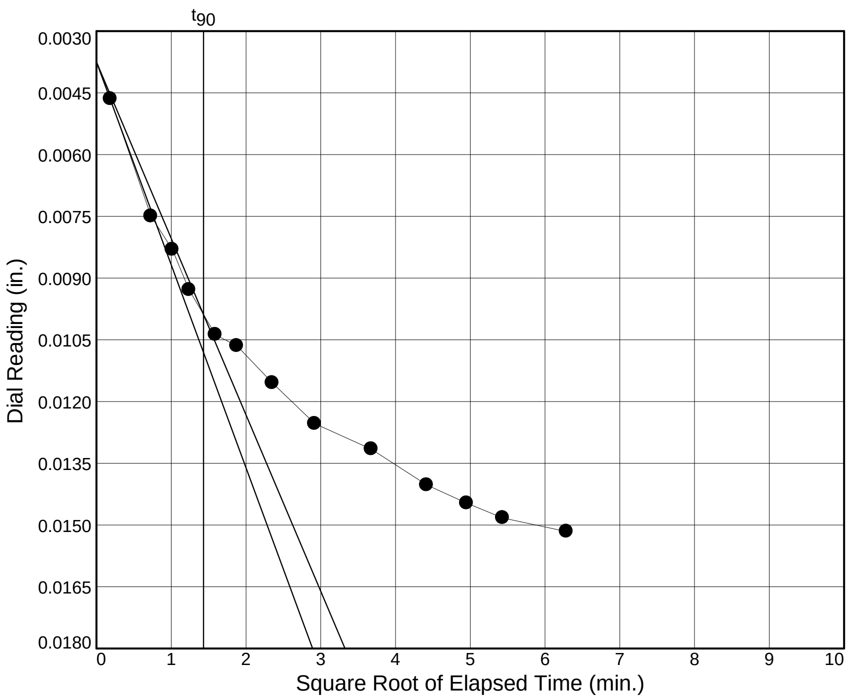
TEST RESULTS SUMMARY

Compression index (C_c), tsf = 1.18 **Preconsolidation pressure (P_p), tsf =** 0.6 **Void ratio at P_p (e_m) =** 2.327

Recompression index (C_r) = 0.17

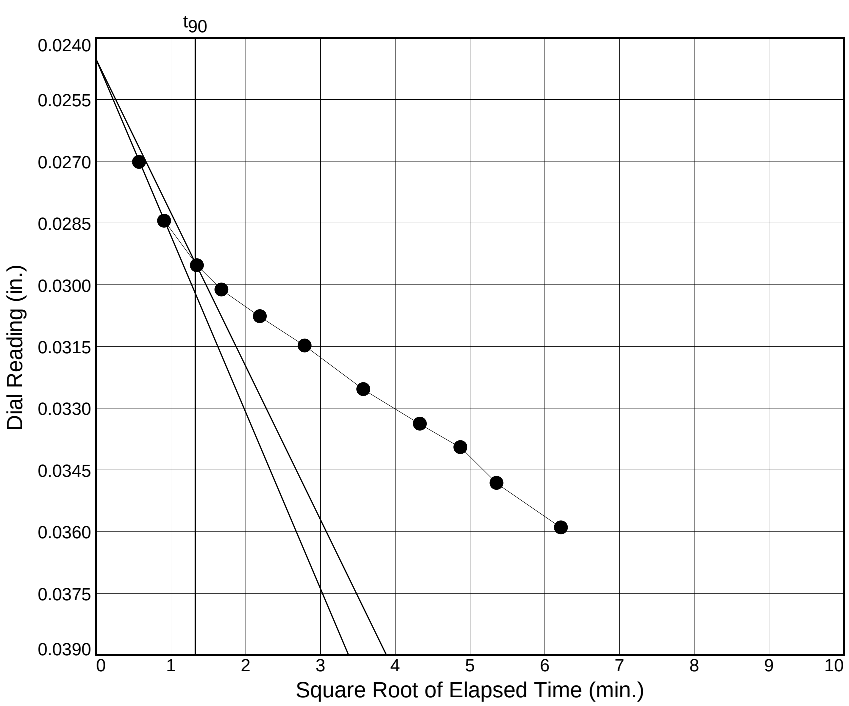
Dial Reading vs. Time

Project No.: 160630.01
Project: Ipswitch - Suppplemental Limitied Subsurface Investigation
10 Estes Street
Source of Sample: SGH-2018-2A Depth: 8-10' Sample Number: US-1



Load No.= 1
Load= 0.06 tsf
D_0 = 0.0037
D_{90} = 0.0099
D_{100} = 0.0106
T_{90} = 2.05 min.

C_v @ T_{90}
0.0110 cm.^{2}/sec.



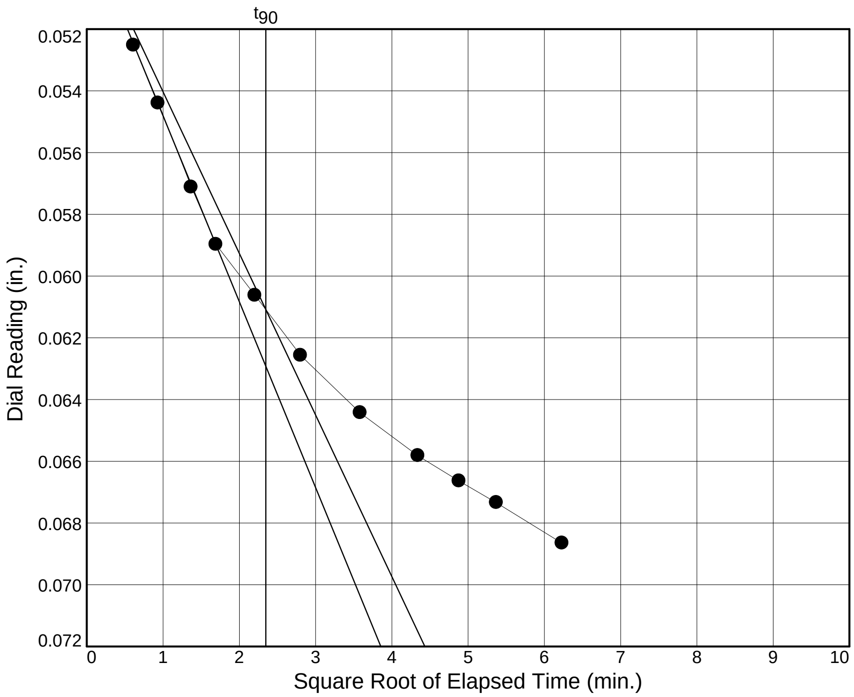
Load No.= 2
Load= 0.13 tsf
D_0 = 0.0245
D_{90} = 0.0295
D_{100} = 0.0300
T_{90} = 1.75 min.

C_v @ T_{90}
0.0124 cm.^{2}/sec.

Figure C-735-2

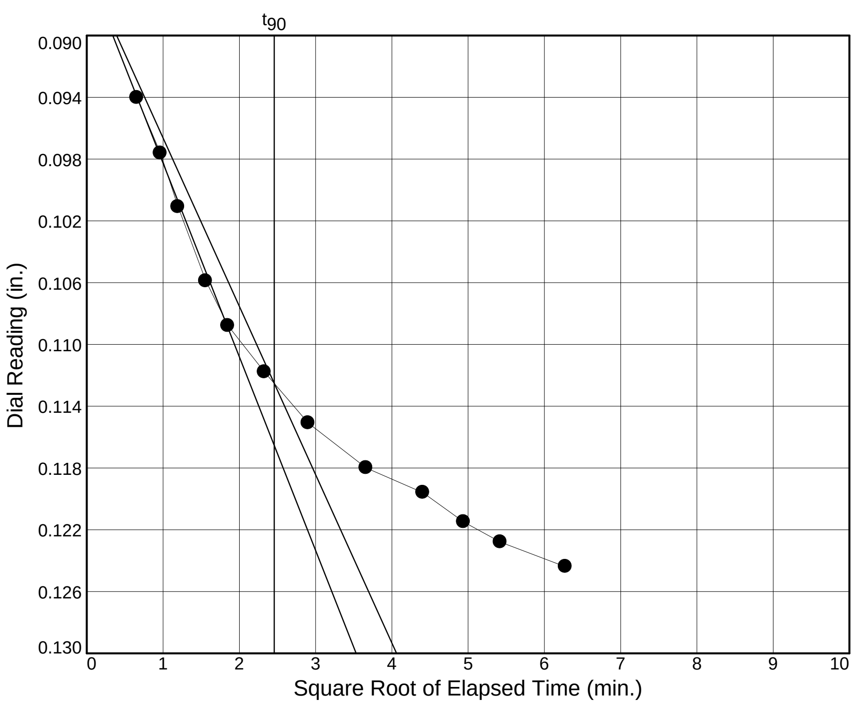
Dial Reading vs. Time

Project No.: 160630.01
Project: Ipswitch - Suppplemental Limitied Subsurface Investigation
10 Estes Street
Source of Sample: SGH-2018-2A   Depth: 8-10'   Sample Number: US-1



Load No.= 3
Load= 0.25 tsf
D_0 = 0.0488
D_{90} = 0.0611
D_{100} = 0.0625
T_{90} = 5.51 min.

C_v @ T_{90}
0.0037 cm.^{2}/sec.



Load No.= 4
Load= 0.50 tsf
D_0 = 0.0857
D_{90} = 0.1125
D_{100} = 0.1155
T_{90} = 6.04 min.

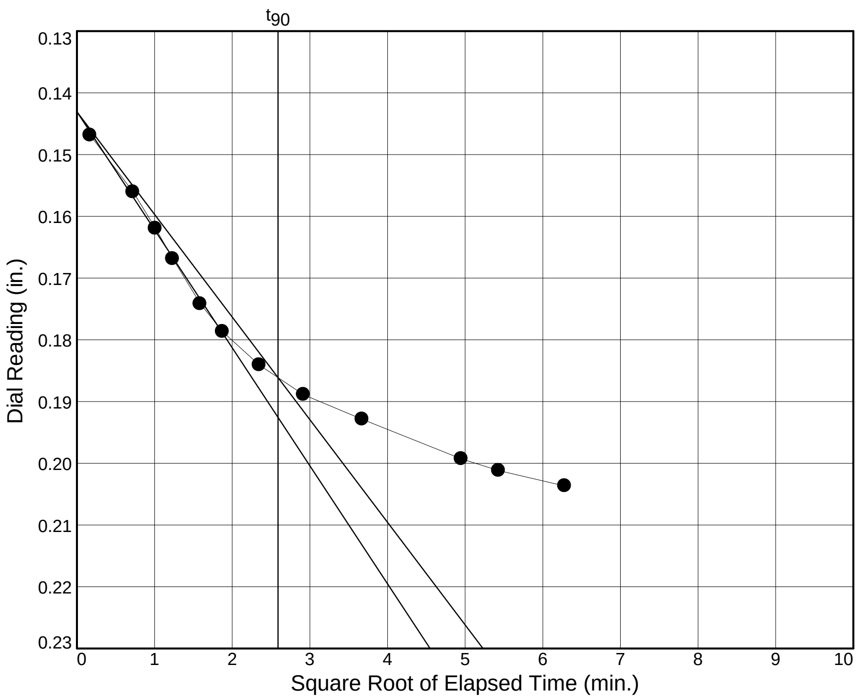
C_v @ T_{90}
0.0031 cm.^{2}/sec.

Figure C-735-3

Thielsch Engineering Inc.

Dial Reading vs. Time

Project No.: 160630.01
Project: Ipswitch - Suppplemental Limitied Subsurface Investigation
10 Estes Street
Source of Sample: SGH-2018-2A Depth: 8-10' Sample Number: US-1



Load No.= 5

Load= 1.00 tsf

D_0 = 0.1431

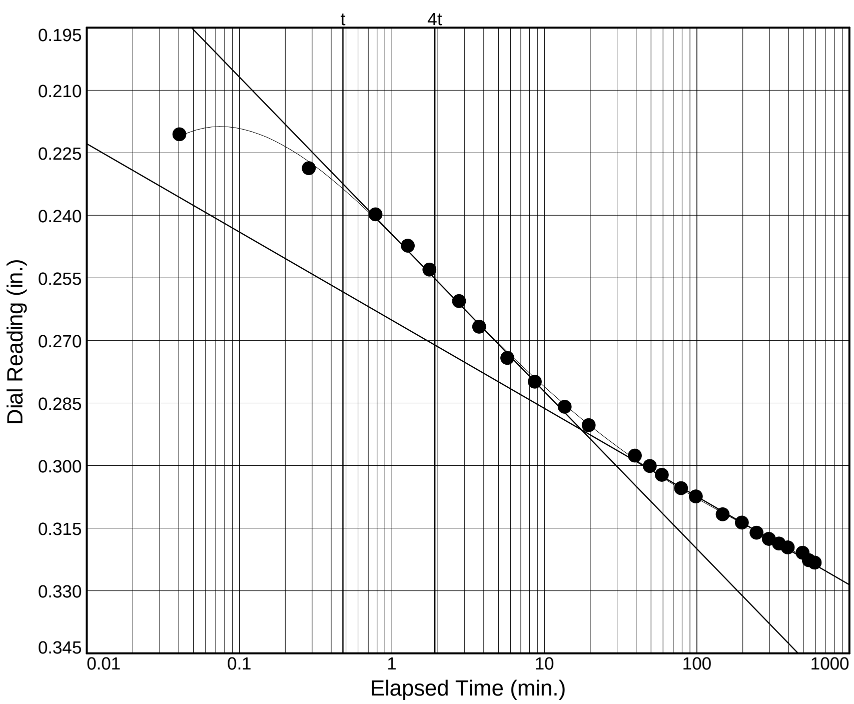
D_{90} = 0.1861

D_{100} = 0.1909

T_{90} = 6.71 min.

C_v @ T_{90}

0.0024 $cm.^2$/sec.



Load No.= 6

Load= 2.00 tsf

D_0 = 0.2121

D_{50} = 0.2518

D_{100} = 0.2914

T_{50} = 1.56 min.

C_v @ T_{50}

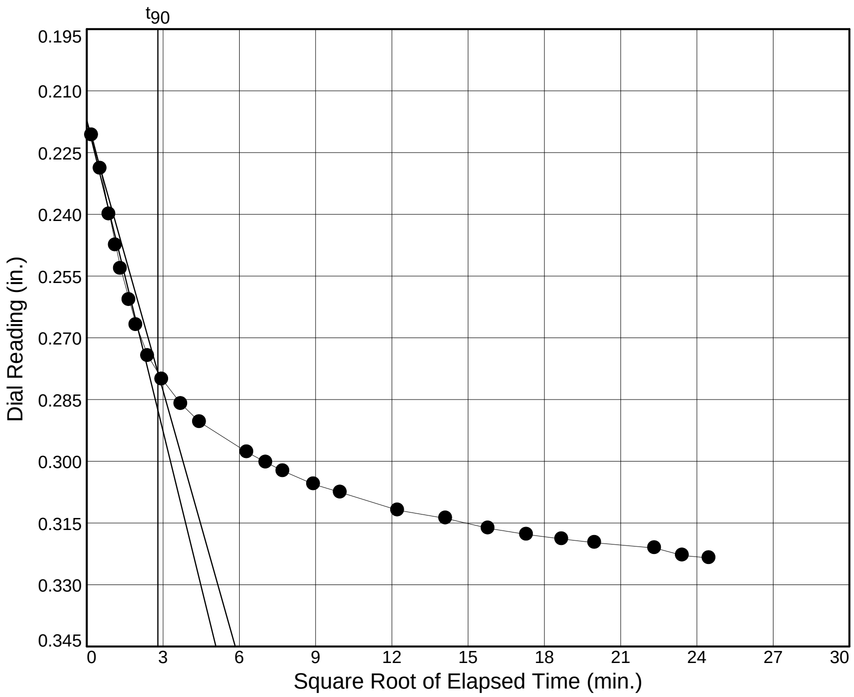
0.0019 $cm.^2$/sec.

C_α = 0.021

Figure C-735-4

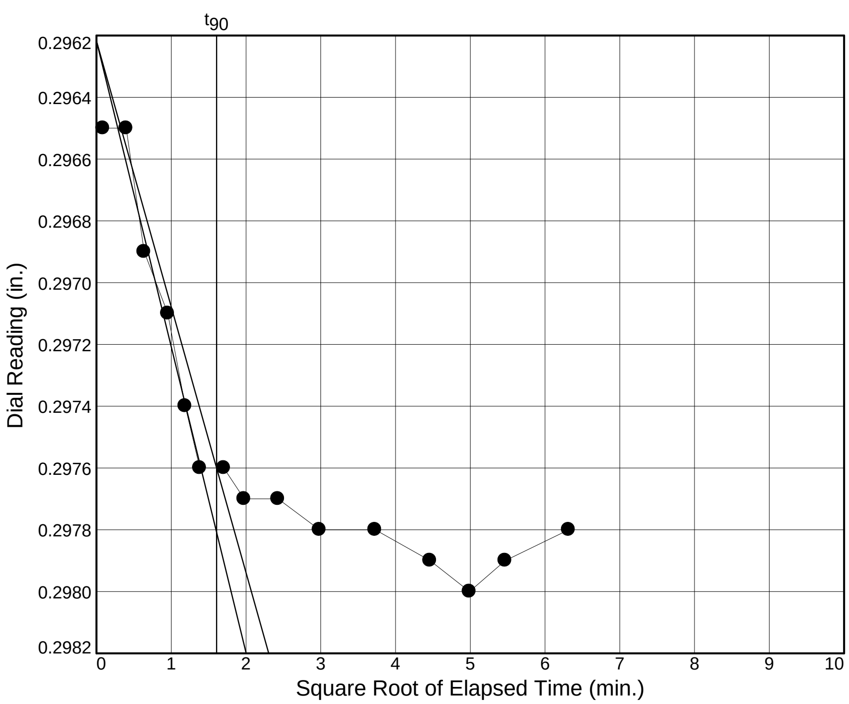
Dial Reading vs. Time

Project No.: 160630.01
Project: Ipswitch - Suppplemental Limitied Subsurface Investigation
10 Estes Street
Source of Sample: SGH-2018-2A   Depth: 8-10'   Sample Number: US-1



Load No.= 6
Load= 2.00 tsf
D_0 = 0.2172
D_{90} = 0.2784
D_{100} = 0.2852
T_{90} = 7.81 min.

C_v @ T_{90}
0.0016 cm.^{2}/sec.



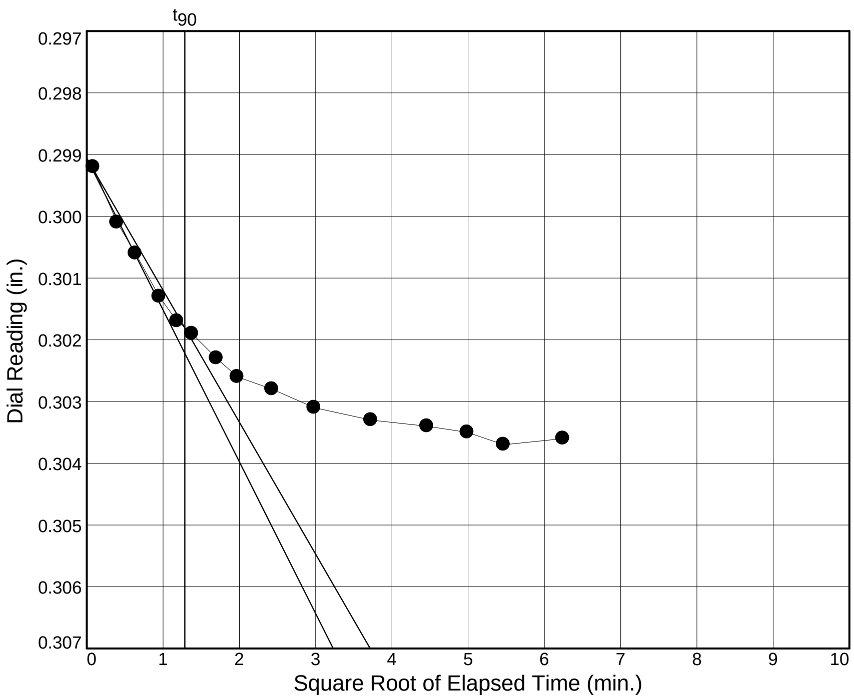
Load No.= 9
Load= 0.25 tsf
D_0 = 0.2962
D_{90} = 0.2976
D_{100} = 0.2978
T_{90} = 2.58 min.

C_v @ T_{90}
0.0044 cm.^{2}/sec.

Figure C-735-5

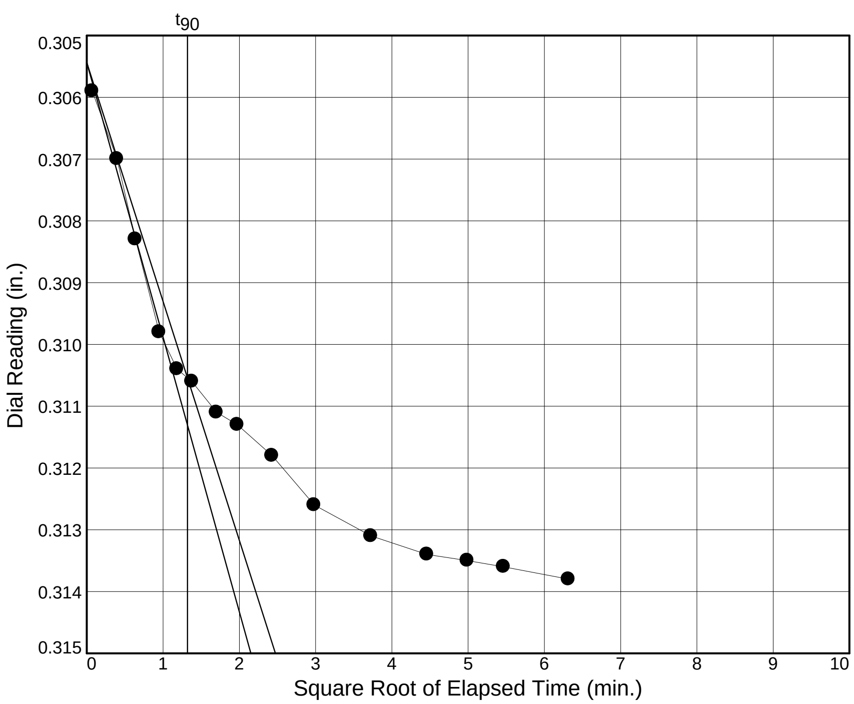
Dial Reading vs. Time

Project No.: 160630.01
Project: Ipswitch - Suppplemental Limitied Subsurface Investigation
10 Estes Street
Source of Sample: SGH-2018-2A Depth: 8-10' Sample Number: US-1



Load No.= 10
Load= 0.50 tsf
D_0 = 0.2991
D_{90} = 0.3018
D_{100} = 0.3021
T_{90} = 1.65 min.

C_v @ T_{90}
0.0068 cm.^{2}/sec.



Load No.= 11
Load= 1.00 tsf
D_0 = 0.3054
D_{90} = 0.3105
D_{100} = 0.3111
T_{90} = 1.74 min.

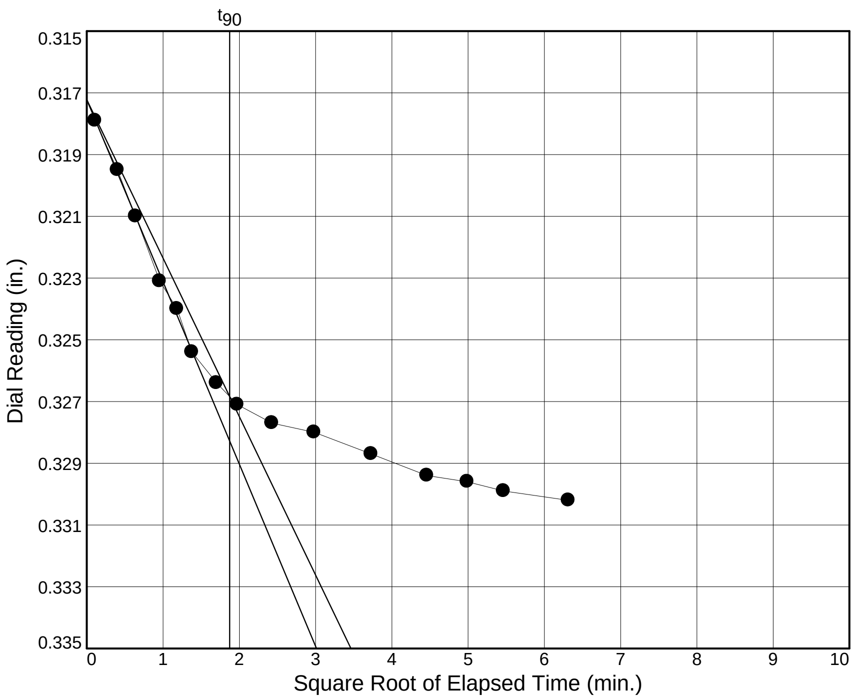
C_v @ T_{90}
0.0063 cm.^{2}/sec.

Figure C-735-6

Thielsch Engineering Inc.

Dial Reading vs. Time

Project No.: 160630.01
Project: Ipswitch - Suppplemental Limitied Subsurface Investigation
10 Estes Street
Source of Sample: SGH-2018-2A Depth: 8-10' Sample Number: US-1



Load No.= 12

Load= 2.00 tsf

D_0 = 0.3172

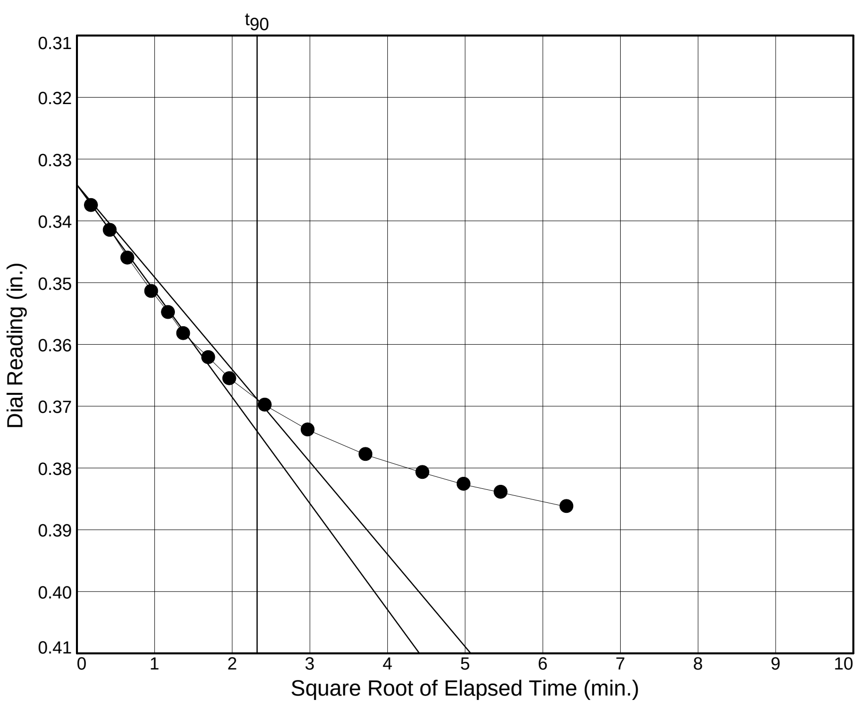
D_{90} = 0.3268

D_{100} = 0.3279

T_{90} = 3.51 min.

C_v @ T_{90}

0.0030 cm.^{2}/sec.



Load No.= 13

Load= 4.00 tsf

D_0 = 0.3342

D_{90} = 0.3689

D_{100} = 0.3727

T_{90} = 5.39 min.

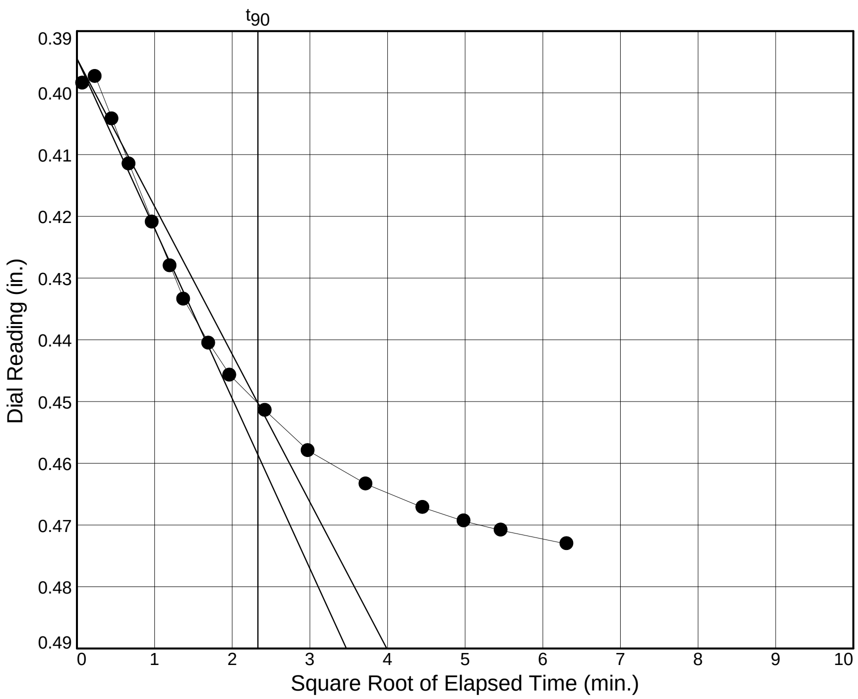
C_v @ T_{90}

0.0018 cm.^{2}/sec.

Figure C-735-7

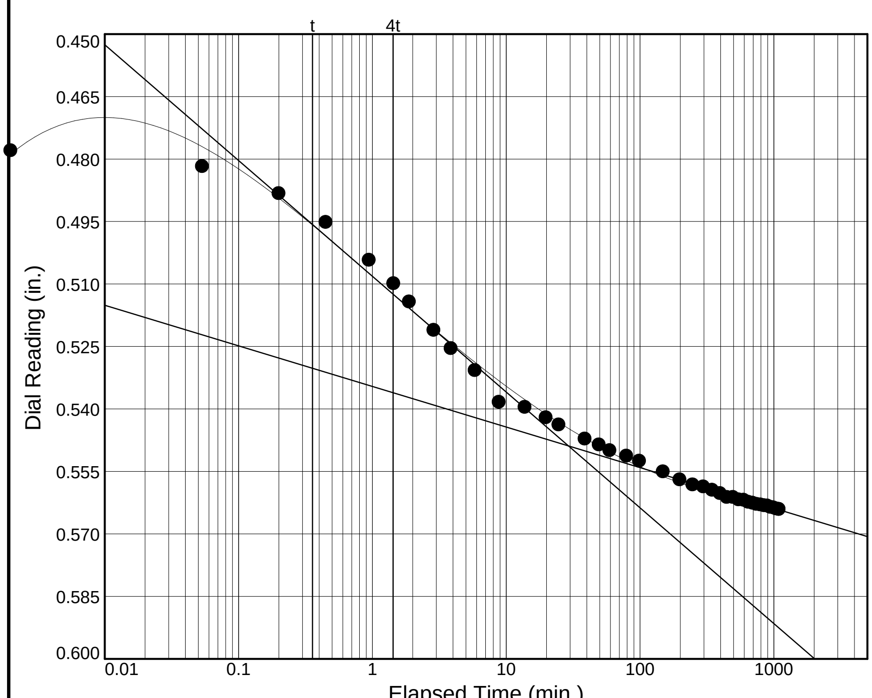
Dial Reading vs. Time

Project No.: 160630.01
Project: Ipswitch - Suppplemental Limitied Subsurface Investigation
10 Estes Street
Source of Sample: SGH-2018-2A   Depth: 8-10'   Sample Number: US-1



Load No.= 14
Load= 8.00 tsf
D_0 = 0.3945
D_{90} = 0.4503
D_{100} = 0.4565
T_{90} = 5.43 min.

C_v @ T_{90}
0.0014 $cm.^2$/sec.



Load No.= 15
Load= 16.00 tsf
D_0 = 0.4793
D_{50} = 0.5141
D_{100} = 0.5489
T_{50} = 1.64 min.

C_v @ T_{50}
0.0008 $cm.^2$/sec.

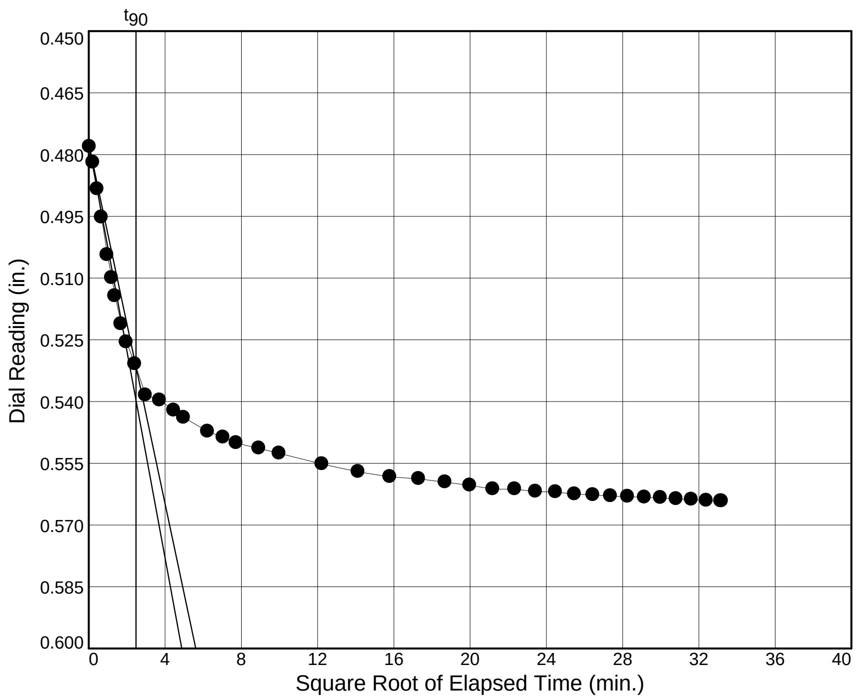
C_α = 0.010

Figure C-735-8

Thielsch Engineering Inc.

Dial Reading vs. Time

Project No.: 160630.01
Project: Ipswitch - Suppplemental Limitied Subsurface Investigation
10 Estes Street
Source of Sample: SGH-2018-2A   Depth: 8-10'   Sample Number: US-1



Load No.= 15

Load= 16.00 tsf

D_0 = 0.4775

D_{90} = 0.5315

D_{100} = 0.5375

T_{90} = 6.13 min.

C_V @ T_{90}

0.0009 cm.^{2}/sec.

Figure C-735-9

Thielsch Engineering Inc.

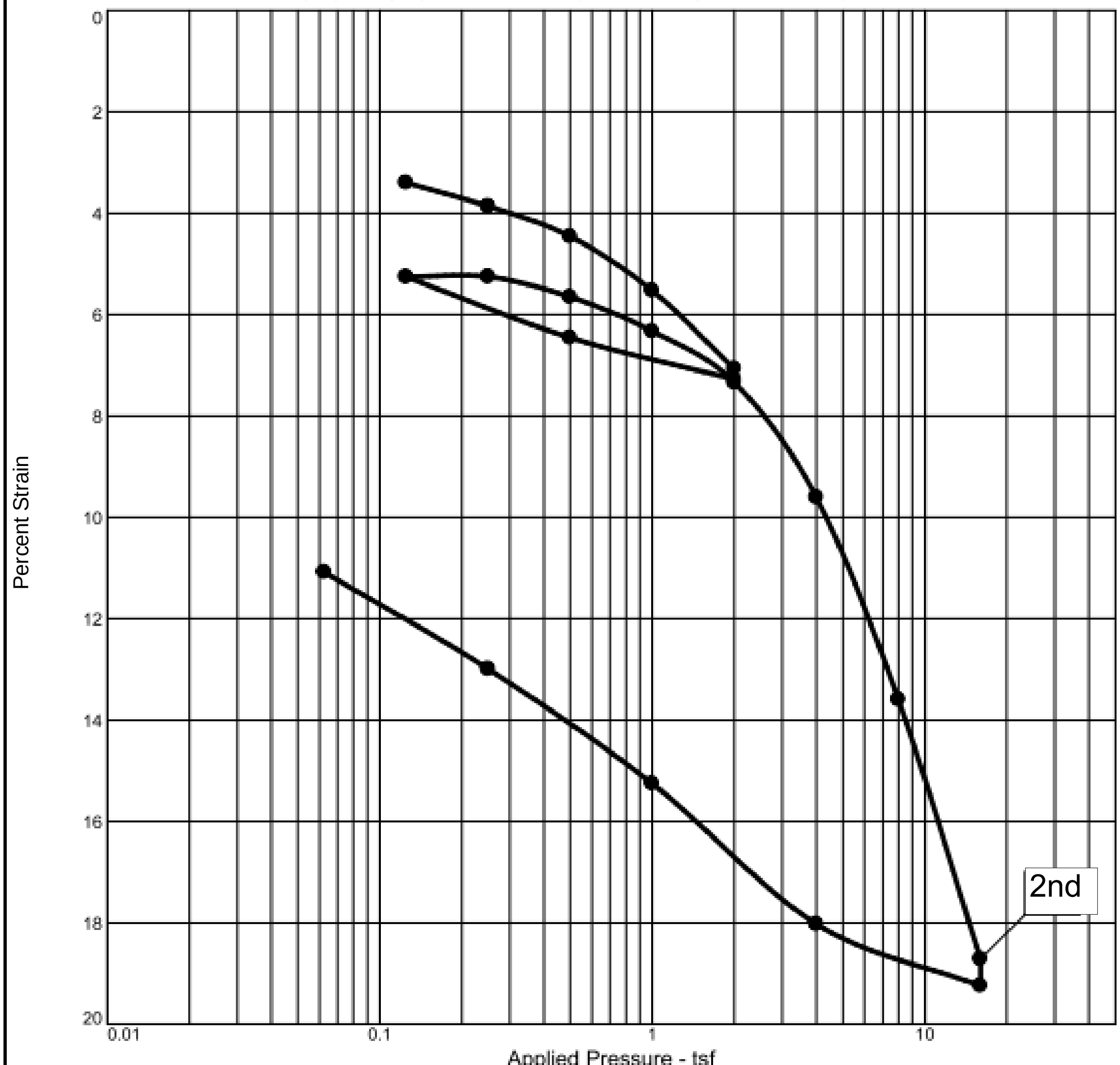
THIELSCH ENGINEERING	195 Frances Avenue Cranston RI, 02910 Phone: (401)-467-6454 Fax: (401)-467-2398 http://www.thielsch.com *Let's Build a Solid Foundation*	Client Information: Simpson Gumpertz & Heger, Inc Waltham, MA PM: Steve Keppel Assigned By: Steve Keppel Collected By: SK and ZB	Project Information: Ipswitch- Supplemental Limited Subsurface Investigation Ipswitch, MA SGH Project Number: 160630.01 Summary Page: 3 of 3 Report Date: 06.14.18

LABORATORY TESTING DATA SHEET

Boring ID	Sample No.	Depth (ft)	Laboratory No.	Identification Tests								Shear / Consolidation Tests								Laboratory Log and Soil Description
				Water Content %	LL %	PL %	Gravel %	Sand %	Fines %	Org. %	G_s	Dry unit wt. pcf	Torvane or Type Test	$\bar{\sigma}_c$ psf	Failure Criteria	$\sigma_1 - \sigma_3$ or τ psf	Strain %	EST. Internal Friction Angle	CR / RR	
SGH-2018-2A	US-3	13.5-15.5	18-T-736		Average Total Unit Weight (13.5-15.5') = 117.1 pcf															Grey Varved Soil
		13.6-13.7		33.0									Tv = 0.40 tsf							Grading from lean clay to silty sand. Varves vary in thickness from 0.13" to 0.5"
		13.7-14.2		(SAVED)																Clay layers vary in thickness from 0.13" to over 1"
		14.5-14.7		32.3								85.8	Cons						0.19/ 0.026	Gray lean clay; Medium to stiff consistency
		14.7-15.2		(SAVED)																
		15.2-15.3		30.7									Tv = 0.55 tsf							
		15.3-15.5																		Light brown silt

Reviewed By ____________________ Date Reviewed *06.15.2018*

CONSOLIDATION TEST REPORT



MATERIAL DESCRIPTION	USCS	AASHTO
Grey lean clay		

LL	PI	Sp. Gr.	Overburden (tsf)	Dry Dens. (pcf)		Moisture		Saturation		Void Ratio		P_c (tsf)	C_c
				Init.	Final	Init.	Final	Init.	Final	Init.	Final		
		2.6		85.5	96.4	32.2 %	35.6	93.4 %	100.0	0.897	0.687	4.3	0.36

Preparation Process: Trimmed using cutting ring

Condition of Test: Saturated at 2 tsf

D2435 Method	C_r	Swell Press. (tsf)	%
B	0.05		

Project No. 160630.01 **Client:** Simpson Gumpertz & Heger

Project: Ipswitch - Suppplemental Limitied Subsurface Investigation
10 Estes Street

Source: SGH-2018-2A **Depth:** 13.5-15.5' **Sample No.:** US-3

Remarks:
End of Primary Test at 14.25-14.35'.
Assumed specific gravity to be 2.6.

Checked By: sa

Title: Laboratory Manager

Thielsch Engineering Inc.

Cranston, RI

Figure C-736-1

Tested By: RR

CONSOLIDATION TEST DATA

6/14/2018

Client: Simpson Gumpertz & Heger
Project: Ipswitch - Suppplemental Limitied Subsurface Investigation
10 Estes Street
Ipswich, MA
Project Number: 160630.01
Location: SGH-2018-2A
Depth: 13.5-15.5' **Sample Number:** US-3
Material Description: Grey lean clay
Preparation Process: Trimmed using cutting ring
Condition of Test: Saturated at 2 tsf
Test Method: B **Final Density:** 96.4 **Figure No.:** C-736-1
Testing Remarks: End of Primary Test specimen taken at 14.25-14.35'. Assumed specific gravity to be 2.6.
Tested By: RR **Checked by:** sa **Title:** Laboratory Manager

Test Specimen Data

NATURAL MOISTURE		VOID RATIO		AFTER TEST	
Wet w+t =	251.83 g.	**Spec. Gr.**	= 2.6	**Wet w+t =**	192.42 g.
Dry w+t =	202.65 g.	**Est. Ht. Solids**	= 0.527 in.	**Dry w+t =**	154.97 g.
Tare Wt. =	50.08 g.	**Init. V.R.**	= 0.897	**Tare Wt. =**	49.73 g.
Moisture =	32.2 %	**Init. Sat.**	= 93.4 %	**Moisture =**	35.6 %
UNIT WEIGHT		**TEST START**		**Dry Wt. =**	105.24 g.
Height =	1.000 in.	**Height**	= 1.000 in.		
Diameter =	2.500 in.	**Diameter**	= 2.500 in.		
Weight =	145.76 g.				
Dry Dens. =	85.5 pcf				

End-Of-Load Summary

Pressure (tsf)	Final Dial (in.)	Deformation (in.)	C_v ($cm.^2/sec.$)	C_α	Void Ratio	% Strain
start	0.00287	0.00000			0.897	
0.13	0.03686	0.03399	0.0285		0.833	3.4 Comprs.
0.25	0.04150	0.03863	0.0174		0.824	3.9 Comprs.
0.50	0.04746	0.04459	0.0103		0.813	4.5 Comprs.
1.00	0.05816	0.05529	0.0106		0.792	5.5 Comprs.
2.00	0.07562	0.07275	0.0070		0.759	7.3 Comprs.
0.50	0.06743	0.06456			0.775	6.5 Comprs.
0.13	0.05545	0.05258			0.798	5.3 Comprs.
0.25	0.05541	0.05254	0.0825		0.798	5.3 Comprs.
0.50	0.05940	0.05653	0.0337		0.790	5.7 Comprs.
1.00	0.06618	0.06331	0.0348		0.777	6.3 Comprs.
2.00	0.07626	0.07339	0.0101		0.758	7.3 Comprs.
4.00	0.09894	0.09607	0.0063		0.715	9.6 Comprs.
8.00	0.13885	0.13598	0.0046		0.639	13.6 Comprs.
16.00	0.19532	0.19245	0.0026		0.532	19.2 Comprs.
4.00	0.18312	0.18025			0.555	18.0 Comprs.
1.00	0.15543	0.15256			0.608	15.3 Comprs.
0.25	0.13282	0.12995			0.651	13.0 Comprs.
0.06	0.11371	0.11084			0.687	11.1 Comprs.

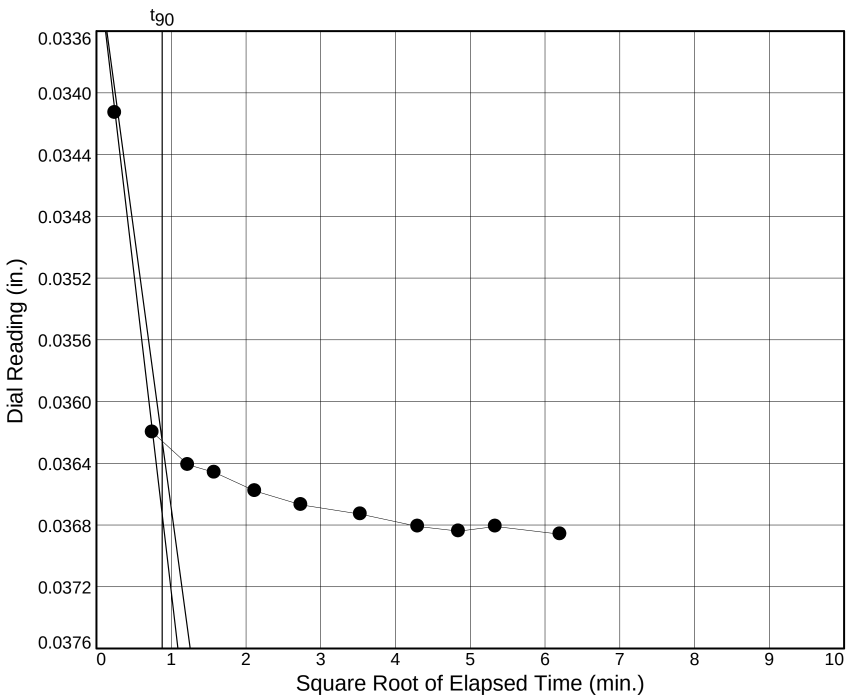
TEST RESULTS SUMMARY

Compression index (C_c), tsf = 0.36 **Preconsolidation pressure (P_p), tsf =** 4.3 **Void ratio at P_p (e_m) =** 0.709

Recompression index (C_r) = 0.05

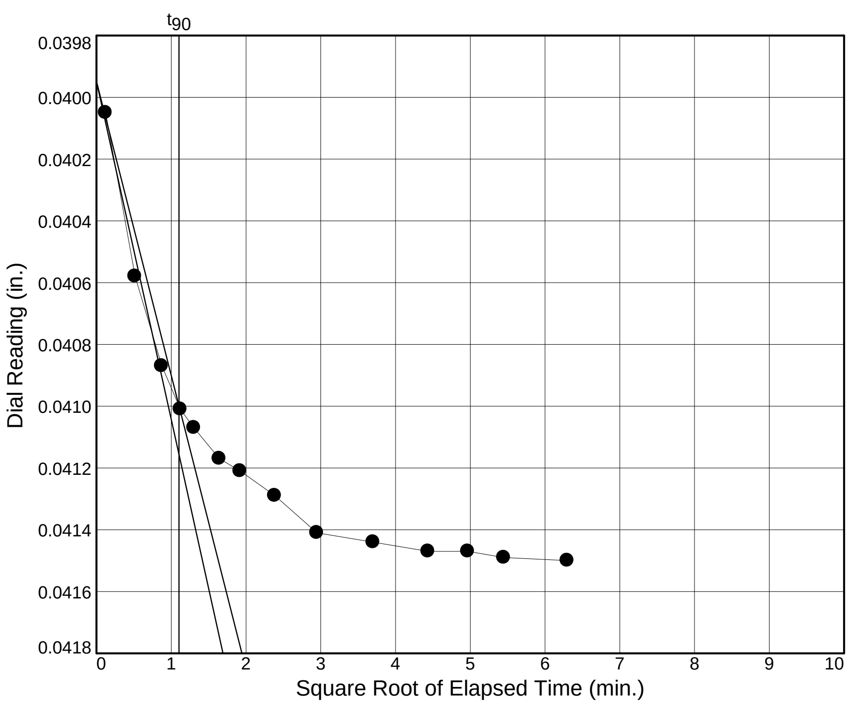
Dial Reading vs. Time

Project No.: 160630.01
Project: Ipswitch - Suppplemental Limitied Subsurface Investigation
10 Estes Street
Source of Sample: SGH-2018-2A Depth: 13.5-15.5' Sample Number: US-3



Load No.= 1
Load= 0.13 tsf
D_0 = 0.0331
D_{90} = 0.0363
D_{100} = 0.0366
T_{90} = 0.77 min.

C_v @ T_{90}
0.0285 cm.^{2}/sec.



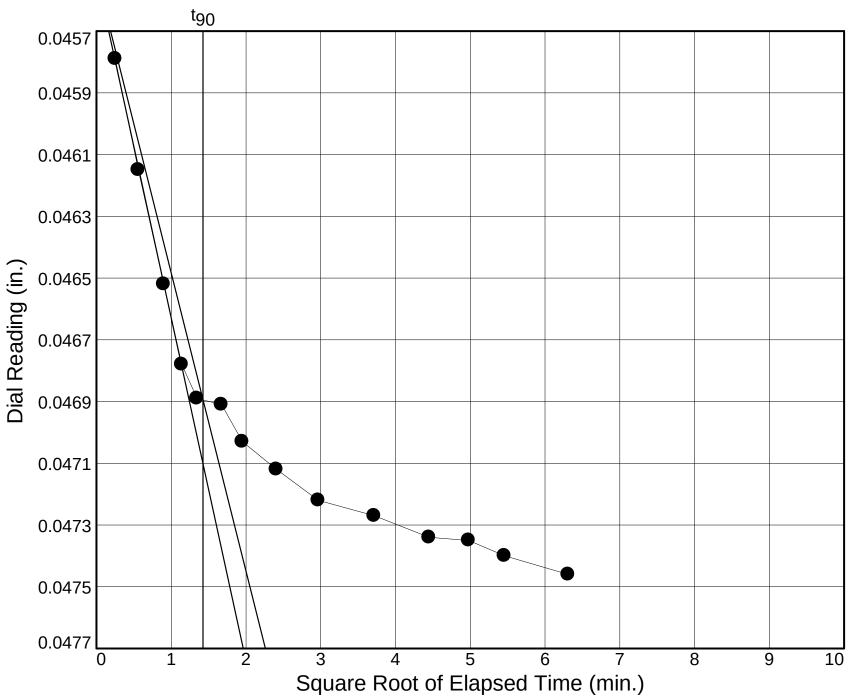
Load No.= 2
Load= 0.25 tsf
D_0 = 0.0399
D_{90} = 0.0410
D_{100} = 0.0411
T_{90} = 1.22 min.

C_v @ T_{90}
0.0174 cm.^{2}/sec.

Figure C-736-2

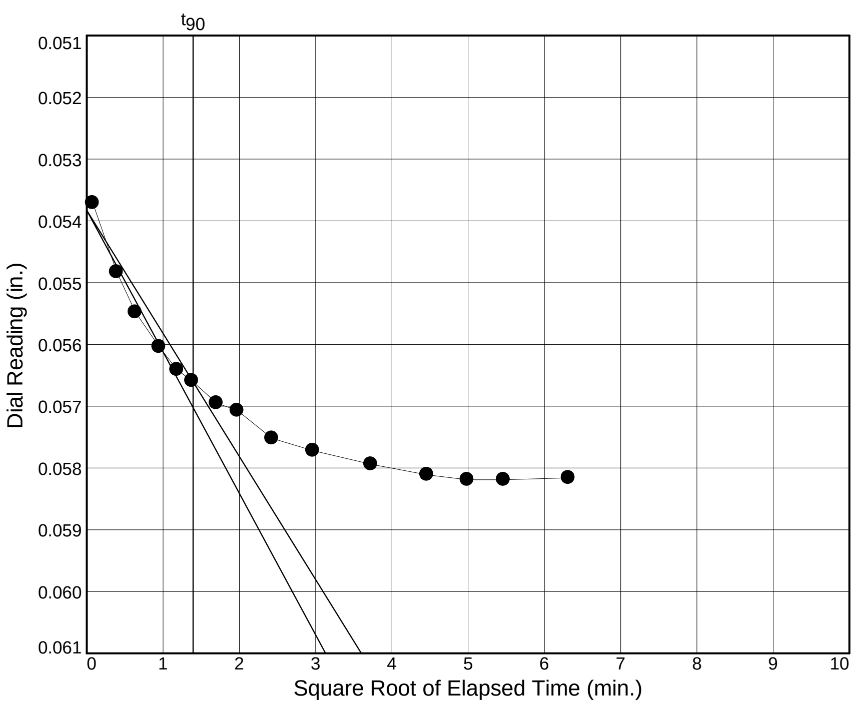
Dial Reading vs. Time

Project No.: 160630.01
Project: Ipswitch - Suppplemental Limitied Subsurface Investigation
10 Estes Street
Source of Sample: SGH-2018-2A Depth: 13.5-15.5' Sample Number: US-3



Load No.= 3
Load= 0.50 tsf
D_0 = 0.0455
D_{90} = 0.0469
D_{100} = 0.0470
T_{90} = 2.03 min.

C_v @ T_{90}
0.0103 cm.^{2}/sec.



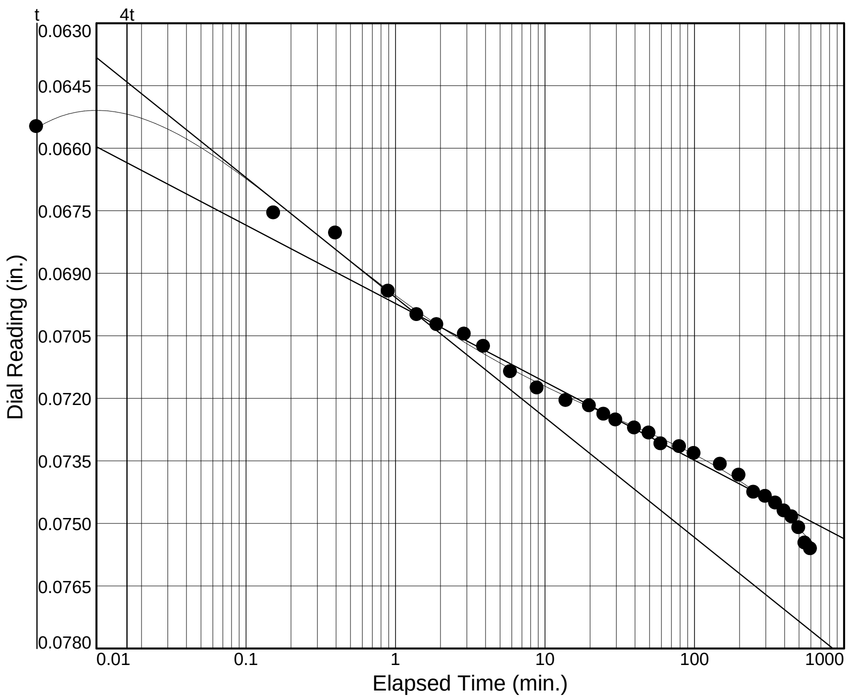
Load No.= 4
Load= 1.00 tsf
D_0 = 0.0538
D_{90} = 0.0566
D_{100} = 0.0569
T_{90} = 1.94 min.

C_v @ T_{90}
0.0106 cm.^{2}/sec.

Figure C-736-3

Dial Reading vs. Time

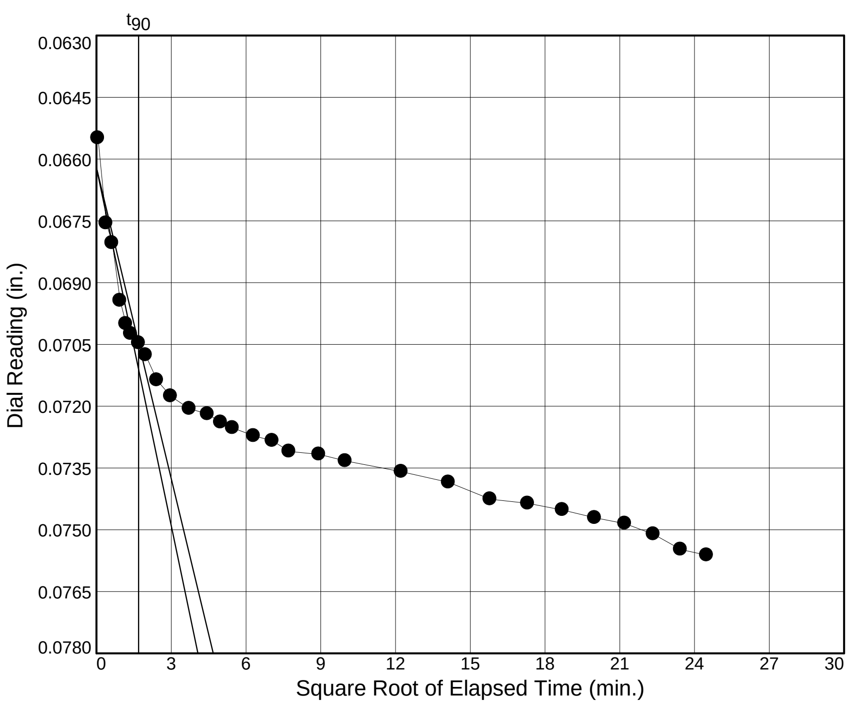
Project No.: 160630.01
Project: Ipswitch - Suppplemental Limitied Subsurface Investigation
10 Estes Street
Source of Sample: SGH-2018-2A Depth: 13.5-15.5' Sample Number: US-3



Load No.= 5
Load= 2.00 tsf
D_0 = 0.0658
D_{50} = 0.0679
D_{100} = 0.0700
T_{50} = 0.26 min.

C_v @ T_{50}
0.0175 cm.^{2}/sec.

C_α = 0.002



Load No.= 5
Load= 2.00 tsf
D_0 = 0.0662
D_{90} = 0.0705
D_{100} = 0.0709
T_{90} = 2.85 min.

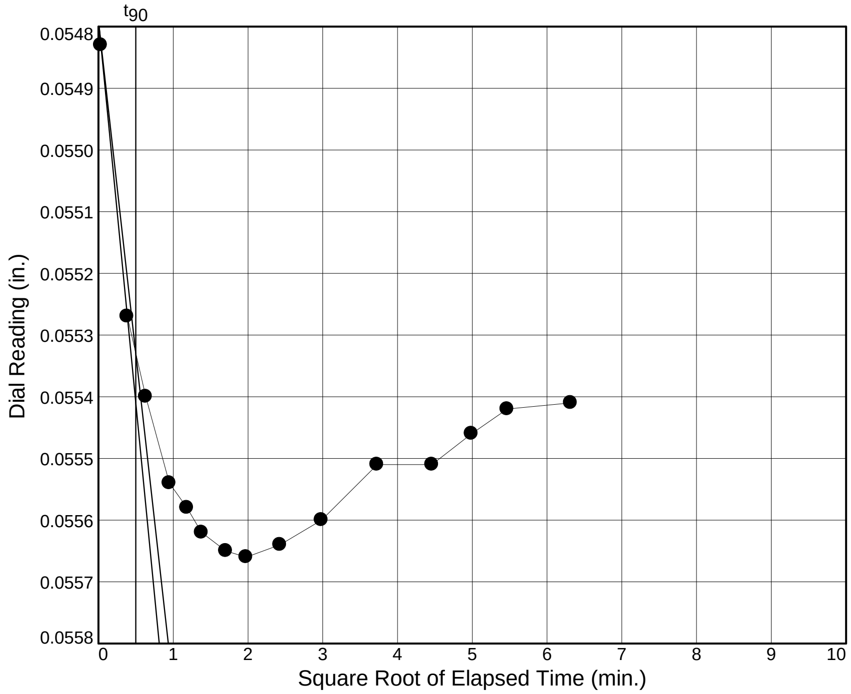
C_v @ T_{90}
0.0070 cm.^{2}/sec.

Figure C-736-4

Thielsch Engineering Inc.

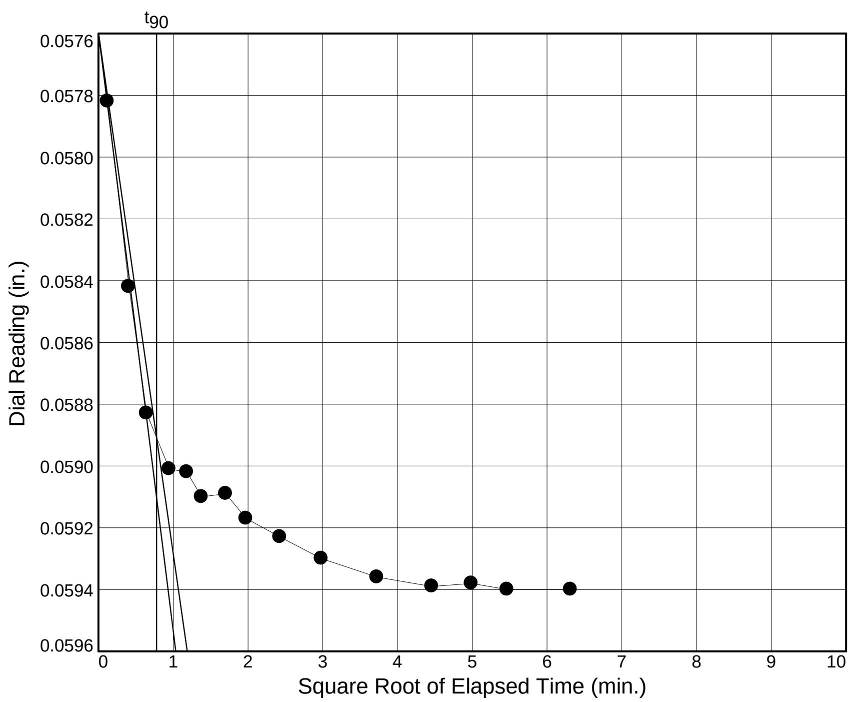
Dial Reading vs. Time

Project No.: 160630.01
Project: Ipswitch - Suppplemental Limitied Subsurface Investigation
10 Estes Street
Source of Sample: SGH-2018-2A Depth: 13.5-15.5' Sample Number: US-3



Load No.= 8
Load= 0.25 tsf
D_0 = 0.0548
D_{90} = 0.0553
D_{100} = 0.0554
T_{90} = 0.25 min.

C_v @ T_{90}
0.0825 cm.^{2}/sec.



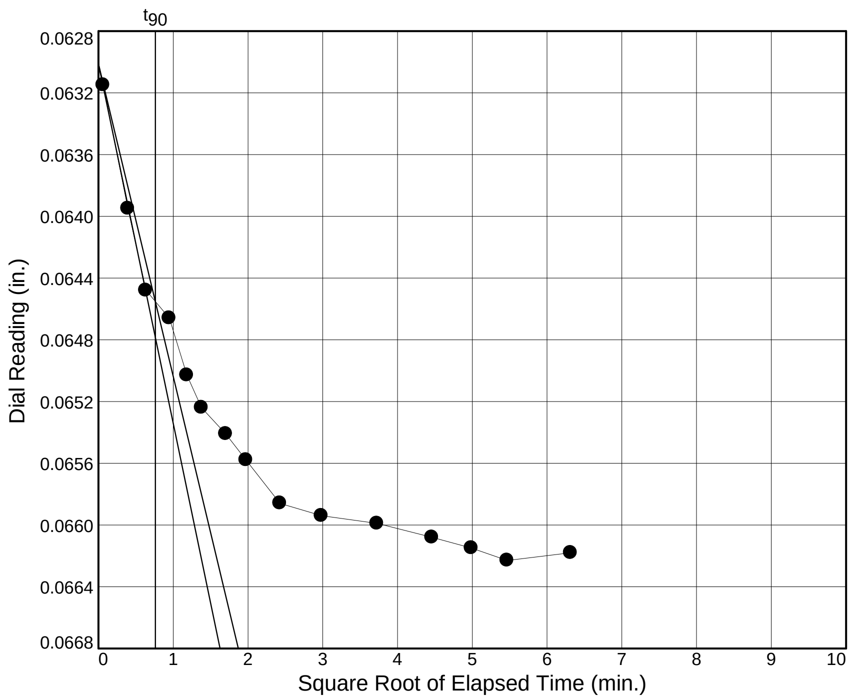
Load No.= 9
Load= 0.50 tsf
D_0 = 0.0576
D_{90} = 0.0589
D_{100} = 0.0591
T_{90} = 0.60 min.

C_v @ T_{90}
0.0337 cm.^{2}/sec.

Figure C-736-5

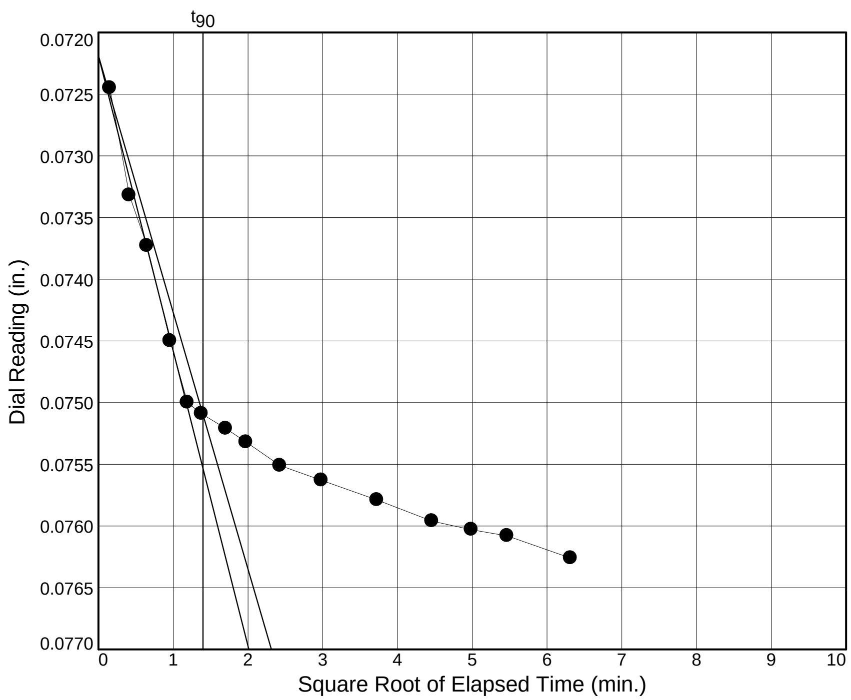
Dial Reading vs. Time

Project No.: 160630.01
Project: Ipswitch - Suppplemental Limitied Subsurface Investigation
10 Estes Street
Source of Sample: SGH-2018-2A   Depth: 13.5-15.5'   Sample Number: US-3



Load No.= 10
Load= 1.00 tsf
D_0 = 0.0630
D_{90} = 0.0646
D_{100} = 0.0647
T_{90} = 0.58 min.

C_v @ T_{90}
0.0348 cm.^{2}/sec.



Load No.= 11
Load= 2.00 tsf
D_0 = 0.0722
D_{90} = 0.0751
D_{100} = 0.0754
T_{90} = 1.95 min.

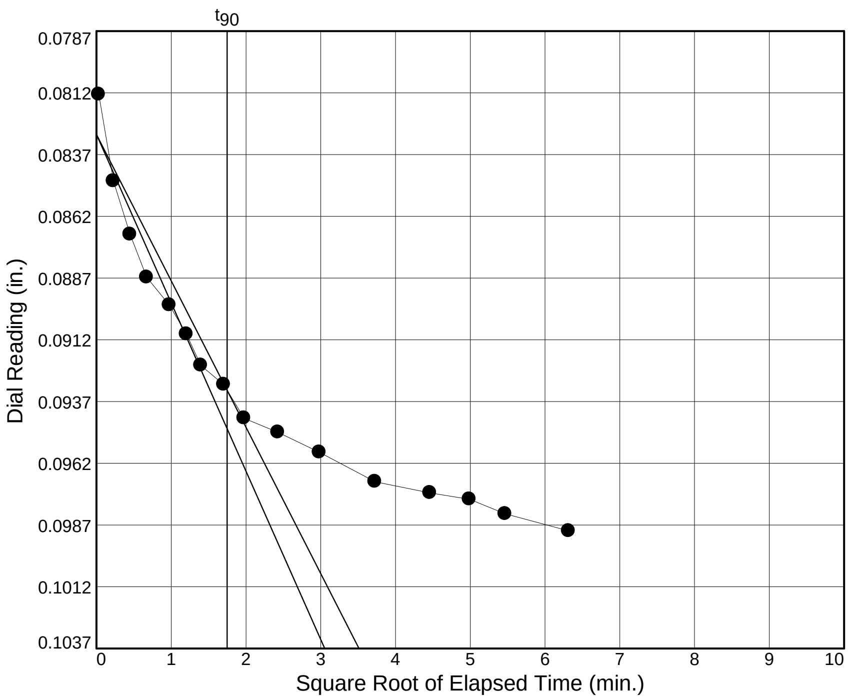
C_v @ T_{90}
0.0101 cm.^{2}/sec.

Figure C-736-6

Thielsch Engineering Inc.

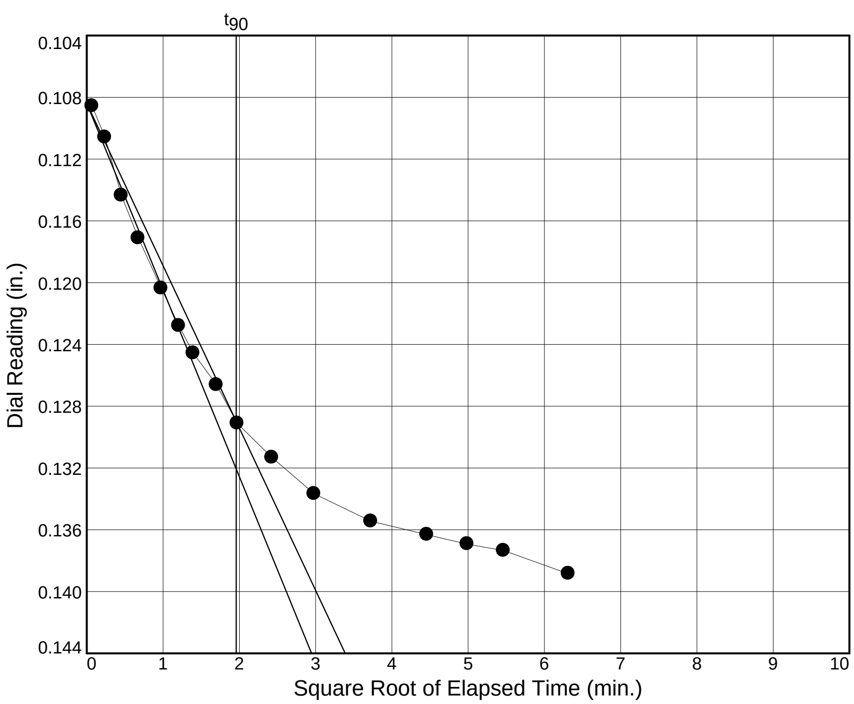
Dial Reading vs. Time

Project No.: 160630.01
Project: Ipswitch - Suppplemental Limitied Subsurface Investigation
10 Estes Street
Source of Sample: SGH-2018-2A Depth: 13.5-15.5' Sample Number: US-3



Load No.= 12
Load= 4.00 tsf
D_0 = 0.0829
D_{90} = 0.0932
D_{100} = 0.0944
T_{90} = 3.05 min.

C_v @ T_{90}
0.0063 cm.^{2}/sec.



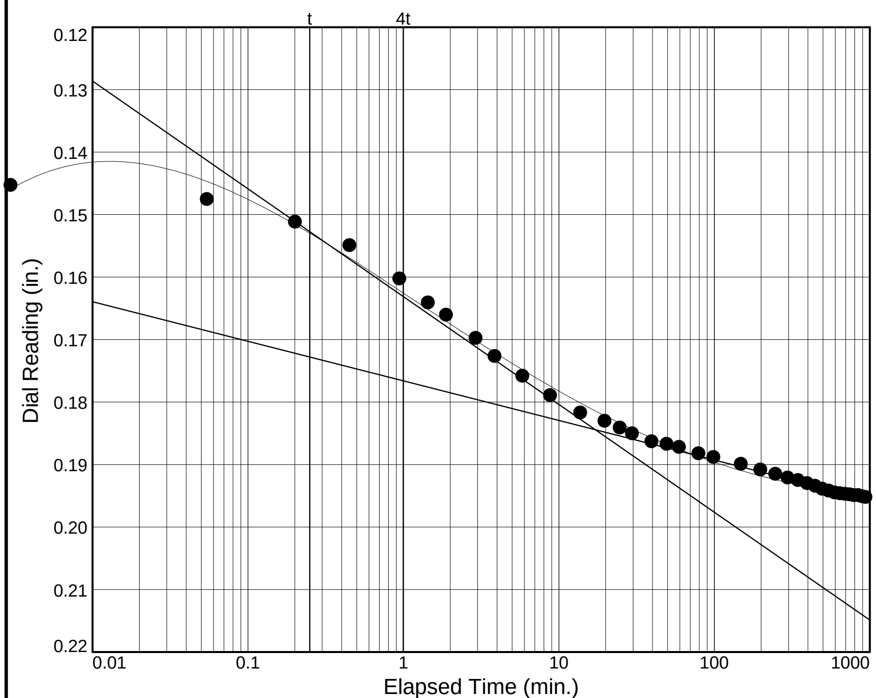
Load No.= 13
Load= 8.00 tsf
D_0 = 0.1084
D_{90} = 0.1290
D_{100} = 0.1313
T_{90} = 3.84 min.

C_v @ T_{90}
0.0046 cm.^{2}/sec.

Figure C-736-7

Dial Reading vs. Time

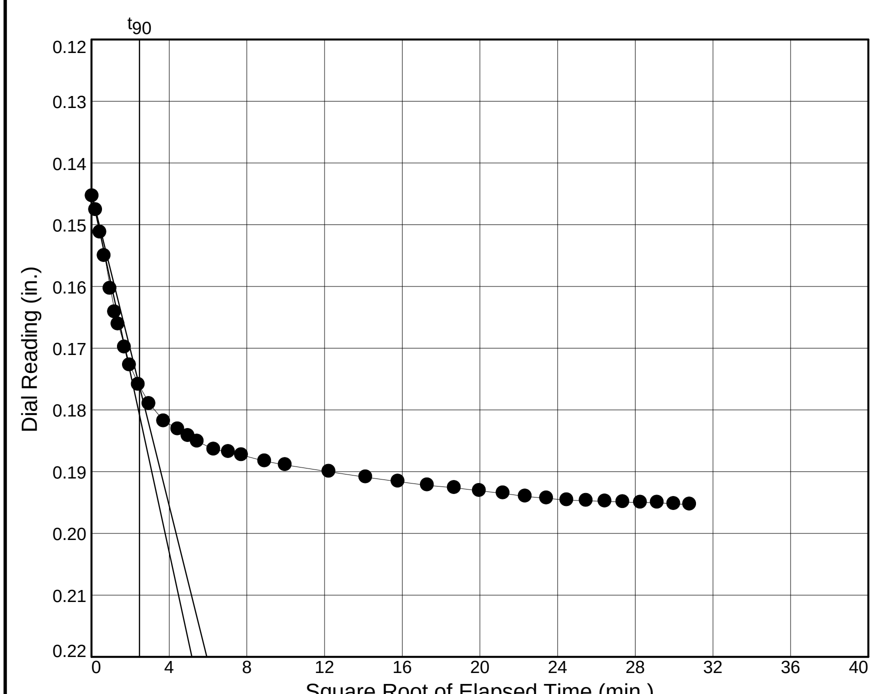
Project No.: 160630.01
Project: Ipswitch - Suppplemental Limitied Subsurface Investigation
10 Estes Street
Source of Sample: SGH-2018-2A Depth: 13.5-15.5' Sample Number: US-3



Load No.= 14
Load= 16.00 tsf
D_0 = 0.1434
D_{50} = 0.1639
D_{100} = 0.1844
T_{50} = 1.21 min.

C_v @ T_{50}
0.0031 cm.^{2}/sec.

C_α = 0.006



Load No.= 14
Load= 16.00 tsf
D_0 = 0.1449
D_{90} = 0.1761
D_{100} = 0.1796
T_{90} = 6.10 min.

C_v @ T_{90}
0.0026 cm.^{2}/sec.

Figure C-736-8

2021 SGH APPENDIX B

Excerpts from the 2020 HGI Report titled, "Final Report of Geophysical Investigation Ipswich Mills Dam Removal, Ipswich, MA," dated August 2020, prepared by Hager Geoscience, Inc. for Horsley Witten Group.

APPENDIX B

Select Pages from 2020 Geophysical Study

Cover
Table of Contents
Executive Summary
Section 6.0 Results
Plate II-F
Appendix III (EBSCO Building Pile Survey)

FINAL REPORT OF
GEOPHYSICAL INVESTIGATION
IPSWICH MILLS DAM REMOVAL
IPSWICH, MASSACHUSETTS

August 13th, 2020
File 2020021

Prepared for
Horsley Witten Group, Inc.
90 Route 6A
Sandwich, MA 02563

EDWOSB CERTIFICATION AWARD
U.S. Women's Chamber of Commerce
EDWOSB Federal
Certified Economically Disadvantaged Women-Owned Small Business

TABLE OF CONTENTS

APPENDICES

LIST OF PLATES

EXECUTIVE SUMMARY

During the period from April 27th to June 22nd, 2020, Hager GeoScience Inc. (HGI) conducted land and river geophysical investigations for Horsley Witten Group (HW) at the Ipswich Mills Dam (Dam) on the Ipswich River (River) and in the adjacent EBSCO building in Ipswich, Massachusetts. The overriding goal of the investigation was to characterize subsurface geologic and anthropogenic conditions upstream of the Dam and around the EBSCO building in support of ongoing dam removal studies. Objectives of the investigation were as follows:

I. Characterize the boulder content adjacent to the upstream side of the Dam.
II. Map sediment distribution and bedrock elevation on land and in the river, focusing on the near-shore and on-shore regions proximal to the Dam.
III. Confirm the presence and extent of soft sediment underlying the EBSCO building and determine if piles are supporting its foundation.
IV. To the extent possible, identify overall bedrock trends in upstream areas of the river between the Dam and railroad bridge.
V. Determine sediment thickness at the mouth of various tributaries of the Ipswich River upstream of the Dam.
VI. Assess the sediment thickness and bedrock elevation near the railroad bridge foundation.

In order to address the objectives, HGI's investigative plan was divided into four parts:

1. Interior EBSCO Building Survey and Exterior Wall Survey
2. Exterior EBSCO Building Land Survey
3. River Survey
4. Dam Survey

The geophysical methods used for this investigation were ground penetrating radar (GPR), seismic methods, and electrical resistivity tomography (ERT). Plate 1 presents a site overview map showing the survey coverage with each method used for all objectives.

The text of this report is organized so that general discussions of site conditions, *a priori* information, geophysical approaches and the equipment used are presented first in the main body of the report. Appendices I-VI of this report present the methodology and implementation strategies, survey results, and discussions pertaining to ***each objective***. They include plates and figures and can be read as stand-alone reports.

The essential findings of the survey are as follows:

I. **Boulders exist near the dam** in a layer that has a thickness ranging from 3 to 6 feet. The top of the boulder layer ranges in elevation from 5.5 to 6 feet and the elevation at its base ranges in elevation from 0 to 3 feet. GPR survey results indicate that the boulder layer overlies river sediments rather than sitting directly on bedrock.

II. **On-shore sediment stratigraphy** consists of several feet of fill material that overlies either soft sediments or denser till. While neither material is well-suited as structural fill, HW has expressed that the natural soils are of primary concern, especially beneath the EBSCO building. Soft soil is thickest in the southeastern portion of the area surveyed on land. A contour map of GPR- and borehole-identified soft sediment thickness is found in Appendix II. Top-of-till elevation is mapped using the results of GPR and HVSR surveys and borehole information provided by HW. Till elevations decrease to the southeast.

Near-shore sediment thickness generally increases with distance upstream from the Dam and is maximal near the east bank of the River south of Sally's Pond. Anomalously thick sediment deposits correlate with bedrock lows.

Bedrock elevation on land and water was mapped continuously by interpolation between River GPR datasets and land datasets comprised of GPR, seismic refraction, and HVSR survey results, as well as borehole data provided by HW. Borehole refusals were interpreted as bedrock, as the inclusion of borehole points did not create spurious deviations from overall bedrock trends. Rock dips eastward and reaches a minimum elevation of approximately -36 feet south of Sally's Pond (Plate IV-A). One well-defined linear east-west trending bedrock depression was observed near the connection between EBSCO Buildings 10 and 10A. Based on the complexity of mapped faults in the area and its glacial history, we assume that bedrock depressions may be related both to fracture distribution and paleo-channels.

III. Sediments beneath the building have the same low seismic shear wave velocity (Vs) characteristics as those immediately outside in the small courtyard south of the EBSCO building. Sub-slab soft sediments are likely loose naturally deposited river silts and clays rather than engineered substrate materials. This provides indirect evidence that the foundation of the EBSCO building rests on piles in this area.

Direct evidence of long structural elements is observed in GPR records collected both inside the building and along the vertical wall on the river, accessed by boat. These structural elements are assumed to be wooden piles based on both the characteristics of the GPR response, which are similar to those we have observed on sites where the nature of the piles was confirmed by excavation, and the typical foundation construction habits in use at the time the building was constructed. The vertical wall survey indicates that pile structures are embedded to at least a depth of approximately 0.5 feet below the top of the floor slab, at a top elevation of approximately 11.17 feet.

Along with GPR reflection amplitude, the pile elevation data indicate that the top of pile structures may regularly be above the water table under existing conditions and therefore subject to degradation due to aerobic bacteria.

IV. Beyond the maximum sediment thickness/minimum bedrock elevation zone discussed in the summary of Objective II findings above, the expansion of the GPR survey southward (upstream) of the Dam toward the railroad bridge did not reveal a consistent trend of the bedrock surface or sediment thickness. Low frequency GPR survey traverses collected by boat over what was presumed to be the thalweg of the River indicate bedrock elevations ranging from approximately -10 to -20 feet and sediment thicknesses of approximately 8 to 20 feet. As suggested by surface topographic trends, bedrock depressions may be related to fracture systems. Additional survey coverage is required to identify any potential east-west trends.

V. An informal survey to identify the depth of soft sediment at the Kimball Brook, Saltonstall Stream, Shady Brook, and Miles River confluences was conducted by probing the soft sediment to refusal using a 2-inch-diameter wooden rod. The elevations of the soft sediment and rod refusal were estimated based on measurements of the River surface elevation at the time of survey, bathymetry measurements made from GPR records collected near the probe points, and depths of rod refusal. These values are reported in the table provided in Appendix V.

VI. GPR transects collected by boat around the foundation of the railroad bridge were analyzed to produce contour maps of the sediment thickness and bedrock depth at this location. Bedrock elevation ranged from approximately -7 to -14 feet and sediment thickness ranged from approximately 11 to 21 feet, with the thinnest areas observed immediately adjacent to the bridge boulder-laden foundation.

Site-specific measurements of soil Vs made in MASW analysis may be used to derive the depth of impedance boundaries that generate observed spectral peaks in 3-component geophone records. In the absence of site-specific data, published regression equations can be used to produce depth estimates.

In our experience, the results of HVSR surveys are best used as an approximation to obtain information on the general spatial distribution of impedance boundaries rather than as absolute values. Where data do not agree with the findings of other, more reliable methods, the HVSR data points should be rejected.

6.0 RESULTS

Results are discussed in detail in the Appendices for each Objective. The essential findings of the survey, labeled according to the Objective to which they are most pertinent, are as follows:

I. Boulders exist near the dam in a layer with thickness ranging from 3 to 6 feet, and the top of the boulder layer ranges in elevation from 5.5 to 6 feet. GPR survey results indicate that the boulder layer overlies the river sediments at an elevation of 0 to 3 feet rather than sitting directly on bedrock.

II. On-shore sediment stratigraphy consists of fill material that overlies either soft sediments or denser till. The soft sediment thickness is represented in Plate II-E of Appendix II. The Appendix II discussion describes the characteristics of the soft sediment and till, respectively. Top-of-till elevation is mapped using the results of GPR and HVSR surveys and borehole information provided by HW. Till elevations decrease to the southeast.

    Near-shore sediment thickness generally increases with distance upstream from the Dam and is maximal near the east bank of the River south of Sally's Pond. Anomalously thick sediment deposits correlate with bedrock lows.

    Bedrock elevation was mapped continuously by interpolation between River GPR datasets and land datasets comprised of GPR, seismic refraction, and HVSR survey results, as well as borehole data provided by HW. Borehole refusals were interpreted as bedrock, as the inclusion of borehole points did not create spurious deviations from overall bedrock trends. Rock dips eastward and reaches a minimum elevation of approximately -36 feet south of Sally's Pond. One well-defined linear east-west trending bedrock depression was observed near the connection between EBSCO Buildings 10 and 10A. Based on the complexity of mapped faults in the area and its glacial history, we assume that bedrock depressions may be related both to fracture distribution and paleochannels.

III. Sediments beneath the building have the same low seismic shear wave velocity (Vs) characteristics as those immediately outside in the small courtyard south of the EBSCO building. Sub-slab soft sediments are likely uncompacted and naturally deposited river silts and clays rather than engineered substrate materials. This provides indirect evidence that the foundation of the EBSCO building rests on piles. Direct evidence of long structural elements is observed in GPR records collected both inside the building and along the vertical wall on the river, accessed by boat. These structural elements are assumed to be wooden piles based both on the characteristics of the GPR response, which are similar to those we have observed on sites where the nature of the piles was confirmed by excavation, and the typical foundation construction habits in use at the time when the building was constructed. The vertical wall survey indicates that pile structures are embedded to at least a depth of approximately 0.5 feet below the top of the floor slab, at a top elevation of approximately 11.17 feet. This indicates that the tops of pile structures may regularly be above the water table under existing conditions, and therefore subject to degradation.

IV. Beyond the maximum sediment thickness/minimum bedrock elevation zone discussed in the summary of Objective II findings above, the expansion of the GPR survey southward (upstream) of the Dam toward the railroad bridge did not reveal a consistent trend to the bedrock surface or sediment thickness. Low-frequency GPR survey traverses collected by boat over what was presumed to be the thalweg of the River indicate bedrock elevations ranging from approximately -10 to -20 feet and sediment thicknesses of approximately 8 to 20 feet. Low-lying bedrock zones may be related to fractures or paleochannel geometry, but additional survey coverage is required to identify any potential east-west trends.

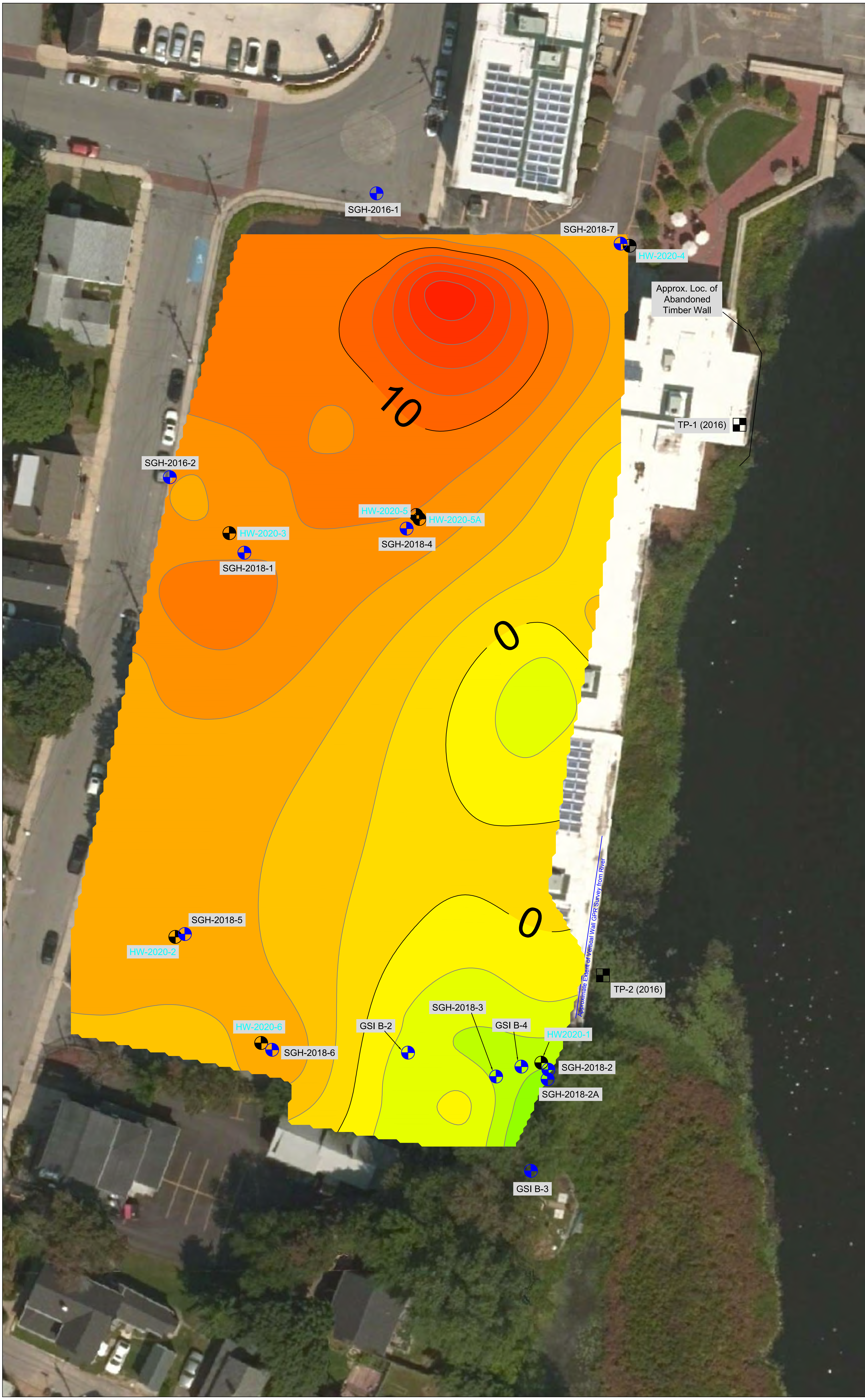
V. An informal ground-truthing survey to identify the depth of soft sediment at the Kimball Brook, Saltonstall Stream, Shady Brook, and Miles River confluences was conducted by probing the soft sediment to refusal using a 2-inch-diameter wooden rod. The elevations of the soft sediment and rod refusal were estimated based on measurements of the River surface elevation at the time of survey, bathymetry measurements made from GPR records collected near the probe points, and depths of rod refusal. These values are reported in the table provided in Appendix V.

VI. GPR transects collected by boat around the foundation of the railroad bridge were analyzed to produce contour maps of the sediment thickness and bedrock depth at this location. Bedrock elevation ranged from approximately -7 to -14 feet and sediment thickness ranged from approximately 11 to 21 feet, with the thinnest areas observed immediately adjacent to the bridge foundation.

7.0 METHODS AND LIMITATIONS

7.1 Ground Penetrating Radar (GPR)

7.1.1 Description of the Method. The principle of ground penetrating radar (GPR) is the same as that used by police radar, except that GPR transmits electromagnetic energy into the ground. The energy is reflected back to the surface from interfaces between materials with contrasting electrical (dielectric and conductivity) and physical properties. The greater the contrast between two materials in the subsurface, the stronger the reflection observed on the GPR record. The depth of GPR signal penetration depends on the properties of the subsurface materials and the frequency of the antenna used to collect radar data. Lower frequency antennas provide greater signal penetration, but result in lower object resolution.



Legend

Horsley Witten Borehole (Location Approximate)

SGH and GSI Boreholes (Location Approximate)

Test Pit (Location Approximate)

0 30 60 Feet

NOTES:
1.) The base map was created from HGI field notes, GPS data, an aerial photograph of the site provided by Microsoft© Bing, information provided in the file "190227_Final Report Ipswich Mills Dam Removal Feasibility_16041.pdf" provided by Horsley Witten, and a site plan of the inside of the building provided by EBSCO.
2.) The locations of indoor features, boreholes, and test pits presented by HGI are a **"best fit"** to the plans provided in the above-referenced file and should be considered approximate. Reference to HGI-located surface features is recommended.
3.) All elevations reference the NAVD88 vertical datum.

Elevation (ft)

20
10
0
-10
-20
-30
-40

PLATE II-F

August 2020 | File 2020021

Till Elevation: On Shore
Geophysical Investigation
Ispwich Mills Dam Removal
Ipswich, MA

Hager GeoScience, Inc.
596 Main Street, Woburn, MA 01801
(781) 935-8111 hgi@hagergeoscience.com

NOT ALL SUBSURFACE FEATURES MAY BE DEPICTED ON THIS MAP

APPENDIX III

EBSCO Building Pile Survey

1.0 INTRODUCTION

Results of previous geotechnical investigations by others suggested that soft sediment present at the south end of the EBSCO building may extend beneath the building and, therefore, also suggested that piles may have been used to support the building foundation. Concerns that the piles may be wooden and their integrity affected by lower river levels after dam removal provided motivation for investigating the building foundation structures.

HGI performed ground penetrating radar (GPR) and multichannel analysis of surface waves (MASW) surveys over accessible areas of the interior of the EBSCO building to determine whether the building is supported by piles and to confirm the presence of soft low-bearing sediment requiring pile support beneath the concrete. Investigation of onshore stratigraphy, including MASW analysis for soft sediment investigation, are reported in detail in Appendix II. Therefore, this Appendix III report will focus on the GPR pile detection effort.

Interior surveys were complemented with GPR surveys along portions of the exterior vertical foundation wall, using HGI's PortaBote for access. Table III-1 below provides the survey dates for the pile investigation.

Table III-1. Survey Objectives, Methods, and Dates

Method	Location	Survey Dates
GPR	EBSCO Office Floor Slab, Building 10-A	4-27-2020
GPR	Building 10-A Exterior Foundation Wall	6-4-2020
MASW	EBSCO Office, Building 10-A	4-27-2020

2.0 TECHNICAL APPROACH

2.1 Indoor Survey

Because the electromagnetic properties of wood and concrete are dissimilar, HGI opted to use GPR as the primary tool of investigation indoors to determine whether piles are present beneath the 3 to 4-foot-thick concrete floor slab of Building 10-A. Survey design was informed by HGI's previous experiences with detecting wood piles in similar settings, and focused on data collection along tightly spaced traverses in open areas of the office space. Furniture rearrangement was kept to a minimum as per EBSCO's requirements. Three GPR survey grids were established, the locations of which are shown on Plate III-A.

HGI elected to collect MASW data to complement the indoor GPR survey. The MASW technique images soil shear wave velocity (Vs), which is directly related to soil stiffness. Determining Vs values for sub-slab sediments can help assess the relative need for piles and, if soft sediments are identified, a minimum pile length can be inferred. The locations of the MASW surveys are also shown on Plate III-A.

2.2 Outdoor Survey

The exterior foundation wall at the south end of the building was accessible by boat. HGI collected GPR data along north-south trending transects above the water line over the foundation wall to detect whether vertical foundation elements existed behind its face. The location of the outer wall survey is shown on Plate III-A.

2.3 Positional Information

HGI personnel used fiberglass measuring tapes and spray paint to lay out the indoor survey grids. As noted, the locations of HGI's traverses and grids are shown on Plate III-A, an AutoCAD® Map 3D 2021 plot created from the HGI field measurement survey notes and the EBSCO-provided floor plan "Lower Riverside Building Floor Plan.pdf.". The grid locations were referenced to surface features in the room (i.e., walls, support columns). The outer wall survey traverse locations were noted in terms of elevation, measured from the water surface elevation at the time of survey, and distances north or south of window edges.

Note that, while the EBSCO drawing is to scale in the areas surveyed, the overall building shape portrayed on the map does not match the extents of the building as shown in aerial photos and other drawings of the site. Note also that the EBSCO map image resolution was low and wall features are fuzzy when the drawing is enlarged to a suitable size for presentation in AutoCAD maps. For this reason, placement of the vertical wall survey extent, which was measured relative to the windows, is approximate. If a to-scale, high-resolution version of the EBSCO interior site plan is developed in the future, it can be incorporated into the HGI project maps.

3.0 DATA ACQUISITION

3.1 GPR Survey-Indoor

HGI personnel used a GSSI SIR-4000 digital GPR acquisition system with 900-MHz and 400-MHz antennas to collect GPR data indoors. The 900-MHz antenna was used to collect data along specific test lines to determine the average EM velocity of the concrete, while the 400-MHz antenna was used over all survey areas to create the main dataset for locating piles. 400-MHz data were collected along either orthogonal traverses (Grid 1) or east-west traverses (Grid 2) spaced no more than 0.167 feet apart. For both antennas, a survey wheel encoder provided horizontal distance control. Signal penetration with the 400-MHz antenna reached an approximate depth of 10 to 15 feet.

3.2 GPR Vertical Wall Survey-Outer Wall

Data were collected along the vertical wall from the river using the SIR-4000 acquisition unit and a 350-MHz antenna. Traverses were limited in length and vertical coverage by the vegetation along the wall and the water level, respectively. Data were collected along horizontal traverses at 0.5-foot vertical intervals over elevations ranging from 9.67 feet (0.5 feet above the water level at the time of survey) to 11.17 feet. All transects produced images below the floor slab elevation, which was measured as 11.65 feet. Data were collected in continuous (time) mode at a rate that allowed for an average of approximately 80 scans per foot. Vertical wall signal penetration reached a distance of approximately 8 to 10 feet behind the face of the wall (into the building).

Table III-2 below shows the pertinent parameters used for indoor and outdoor GPR data collection.

Table III-2. GPR Survey Acquisition Parameters

Antenna Frequency (MHz)	Range (ns)	Survey Mode	Scan Rate (per sec)	Scan Rate (per ft)	Sample Rate (per scan)	Effective Signal Depth (ft)
900	40	Wheel	100	60	1024	1-4
400	135	Wheel	100	36	1024	10-12
350	90	Time	100	73-100	512	6-10

4.0 DATA REDUCTION AND ANALYSIS

Data from the GPR survey were downloaded to a PC at the HGI office for processing and analysis using GSSI's RADAN® 7 software. Although overall signal quality was good, significant signal processing was necessary to mitigate detrimental effects from dense concrete reinforcement and signal attenuation due to concrete thickness and conductive soils beneath the concrete. Band-pass filtering, background removal, migration, and horizontal trace stacking were typically applied to enhance GPR records for analysis.

4.1 GPR Survey-Indoor

Processed 400-MHz records were used to construct 3D models of the surveyed areas, while 900- and 350-MHz records were examined as 2D sections. 3D models are useful for viewing the spatial qualities of the data and identifying subtle spatial features that may not be apparent in individual 2D records. The 3D models are sliced horizontally and vertically to observe patterns of GPR anomalies present in the radar data.

Each 400-MHz 2D record was also individually evaluated for possible anomalies. Preliminary interpretations based on analysis of the individual 2D records were plotted and evaluated in a spatial context using the 3D models. Conversely, spatial anomalies observed in the 3D models were re-examined on the individual records to ensure that all possible anomalies were evaluated. Interpreted individual GPR targets were then exported to AutoCAD, where the extents of anomalies and anomalous areas and linear trends were mapped.

4.2 GPR Vertical Wall Survey-Outer Wall

350-MHz records were examined in sequence from lowest to highest elevation. The positions of interpreted piles in terms of distance behind the wall were noted and mapped using AutoCAD. With the exception of lowest-elevation traverse results, only anomalies that persisted across multiple traverses were interpreted as potential piles.

5.0 RESULTS

The objective of the indoor GPR surveys was to establish whether soft sediment extended beneath the southeast end of the building and, if so, whether piles are present within the soft soil beneath the concrete slab. The results of indoor floor and outer vertical wall pile investigations show that piles are part of the building foundation structures and that soft soils are present below the concrete floor slab throughout the surveyed area.

A detailed accounting and cataloguing of individual piles requires significantly more analysis than that indicated by the original objective. However, the locations of piles manifesting strong signal response in the indoor floor and outer vertical wall surveys are presented in Plates III-B and III-C, respectively. Due to the thick and heavily reinforced concrete slab, not all of the piles could be imaged. Despite this, enough piles were imaged to suggest a pattern of pile distribution.

Figures IIIA(a) and IIIA(b) show the appearance of the same piles in a GPR record collected in indoor Grid 2 (traverse location shown in cyan on Plate III-A for reference) and a vertical wall traverse (approximate image extents boxed in cyan on Plate III-A), respectively.

Significant findings from the indoor survey include:

- The alignment of pile columns within the area surveyed appears to be north-south.
- Along the pile column, the piles appear as clusters or as individual piles.
- The distance between pile columns ranges from approximately 2.5 to 4 feet.
- The amplitude of reflections from the top of piles is variable, suggesting different pile conditions
- In general, the depths to the apparent top of piles are variable, ranging from 2.5 to 4.5 feet below ground level (elevation ranging from 7.15 to 9.15 feet).
- Although difficult to assess, the vertical pile lengths may extend between 12 to 15 feet below ground surface.
- The pile column closest to the exterior wall in Grid 3 is located approximately 4.5 feet from the outer wall.

Vertical wall findings are as follows:

- Depth of penetration in the vertical wall survey was influenced by the presence or absence of vertical rebar in the wall, which was most common in the southernmost 50 feet of each traverse and was sporadically observed in northern sections, and wall moisture content. Moisture content increased from south to north, correlating with the erosion of the concrete matrix noted along the wall.
- Elevation of the top of observable pile structures ranges from 9.67 feet to above 11.17 feet as shown in Plate III-C. The top of pile structures may include structural components on top of the pile and may not, therefore, represent the top of the pile. This was deduced by noting where the vertical features no longer appear in records collected at increasing elevation. The termination point of pile structures observed in the topmost

traverse (elevation 11.17 feet) cannot be determined as they appear to continue upward beyond the survey extents.

- Pile shape is difficult to ascertain, but the piles appear to be on the order of approximately 1 foot wide.
- Piles are clustered together in groups at the southern end of the building. North of this cluster, pile separation appears to be approximately 5 to 6 feet where it is regular.
- The distance to the piles from the outer wall ranges from 3.5 to 6 feet, which is consistent with the easternmost pile column in Grid 2.
- GPR reflection amplitudes for piles was generally high throughout the vertical wall survey area. Anomalously bright reflections for wood piles commonly occur in cases where wood is degraded by aerobic bacterial activity resulting from exposure to air.

6.0 DISCUSSION

The following discussion presents observations and interpretations intended to guide possible future activities. These include:

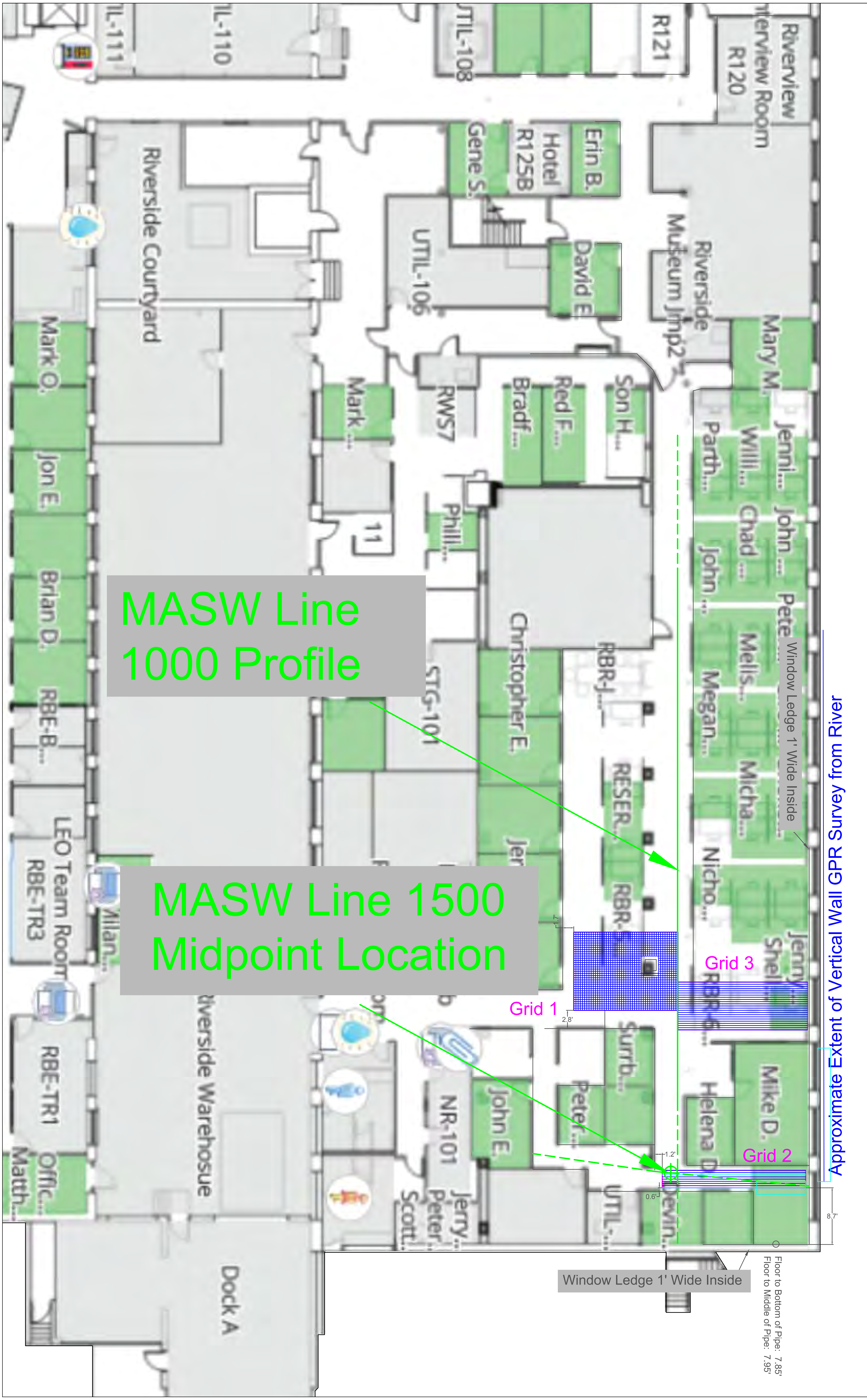
- The pile pattern observed in the indoor and vertical wall survey areas may not extend to the rest of the building areas.
- All vertical wall records were collected above the water, so these targets are already partially exposed to aerobic conditions that occur as a result of river level changes.
- Reflection amplitude is generally an indication of pile condition. Bright spots (high-amplitude reflections) at the tops of piles, as observed in vertical wall records and in certain parts of interior grid records, may indicate unstable conditions due to air gaps or conductive aerobic bacteria, suggesting rot.

GPR is an indirect method. We interpret the vertical features as piles because of their shape, consistency from record to record, position within the slab, and similarity in response to that at other sites. GPR imaging has confirmed piles using the same technique and interpretation criteria. However, we cannot conclusively determine the nature of the material used, the shape of the pile, or its integrity from GPR alone. Note that other geophysical techniques can be used to assess these characteristics if pile tops can be exposed, or if boreholes can be constructed in the river immediately adjacent to targets identified in the vertical wall survey.

Vertical wall records were collected as a field experiment. We noted that a portion of the wall was accessible when we were on the river conducting bedrock mapping with ERT and decided to take advantage of the run time that occurs during ERT data acquisition to image the vertical wall. The results of the survey are better than expected. Data quality is significantly higher than it was for the indoor survey, where the thick rebar mat inhibited easy identification of pile tops.

If a more concerted effort to map piles from vertical wall imaging is desired, we recommend repeating the survey with tighter distance control via survey wheel, establishment of a grid using chalk snap lines or other marking devices, and measurement of locations for multiple building features (e.g., southern edge of building, window ledges and intervening brick columns). If the water level is naturally lower at the time of a new survey, or if the water level can be lowered by dam control, additional traverses at lower elevations would image lower portions of the piles.

Repeating the survey farther to the north along the building wall can confirm the absence or presence of piles where the compressible soils layer is thinner, as revealed by the MASW survey.



Legend

- 400-MHz GPR Traverse
- Report Figure 400-MHz Traverse
- Report Figure Coverage
- MASW Geophone Spread
- MASW 2D Profile
- MASW 1D Profile Point

(Approximate)

0 10 20 Feet

NOTES:
1.) The base map was created from HGI field notes and a site plan of the inside of the building provided by EBSCO.
2.) The locations of indoor features presented by HGI are a **"best fit"** to the EBSCO plan and should be considered approximate. Reference to HGI-located surface features is recommended.

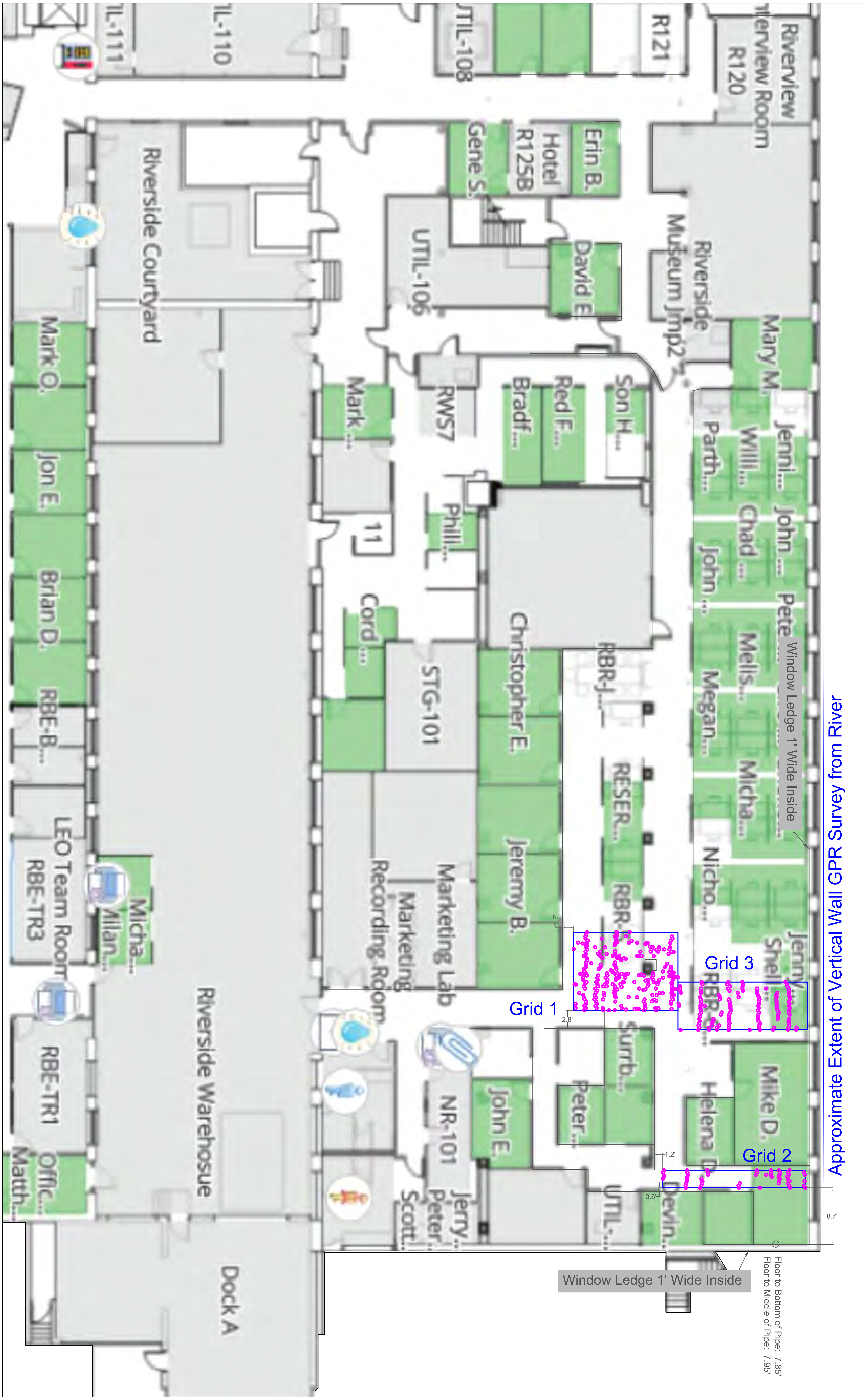
PLATE III-A

August 2020	File 2020021

Pile Survey Locations
Geophysical Investigation
Ipswich Mills Dam Removal
Ipswich, MA

Hager GeoScience, Inc.
596 Main Street, Woburn, MA 01801
(781) 935-8111 hgi@hagergeoscience.com

NOT ALL SUBSURFACE FEATURES MAY BE DEPICTED ON THIS MAP



Legend

400-MHz GPR Survey Extent

Location of Pile and/or Foundation Structural Component

(Approximate)

0 10 20 Feet

NOTES:
1.) The base map was created from HGI field notes and a site plan of the inside of the building provided by EBSCO.
2.) The locations of indoor features presented by HGI are a **"best fit"** to the EBSCO plan and should be considered approximate. Reference to HGI-located surface features is recommended.

PLATE III-B

August 2020 | File 2020021

Pile Locations
Geophysical Investigation
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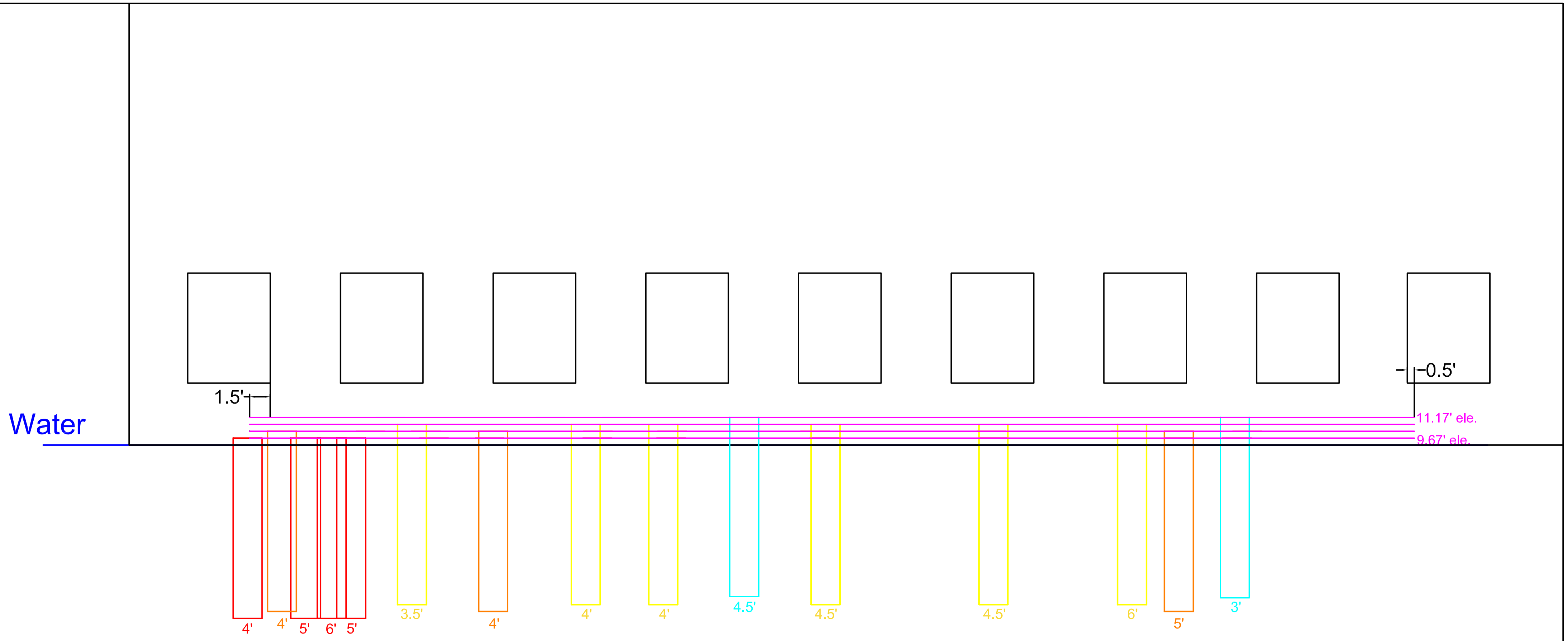

Legend

- 350-MHz GPR Traverse
- Pile Top Ele. 9.67'
- Pile Top Ele. 10.17'
- Pile Top Ele. 10.67'
- Pile Top Ele. ≥ 11.17'

(Approximate)

0 10 20 Feet

NOTES:
1.) The schematic drawing was created from HGI field notes, grid marks, and GPR interpretations.
2.) Building corner and window locations are approximate and should be verified with measurements in the field.
3.) The locations of structural elements are based on distance conversions of time-mode GPR data and are approximate.
4.) Numbers below columns indicate the approximate distance of that feature behind the wall.
5.) **Pile lengths are not drawn to scale.**
6.) **Shape of pile may not reflect true pile geometry.**
6.) Water level elevation at time of survey was 9.61 feet.
7.) EBSCO office building floor slab elevation is 11.65 feet. FOundation wall thickness is approximately 2.5 feet.

PLATE III-C

August 2020	File 2020021

Schematic Sketch of Pile Locations
Outer Foundation Wall GPR Survey
EBSCO Building East Wall
Ipswich River
Ipswich, MA

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NOT ALL SUBSURFACE FEATURES MAY BE DEPICTED ON THIS MAP

Figure III-A
Indoor GPR Survey Profiles

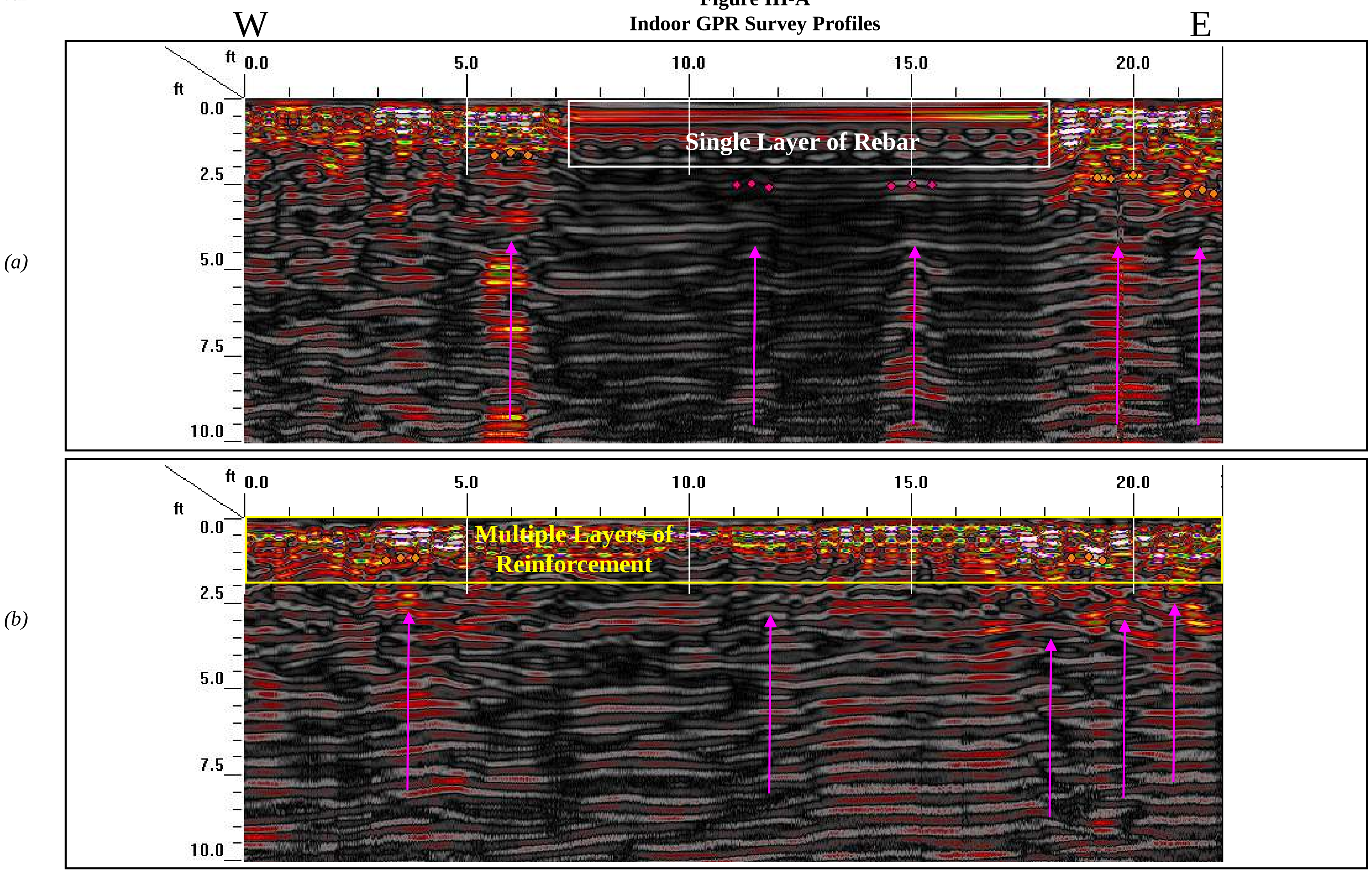
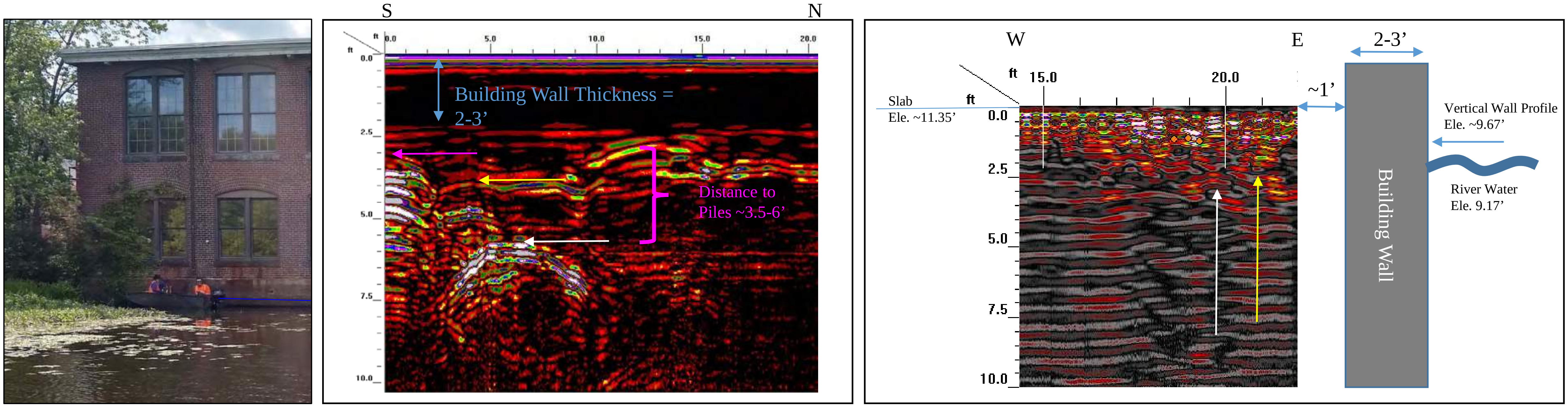


Figure III-A. Piles identified in records from GPR Grid 2, collected in the southeastern section of the EBSCO building from west to east (toward windows). Concrete floor slab reinforcement varied throughout the surveyed areas. Identification of potential piles was straight-forward in *(a)* areas where the slab reinforcement consisted of a single layer of rebar (white box). In areas with multiple layers of slab reinforcement, an example of which is highlighted with a yellow box in *(b)*, pile identification required additional effort with data processing and interpretation. Pink arrows indicate the "ringing" signature of potential piles, while dots indicate the interpreted tops of piles where clear.

Figure III-B
Vertical Wall GPR Survey Profile



(a) *(b)* *(c)*

Figure III-B. Additional GPR data were collected along the exterior of the EBSCO building by boat, with approximate traverse location shown as blue line *(a)*. Traverse collected from south to north along the boat-accessible portion of the building at an elevation of 9.67 feet shows evidence of wood piles, indicated with pink, yellow, and white arrows in *(b)*. Note that distance behind the vertical face of the wall is expressed along the y-axis of *(b)* and depths expressed along the y-axis of *(c)* are converted from two-way travel times and are dependent on the water content in the concrete, which varied within the survey area. As such, these distance and depth estimates are approximate. Features indicated with yellow and white arrows in *(b)* are also identified in *(c)*, a reproduction of a portion of Figure G-1(b) showing the lower-confidence interpreted pile locations.

2021 SGH APPENDIX C

2021 RSI Report: letter from RSI to SGH titled, "GPR, Impact Echo, and Sonic Echo/Impulse Response Surveys for Structural Assessment of the Slab and Foundation Walls EBSCO Property, Ipswich, MA," dated 28 June 2021.

Radar Solutions International, Inc.™

Geophysics for People and the Environment

June 30th, 2021

Ms.Giuliana A. Zelada-Tumialan, P.E., Sr. P.M.
Mr. Steve Keppel, P.E., Sr. Consulting Engineer
Simpson Gumpertz and Heger
41 Seyon Street, Building 1, Suite 500
Waltham, MA 02453-3819

Via Email: gazelada@sgh.com , SFKeppel@sgh.com

Re: GPR, Impact Echo, and Sonic Echo/Impulse Response Surveys
For Structural Assessment of the Slab and Foundation Walls
EBSCO Property, Ipswich, Massachusetts

Dear Giuliana and Stephen:

Per your authorization, Radar Solutions International, Inc. (RSI), a WBE/DBE Certified firm based in Waltham, Massachusetts, performed Ground Penetrating Radar (GPR), impact echo (IE), and Sonic Echo/Impulse Response (SEIR) investigations at the EBSCO Property in Ipswich, MA on June 7th and 10th, 2021. The goals of this investigation were to: 1) characterize the slab within the area of investigation, 2) determine whether pier caps are present beneath the column, and the depth of footing and/or pile beneath the footing, 3) determine whether grade beams are present beneath the slab along the column lines, and whether they are supported by wooden timber piles, 4) locate potential voids beneath the concrete, and outside the building 4) determine the foundation wall depth and thickness a the top of the south and east foundation walls. RSI's Associate Geophysicists, Cameron Russ and Mackenzie Kilpatrick, and Geophysical Technician Richard Lammey, were on site both days. Following summarizes the results from our June 2021 surveys. Thank you again for your business.

BACKGROUND

The EBSCO Facility is located at 10 Estes Street, Ipswich, MA. The assessment of the structural integrity of the building has been an on-going process for several years as part of a larger impact assessment the Town of Ipswich is conducting should the dam downstream be removed. In Spring and Summer of 2020, Hager Geoscience, Inc. conducted its own geophysical investigation for Horsley Witten Group, which answered some questions about soil and bedrock conditions outside of the building, but raised additional questions. At the behest of SGH, RSI conducted its own Geophysical surveys to help answer some of the questions raised in the previous survey.

METHODS

RSI used three different geophysical methods, Ground Penetrating Radar (GPR), Impact Echo (IE), and Sonic Echo/Impulse Response to help evaluate the structural integrity of the building's slab and foundation. The GPR was used to help determine slab thickness, whether there are areas of thick concrete present, corresponding to grade beams and/or footings, to help determine the presence, geometry, and depth to the top of potential wooden timber piles, and to help locate potential voids beneath the slab. Initially, GPR was going to be used to help confirm the thickness of the south and east foundation walls, but unfortunately, without a test pit along the south wall, and lower the water level along the east wall, these measurements could not be done.

Both IE and SEIR are ultrasonic methods, where acoustic waves are utilized instead of EM waves. IE measurements were done to help confirm the thickness of the slab, to determine the thicknesses of any grade beams, if present, and to determine whether there are footings present, and if so how deep they extend beneath the columns. The IE was also used to assess the thickness of the top portion of the south and east foundation walls.

Sonic Echo/Impulse Response was used to determine whether there is any wooden timber pile in structural contact with the slab, and if so, how deep it extends below the top of slab into the ground. These three methods were used together in efforts to provide a more detailed assessment of the slab, its support, and the foundation walls. Below describes each method in detail.

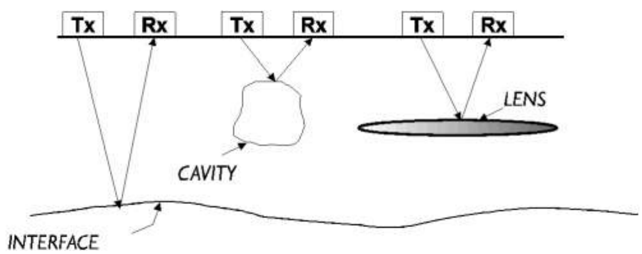
Ground Penetrating Radar (GPR)

The GPR method creates a cross-section of reflections as a function of horizontal distance versus approximate depth everywhere the antenna is moved. For the June 2021 survey, RSI used two GPR antennas, a 1.5 GHz (1500 MHz) antenna, which provides high-resolution of the upper 20 inches or so from top of slab, and a 500 MHz antenna, which provides sub-slab information up to about 8 feet below grade.

The GPR method operates by transmitting low-powered microwave energy into the ground using an ultra-wide band (UWB) transceiver antenna. The peak power of any GPR antenna is 20 to 100 times less the wattage of a cellular phone, and the energy is directed into the ground (and not at the operator) by means of shielding on the top side of the antenna. The GPR signal is then reflected back to the antenna by materials with contrasting electrical impedance, which is primarily determined by dielectric and conductivity properties of the material, its magnetic permeability, and its physical properties. The greater the contrast in the real dielectric permittivity (RDP) of two materials, the greater the reflection amplitude. Typically, high-amplitude reflections occur at lithologic or mineralogic changes, or where there is a sudden change in water content.

A material's dielectric properties are primarily determined by mineralogy, and water content. A soil with a high iron and/or magnesium content, or one that contains mineralogical clay or other platey minerals, will have a higher RPD value than a quartz-rich sand. Similarly, a soil that has a high porosity and is water saturated will have a higher RDP for the same unsaturated soil.

Reflections observed on GPR records can be non-unique, meaning that a similar reflector can be caused by different objects. Strong reflections are typically produced from metal objects, which has an RDP of 1,000, the water-table, and clay layers. The schematic below shows the different ways reflections can occur. Objects, such as utilities, that have a discrete length and width typically produce hyperbolic reflections on GPR records.

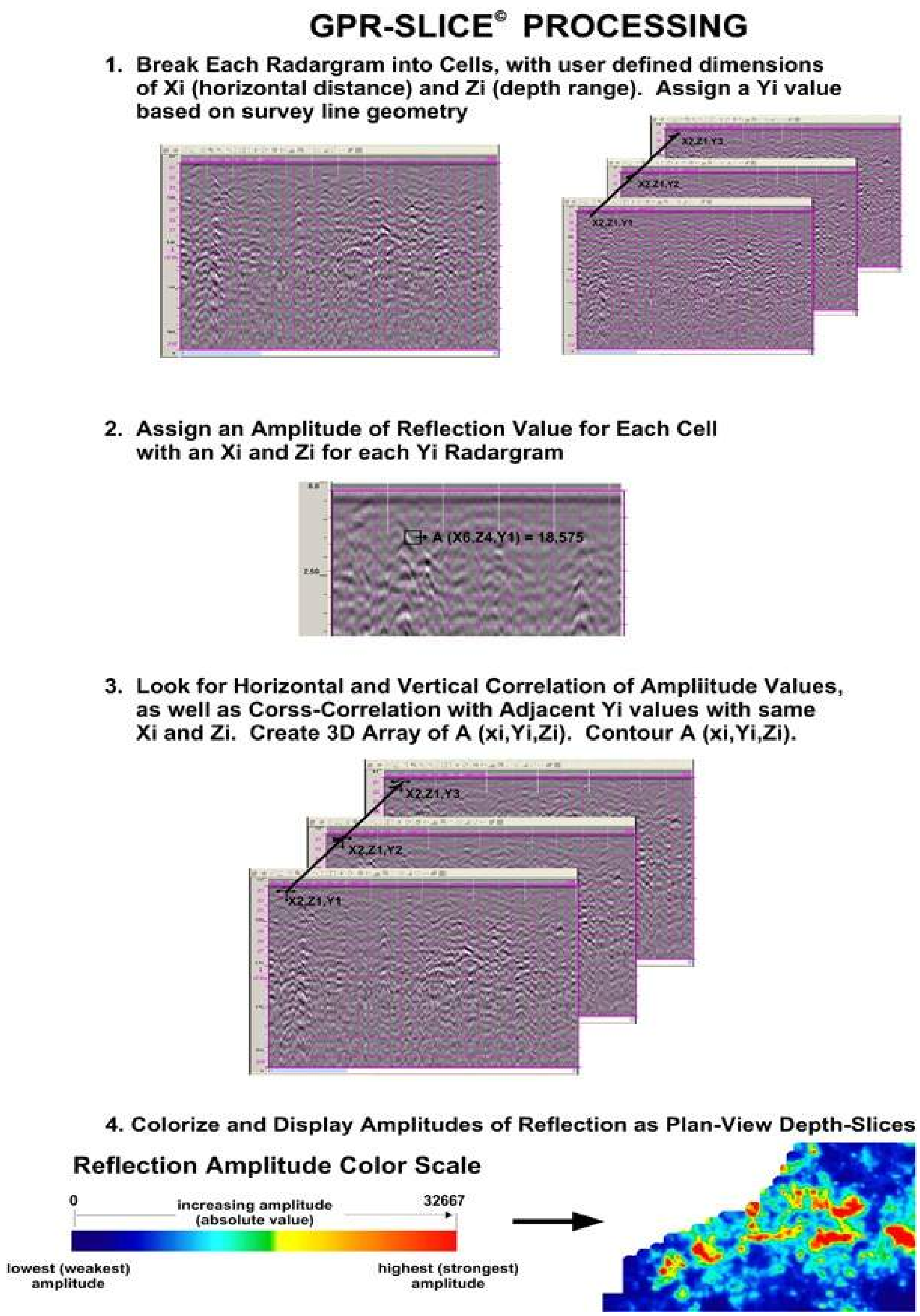


LEFT: Schematic showing that GPR reflections occur where there is a change in dielectric properties, such as at an interface of two materials or at an object.

In the absence of water, physical changes in densities, or a relatively large or irregularly-shaped inclusion in the soil, can cause weak radar reflections. For instance, boulders are detectable within the surrounding soil, not only because of subtle lithologic contrasts, but because their irregular shape causes reflection and diffraction of the GPR waves. A wooden timber pile, while not electrically not much different from fill material, would produce a similar reflection/diffraction pattern on the radargram.

The success of the GPR methodology also depends on the amount of EM signal attenuation experienced at any given site. GPR signal attenuation is caused by four loss mechanisms: conductive losses, molecular relaxation losses, "clay" (or interfacial polarization) losses, and scattering losses (Kutrubes, 1986). By far, the greatest source of loss is caused by conduction losses, which are most severe at frequencies of 300 MHz and below. By mapping areas of attenuation, GPR can be used to identify such subsurface conditions as thick concrete and groundwater plumes.

GPR data for this survey were visually inspected using proprietary software programs: RADAN© (developed by GSSI), and EKKO PROJECT© (developed by Sensors and Software). GPR data are also processed and imaged as 2D and 3D time-depth slices using GPR-Slice©, software developed by Dr. Dean Goodman of the Geoarcheometry Laboratory <www.gpr-survey.com>. This state-of-the-art 2D and 3D GPR imaging software quantifies GPR results by digitizing the amplitude of reflection from each GPR record, looks for horizontal correlation of features across adjacent and nearby parallel lines, assembles them into a 3D image, then contours the data at each time/depth interval specified by the user. Below is a schematic showing how this imaging process works.



The greater density of GPR lines helps RSI to generate depth-slice images of the structural steel, potential footings and grade beams, as well as help identify voids and areas of loose material, if present. Concrete naturally attenuates, or “absorbs” GPR signal. So, areas of thick concrete, such as where footings or grade beams are present, appear as dark-blue filled areas on the GPR depth-slices. Also, dark blue-filled contours correspond to areas of high GPR signal loss occur where concrete is deteriorated (i.e. “wet” concrete), or where there is conductive soil and/or water in the fill, on the depth-slices.

GPR reflections are non-unique, as conduits, post-tension cables, mesh, and rebar appear similarly on 1500 MHz radargrams, while utilities, boulders, and voids and/or loosely packed soil appear similar on 500 MHz radargrams. This is due to the broad, four-lobed radiation pattern of the GPR, which records data up to five feet fore and aft of the 500 MHz antenna housing, and about 1.5 to 2 feet side to side. For this reason, we use the GPR depth slice images to anticipate the location, trend, and depth of GPR reflections.

The depth of investigation and the resolution of the GPR is site-specific. The less cured the concrete, or the more deteriorated the concrete or conductive/wet the underlying fill material, the less penetration and resolution there will be. (Typically, the 1500 MHz antenna achieves a 16 to 22 inch penetration depth, which is reduced to a few inches in wet concrete.)

Also, it should also be noted that GPR image resolution beneath mesh and/or rebar can be limited as metal is a perfect reflector. The greater the amount of metal reinforcing located above potential conduits, the less penetration there will be. For instance, the GPR method can not detect conduits beneath metal pan or decking as all the energy is reflected upwards and none gets through the metal pan. Therefore, locating conduits beneath the upper rebar schedule or mesh is a function of the density of the reinforcing as well as the degree of curing in the concrete.

Impact Echo

Impact-Echo (IE) is a standardized acoustic wave technique for measuring the thickness of concrete pavement, slabs, footings, foundations, and piers (ASTM C1383 and D5882-00). RSI uses an Olson Engineering NDE 360 ultrasonic instrument. When used in the impact echo configuration, a solenoid impactor (or hand-held hammer), strikes the surface of bare concrete. A pressure or compressional wave (p-wave) is generated by the source, which travels down through a slab. When this compressional wave encounters the underlying fill material, which is significantly less dense than the reinforced concrete (i.e. a lower acoustic impedance), it is reflected back from the bottom of the slab. The transmitted and reflected acoustic energy is recorded within the solinoid, where the waveform is evaluated for reflections in the time-domain, i.e. by looking for reflections within the waveform.

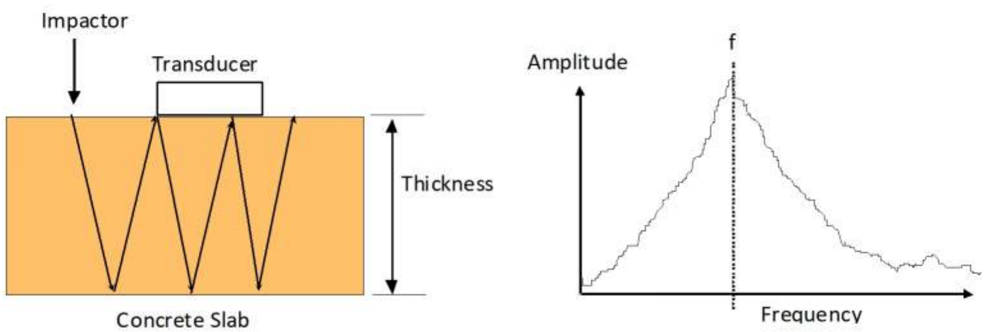
The depth to the bottom of the foundation is determined by the simple relationship:

$$\text{Thickness (ft.)} = V_p{*}(t/2) \qquad (1)$$

where V_p is the p-wave velocity of 12,000 ft/s is used for good concrete, and “t” is the two-way travel time, recorded at the NDE 360, in micro seconds (uS).

As shown in the schematic below, the acoustic wave travels from the surface into the concrete, and is reflected back from the bottom of concrete. As air also has a lower impedance than concrete, some reflected signal bounces back through the concrete, where it is again reflected back to the surface at the bottom of the concrete, producing the repetitive reflection pattern shown below. Rather than measure the travel time directly, it has been shown that measurement of the frequency spectrum of the reflected signal is much more effective and accurate. The reflected signal frequency characteristics are shown in Figure 1b. The frequency peak, *f*, or "thickness resonance" represents the repetition of reflected arrivals, or arrivals per second. The inverse of f (frequency as measured in the frequency domain) is then the round-trip travel time. Therefore, Equation (1) becomes:

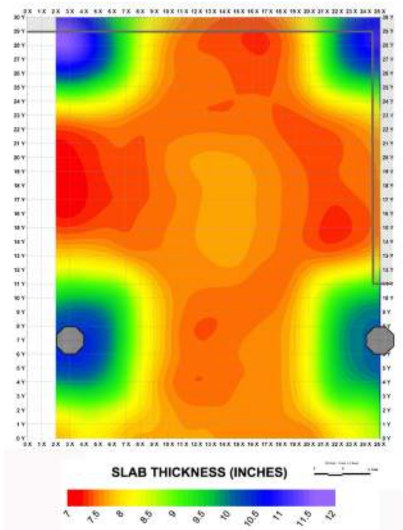
$$\text{Thickness} = V_p\,/2f \qquad (2)$$



(a) Implementation and Wave Paths

(b) Resulting Frequency Spectrum

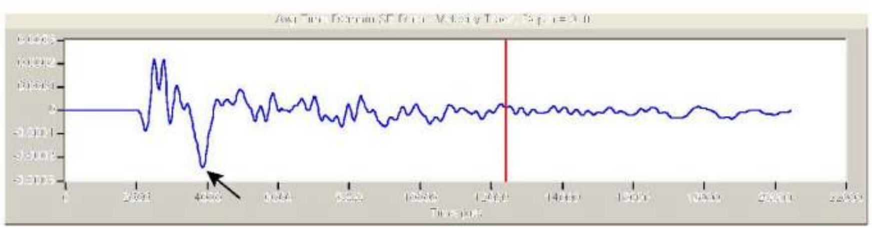
For the survey inside the building, IE data were collected at 6 inch station spacing along lines spaced 6 inches apart for a highly detailed survey. Outside of the building, IE was conducted in several locations along the exposed foundation wall, as shown on Figure 1B. For each measurement, one file is generated, and is evaluated using WinIE©, developed by Olson Engineering for processing and evaluating IE data. Thickness information is recorded in an Microsoft Excel© Spreadsheet, as a function of X and Y position, and Z depth, where Z represents thickness of concrete in inches. Typically, data are from the Excel© spreadsheet is then contoured, with similar values being assigned similar colors, and presented as a color map, such as the example below:



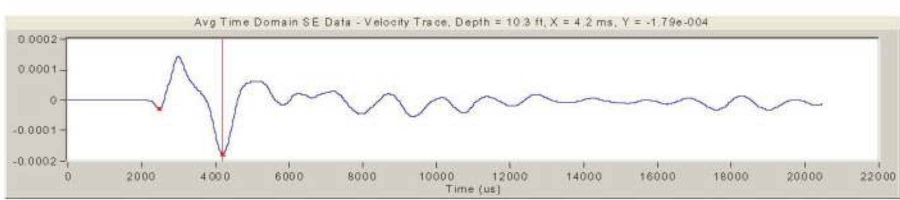
Sonic Echo/Impulse Response

Sonic echo uses the same principles as IE, reflecting back at interfaces where there is a negative acoustical impedance (i.e. a lower impedance than the concrete), except that the source is an instrumented hammer and the recording device is a 1,000 hertz (Hz) accelerometer. Because a larger source is used, greater energy is produced, which enables imaging of pier and foundation footings up to 100+ feet. Below is an example of annotated output from a SEIR survey looking for the depth of a central tower foundation.

Central Tower



Raw data from Ch 2 (Accelerometer) and Plot 3 (integrated data) – The echo is apparent (arrow)



Integrated Acceleration Data with a lowpass filter of 1000 Hz.. - The depth is 10.3 ft

For this survey, a 3 pound Dytrans hammer was used for the source, and a 1,000 Hz accelerometer was affixed to the top of both south and east foundation walls where there was sufficient room for the accelerometer and for the hammer to strike the top of the foundation wall. The other siting criteria for outdoor SEIR measurements was proximity to the equipment: while the hammer can extend up to 100 feet from the NDE 360, the Accelerometer must be within approximately 6 feet of the instrument, otherwise there can be noise issues. For this reason, RSI could only survey 10 feet from the southeast building corner along the east foundation wall. Figure 1C shows the locations of SEIR measurements.

RESULTS

The attached Figures summarize our interpreted results, as summarized on Figures 1A through 9. Key results are summarized below. Please note, the scale ranges from 1 inch = 4 feet to 1 inch = 1 Foot, and is as noted at the bottom right of each figure.

<u>Slab and Slab Support Assessment</u>

In order to use IE around a 10x8 foot box surrounding Column 5, the carpeted tiles had to be removed, exposing the bare concrete. The surface showed several defects, including several areas of spalled and chipped concrete, and a long crack about 10 inches west of the column line and paralleling it. The long crack delineates the boundary from where the slab appears to be level, from where the slab appears to be settling. The photo (LEFT) shows the most prominent crack, while the RIGHT photo shows the same photo, but annotated.

This observation shows physical evidence for the presence of a grade beam. The surface condition of the concrete was problematic for getting sufficient energy into the concrete, as

the cracking, chipping, and spalling, and likely as well as residual adhesive, produced an uneven surface to which to couple the IE receiver. For this reason, data from the solenoid impactor was not collected. Moreover, when RSI used a heavier 12 oz ball pein hammer, often the concrete sounded hollow, and the sound emanating from the hammer was more like a “thud” rather than a ping in several areas. Multiple strikes had to be conducted at the same station. Impact echo data were evaluated by RSI, and their expert, Larry Olson, Founder and President of Olson Instruments and Engineering, Inc. and Vice President Dennis Sack.
In the end, between poor coupling and ambient electrical noise, only a handful of the nearly 300 points collected had sufficiently clear data to interpret.

Figure 2 shows representative and lower-noise values determined from the Impact Echo measurements. Away from column centerlines, shown on the AutoCAD map provided by SGH and its client, it shows that the thickness of the slab on grade is nominally about 5.5 to 6 inches in thickness. Concrete appears significantly thicker along column lines, ranging from about 28 to 35 inches in thickness. This, along with physical evidence shown in the above photo, would suggest that the grade beams exist along column lines, and that they are approximately the same width, or slightly less, as the approximately 21x21 inch width of the pedestal (show beneath the column in the photos above and as the pink square on Figure 2). Figure 2 also shows that total concrete thickness coincident with the pedestal, ranges from about 24 to 34 inches. The larger thicknesses are indicated at the edges of the grade beam-pedestal boundary, and so it may indicate that the grade beams are incorporated into the pedestal or that the measurement is picking up side reflection from the grade beam.

Figure 2 also shows that the northwest quadrant (i.e. the quadrant to the left and top of the column as one is looking up-station) has no measurements. Here, we observed the most interference from electrical noise.

Figure 3A shows contoured slab thicknesses as determined from the GPR 1500 MHz survey. Based on the color-filled contours, the majority of the slab appears to be about 6.3 to 7.2 inches in thickness. This is slightly higher than the 5.5 to 6 inch thicknesses identified using IE. This difference is likely explained by an over-estimation of GPR signal velocity, slower than the 3.94 inches per nano-second (in/ns) used, or an IE velocity slower than 12,000 ft/s used. Either way, we believe that the slab is nominally 6 inches in thickness. Figure 3D shows interpreted results from both methods.

Visual inspection of the 1500 MHz data also shows that the slab on grade has mesh that is 6x6 inch on center. Moreover, the small reflection would suggest a finer gage wire used for the mesh– likely 1/4 to 3/16th inch in diameter. We also observe areas of overlapping mesh, which is most evident on the GPR depth slice shown on Figure 4E. These seams appear to overlap at 4.5 foot intervals in the (grid) north-south direction, but is less evident in the (grid) east-west direction.

Figure 3A also shows three areas of very thick concrete, two located in the southern portion of the area of investigation, and one around 26X and 65Y. Interestingly, deep concrete was not indicated along the column lines. We believe this is because the grade beams are below, and not incorporated, into the slab on grade, and that the mesh provides sufficient interference to not to see much deeper, whereas these other "thick" areas of concrete had much deeper structural steel. The grade beam is best shown on the 500 MHz slice data on Figure 7K, where there is a "+" shaped area of attenuation with the column centered around it.

From the 1500 MHz data, in the large thick area located along Lines 21.25X through 25X, from 13Y through 21Y, we also observe mesh, but this mesh is almost 12 inches below grade, rather than 3.5 to 4 inches below grade relative to the rest of the slab. Areas of thick concrete are also visible on depth-slices on Figures 4J through 4M.

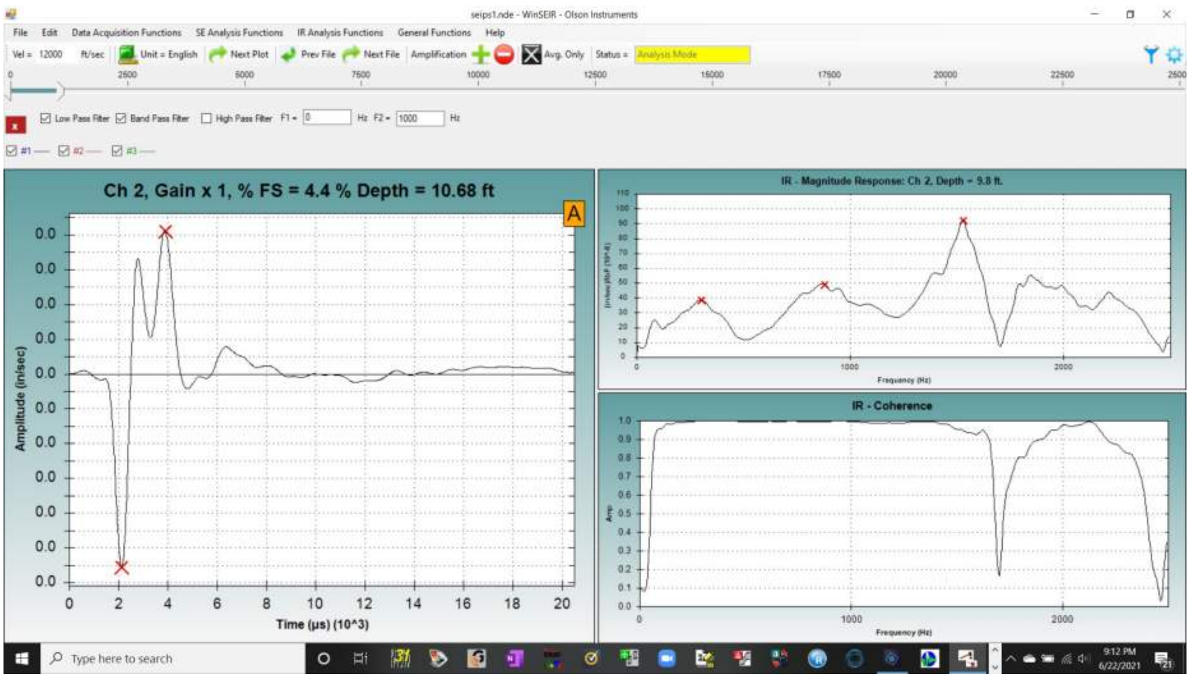
1500 MHz data also shows a second reflective layer below the slab. This has been presented on Figure 3C. While the bottom of this layer is relatively the same thickness throughout the area of investigation, we believe that it represents a granular material, such as a gravel layer. This layer is nominally 12 inches below grade, indicating a typical thickness of 6 inches. The only exception is in proximity to the area of deep concrete that is visible along Lines 21.25X to 25X, and 15Y to 21.5Y, and not located along any column lines. From about 12Y to 26Y we observe the bottom of this second layer about 14 to 16 inches below top of slab. Moreover, within the area of thick concrete, we observe a weak reflector at the same depth. We believe that this reflector could represent the bottom of the thick concrete, and that this area has been modified, for whatever reason, with the bottom of the interpreted gravel layer at the same depth below grade with the interpreted bottom of the thick concrete.

Figure 3C is a color-filled contour plot showing the depth to a third reflective layer. As this layer changes depth over a relatively short distance, we believe that this reflector is from the bottom of a subbase material. This third reflector is also mostly visible in the southeast section of the grid, just south of the thick concrete area. Here, depth to the bottom of this layer is about 16 to 20 inches below grade. North of the thick concrete area, we do not observe any deeper layer. However, we do observe an occasional large hyperbolic reflector. We have interpreted a buried conduit (or group of conduits), shown in long black dashed rule on Figure 3C, where these reflectors of same approximate size and depth align from line to line. Other isolated weak and high-amplitude reflectors are attributed to settlement of fill and/or voiding. We also observe evidence of at least one conduit trending towards Column 5B from that northwest quadrant on depth slices on Figures 4B and 4C, and 4N and 4O.

On the various depth slice images from 1500 MHz and 500 MHz data, we also observe a lot of broad, high-amplitude reflectors (Figures 4A through 4O, and 7A through 7M) Some of the shallow high-amplitude reflections may be attributed to delamination between the mesh and slab, such as observed on Figures 4C and 4D. High-amplitude reflections observed on deeper depth slices could be attributed to chimney type voids and areas of loosely packed fill material.

Sub-Slab Support and SEIR Results from Indoors

The purpose of conducting SEIR in proximity of the column and on top of the pedestal was to determine how the column is supported. Based on IE data, we believe that the pedestal is about 24 to 34 inches in thickness, and that there are grade beams tied into the pedestal. What was unknown prior to the SEIR survey was if there was a timber pile (or timber piles) beneath the pedestal. Three measurements were made at three locations, approximately 120 degrees from each other, around the column. The data shows that at each location where the measurement was taken, the pedestal is in structural contact with a pile. We can not tell whether that there are three piles or one large pile, just that the pedestal is in structural contact with it/them. Figure 5 shows that the depth to the bottom of the wood timber pile is about 10.7 to 10.8 feet, based upon the arrival time of the highest positive peak. Below is a representative SEIR record from the SEIR survey at column 5.



Typically, the polarity of the highest peak is negative, assuming that the wave is traveling from a higher impedance material to a lower impedance material. The positive peak, shown below, suggests that the wood pile timber is imbedded in a higher impedance material at about 10.7 feet. Based on HGI's August 2020 report and from SGH personal communications, we believe that this pile sits on top of a compact glacial till.

The Impulse Response data, plotted as a function of frequency, confirms the approximate 10 foot depth of the timber pile. If anything, it slightly under-predicts the pile depth when compared to the amplitude plot in the time-domain. In the frequency domain, we observe resonant frequencies at 300 Hz, 900 Hz, and at 1500 MHz. The first peak at 300 Hz is half the difference of frequencies between 900 Hz and 300 Hz, and 1500 Hz and 900 Hz, also suggesting that the timber is in structural contact with a higher impedance material. The resonance frequency is also what determines the timber depth.

Determination of Presence or Absence of Wooden Timber Piles beneath the slab

Inside the building, there was a question as to how the building's slab is supported. HGI's previous GPR investigation summarized in their August 2020 report, suggested that wooden timber piles were located 4 feet below the top of slab. No additional SEIR was conducted by RSI to determine whether there were wooden timber piles beneath the grade beams in between the columns. This was out of concern that a large amount of carpet tiles would first have to be removed in between two columns, potential damage to those tiles that were already removed once, and the permissions required to do it.

However, RSI conducted a 500 MHz GPR survey, using a high-density (1.25 foot) line spacing and GPR slice to image GPR, to confirm HGI's interpretation. Based on the dielectric properties of the slab, wooden timber pile, and fill, we would anticipate a weak to moderate-amplitude, flat reflection as the antenna passes over the timber pile. We would also expect that a timber having a 12 inch cross-section would produce diffractions (i.e. hyperbolic "tails") at the edges of the top of the timber pile. Moreover, if there were rows of timber piles, we would expect a repetitive reflective pattern, which would be evident on both GPR depth-slices and on the GPR records in the 3D volume. Our interpreted results from the visual inspection are summarized on Figure 6. The GPR data is not of high quality, which is due to EM energy energizing the mesh and creating noise in the GPR data. However, it is clear that we do not see a repetitive pattern of flat (or hyperbolic) GPR reflectors. Rather, we observe in several areas where flat and hyperbolic reflectors, observed typically in the 15 to 24 inch depth range, that align from line to line. This indicates a target that is horizontally oriented, not vertically. We believe, especially as there is no repetitive pattern, that these are reflections from buried conduits and utilities.

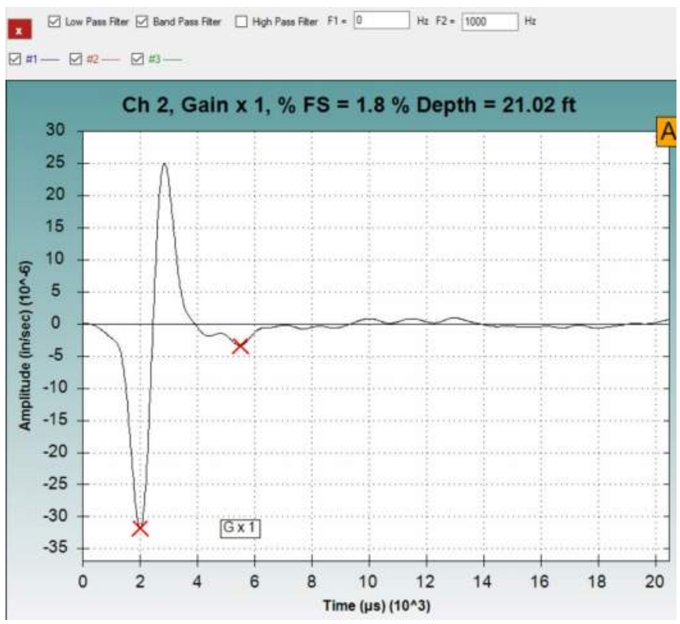
500 MHz GPR slice images are presented on Figures 7A through 7M. Figure 7I shows a series of circular, magenta lined ellipses along Line 22.5N, between 24Y and 38Y. Again, if there were wooden timbers, we would expect a repetitive pattern of flat (or hyperbolic) reflections, which we do not see. Moreover, the depth of this slice is in the 2.0 to 2.5 range, well below the slab, which we observed at 6 inches, and the granular material that extends to about a foot below grade. Likely, these circular reflective patterns in the base material represent chimney type voids rather than wooden timber piles. We just do not see evidence of the wooden timber piles in the GPR slice data.

Condition of Fill Beneath Slab on Grade

There is some evidence of delamination in the slab between the mesh and concrete. This is evident in the shallow GPR depth slice images of the 1500 MHz data. The very high amplitude reflections are caused by the reverberation of trapped EM energy between the reflective metal and concrete within the air gap. In localized areas below the slab, we also observe broad, high-amplitude reflections that would suggest that fill has settled from the slab. The 500 MHz depth slices also show potential voiding west and northwest of Column 5B. We also observe possible loose fill (and potentially voided fill) in the southwest and southern portions of the area of investigation. We also observe a broad reflective area immediately north of the deep concrete area in the central portion of the site. This adds speculation as to why the area of thick concrete was repaired, if that was the reasoning for it. Settlement of fill, especially given the age of the building and proximity to the Ipswich River, is expected over time. However, further evaluation by means of a geotechnical boring would help evaluate conditions below the investigative depth of the GPR and confirm conditions interpreted here.

Foundation Wall Evaluation

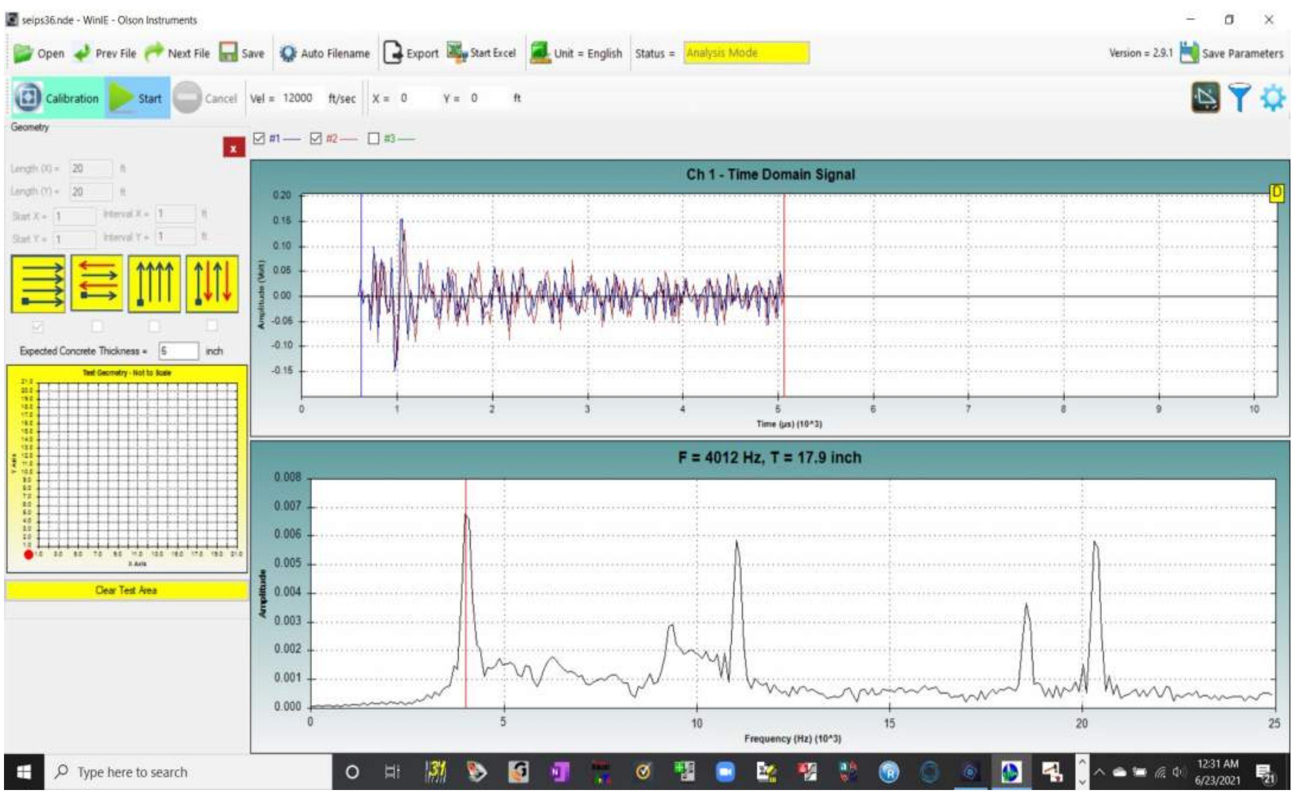
SEIR data from outside the building was used to evaluate foundation wall depths of the south and east walls, respectively. Figure 8 summarizes the depth of the foundation walls based upon location along the wall. Below is an example amplitude plot located about 50 feet east of the southwest corner between the original building and addition. Note that the highest reflection after the hammer reflection attributed to the hammer is negative, indicating that the material beneath the concrete wall has a lower
impedance than the concrete.



SEIR data indicate that the south wall deepens away from the new addition, where the south foundation wall is about 18 feet in depth 2 feet east of the corner, and deepens to 21 feet in the middle of the wall, between 35 and 50 feet east of the SW corner. At the southeast corner of the historic building, the wall appears to be about 20 feet along the south and east walls (Figure 5). North from the SE corner of the building, the east wall appears to shallow, going from about a 19 foot depth 6 feet north of the SE corner to about 17 feet at 9.5 feet north of the corner. Please note, data on the east wall is of lesser quality than along the south foundation wall. We observe vertical cracking along the east wall about 1 foot north of the middle station. We also observe horizontal cracking in one location along the east wall. Reflections from these internal cracks of the foundation wall create noise in the data.

However, we are confident that the east foundation wall shallows to the north, likely mimicking the topography of the till.

Evaluation of IE data, the results of which are presented on Figure 9, were a little more challenging due to the top of the foundation wall getting closer to grade as one moves to the east along the south foundation wall. Hence, we could only obtain IE readings only at the western most stations, and of that, only the station at 22E, located 22 feet east of the SW corner between the addition and original buildings, produced good data. Here, IE indicates that the south wall has a thickness at the top of about 18 inches. Below is the record from Station 22E. The highest peak is from the actual wall-fill interface, while the other peaks are from ambient electrical noise. The top of the south foundation wall was not exposed east of Station 22E.



Along the east foundation wall, we were able to make two measurements, 2.5 and about 8 feet north of the SE corner (Figure 9). IE data at these locations indicate that the east foundation wall thickness at the top is about 2 feet in thickness.

FUTURE CONSIDERATIONS

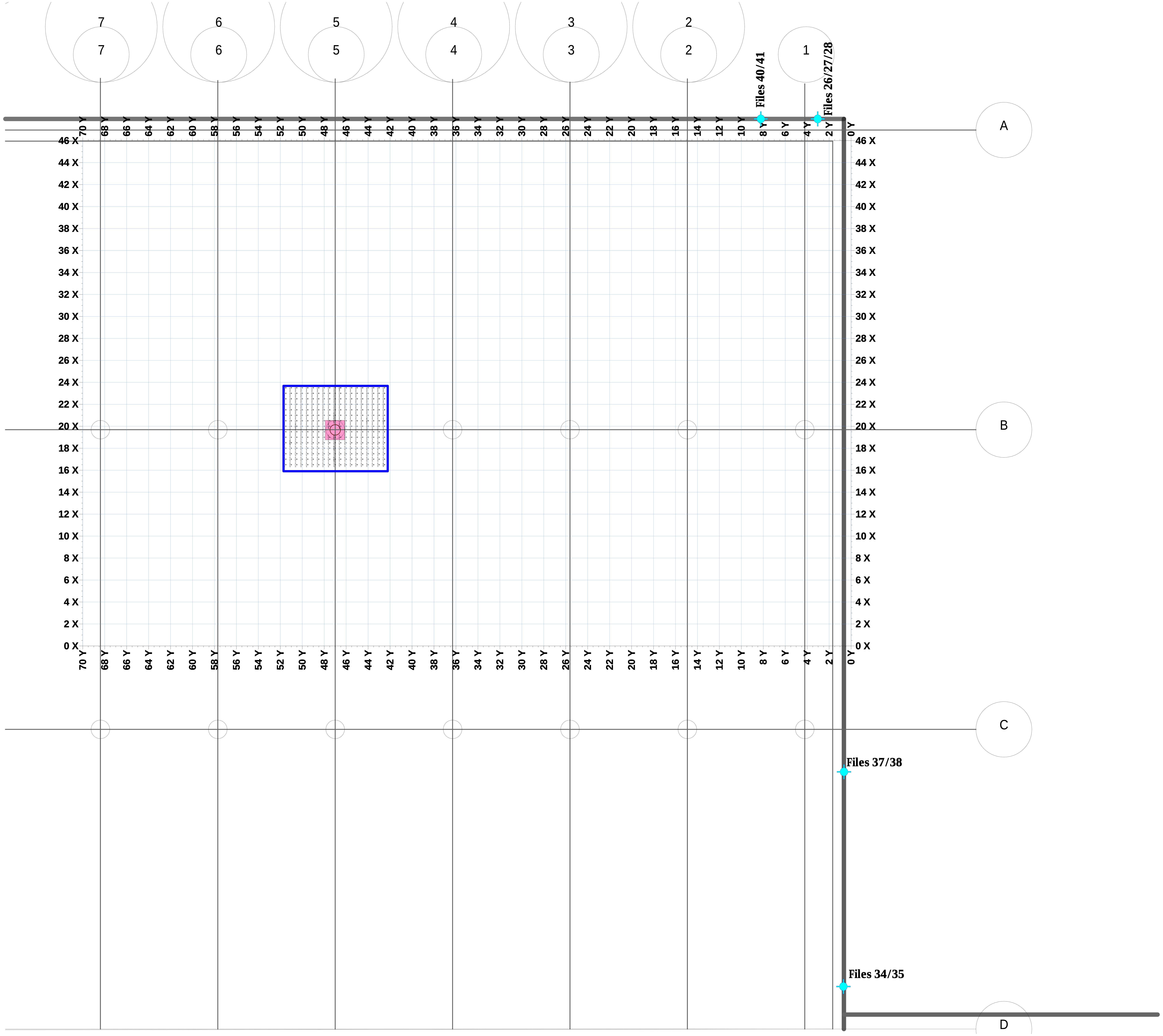
The nature of geophysics is interpretive, and reflections from GPR are non-unique. For this reason, RSI recommends that geotechnical borings be conducted to help determine the extent of voiding at depth beneath the slab. Additional work can also be conducted to determine how the grade beams are supported, if at all, using SEIR along the column lines. However, this requires removing carpeted tiles along one column line and cleaning down to bare concrete.

****

We appreciate the opportunity work on this project with SGH, and look forward to future opportunities. In the meantime, please do not hesitate to call or email to arrange a Zoom meeting to further discuss these findings.

Sincerely,
RADAR SOLUTIONS INTERNATIONAL, Inc.

Doria Kutrubes. M.Sc., P.G.
President and Sr. Geophysicist



LEGEND

Column 5, Used for Sonic Echo/ Impulse Response & Impact Echo Measurements

Limits of Impact Echo Testing on inside slab and Column 5 (6x 6 in. grid)

Impact Echo Testing Locations Along Outer (Top of) Foundation Walls

SCALE: 1 Inch = 4 Feet

4 0 4 Feet

FIGURE 1A
AREA OF INVESTIGATION
IMPACT ECHO LOCATIONS
EBSCO
10 ESTES STREET
IPSWICH, MASSACHUSETTS

Prepared for
SIMPSON GUMPERTZ & HEGER
JUNE 2021

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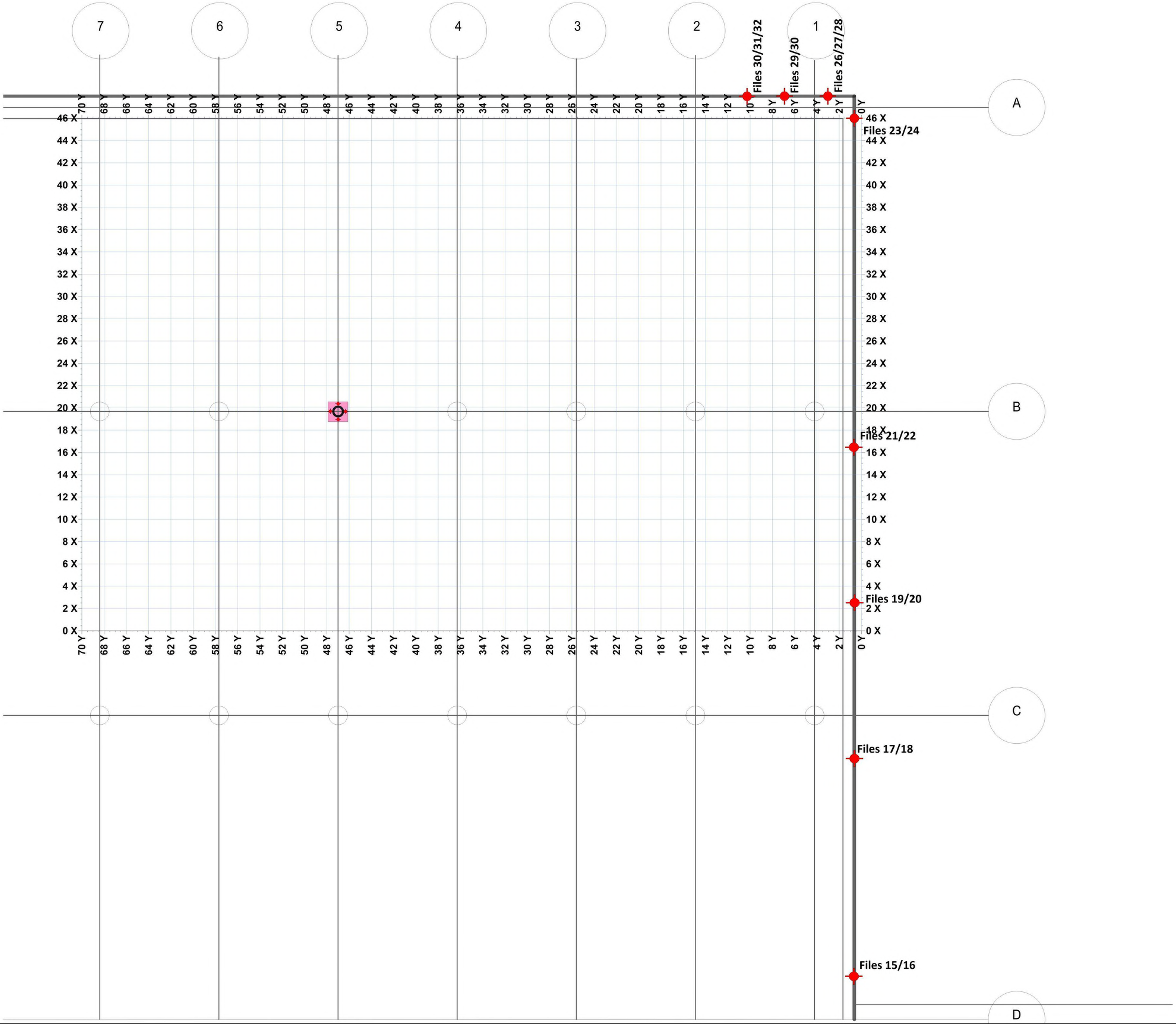

LEGEND

- Pedistal Around Column 5B
- Column 5B, Used for Sonic Echo/ Impulse Response Tests
- Location of Sonic Echo/ Impulse Response Measurements, Inside near Column 5B and Outside Along Outer South and East Walls

SCALE: 1 Inch = 4 Feet

4 0 4 Feet

FIGURE 1B
AREA OF INVESTIGATION
SONIC ECHO AND IMPULSE RESPONSE LOCATIONS
EBSCO
10 ESTES STREET
IPSWICH, MASSACHUSETTS

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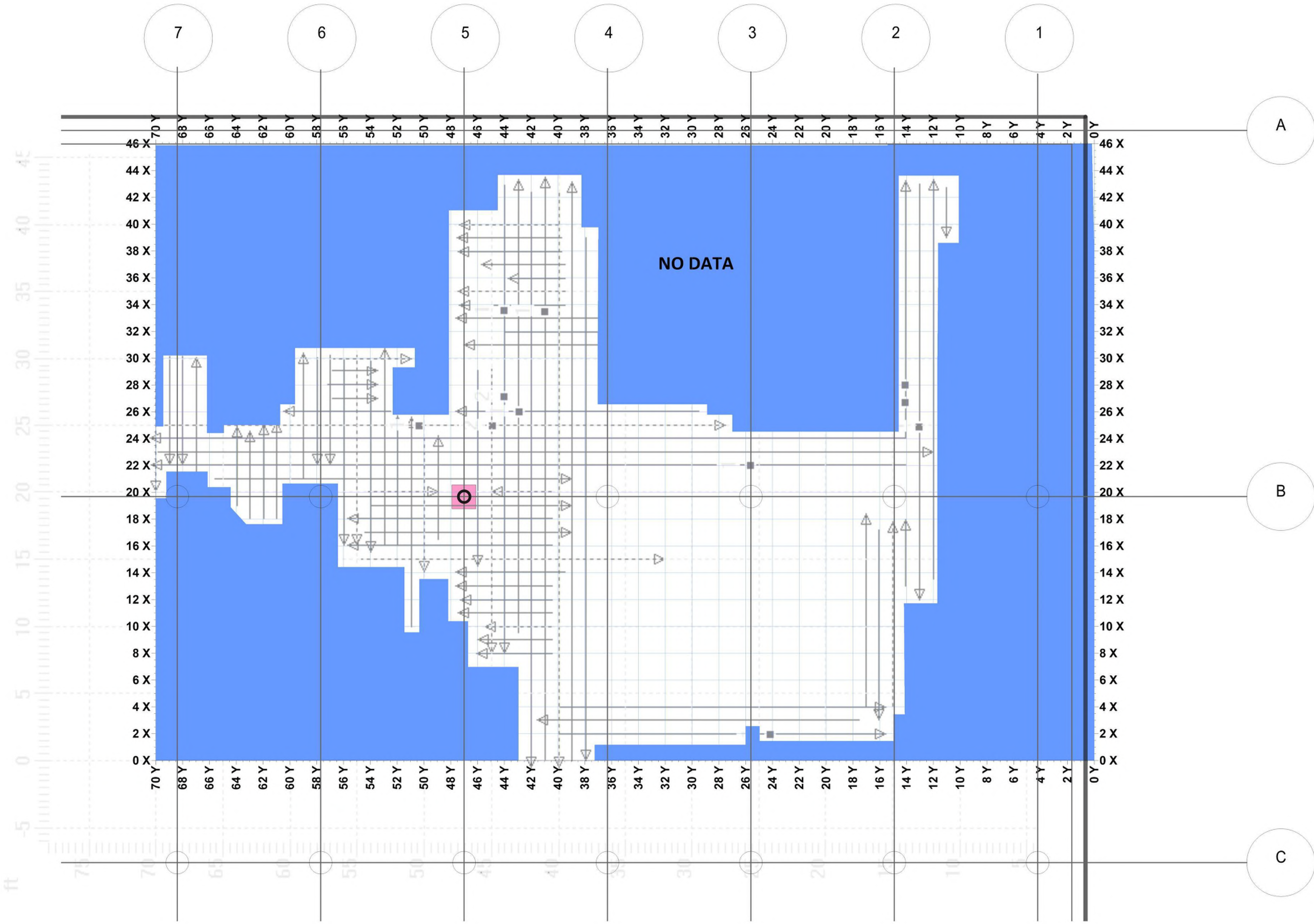

LEGEND

Pedistal Around Column 5B

Column 5B, Used for Sonic Echo/ Impulse Response Tests

Location and Orientation of GPR Traverse - 500 MHz Antenna

SCALE: 1 Inch = 4 Feet

4 0 4 Feet

FIGURE 1C
GPR AREA OF INVESTIGATION
500 MHz ANTENNA
EBSCO
10 ESTES STREET
IPSWICH, MASSACHUSETTS

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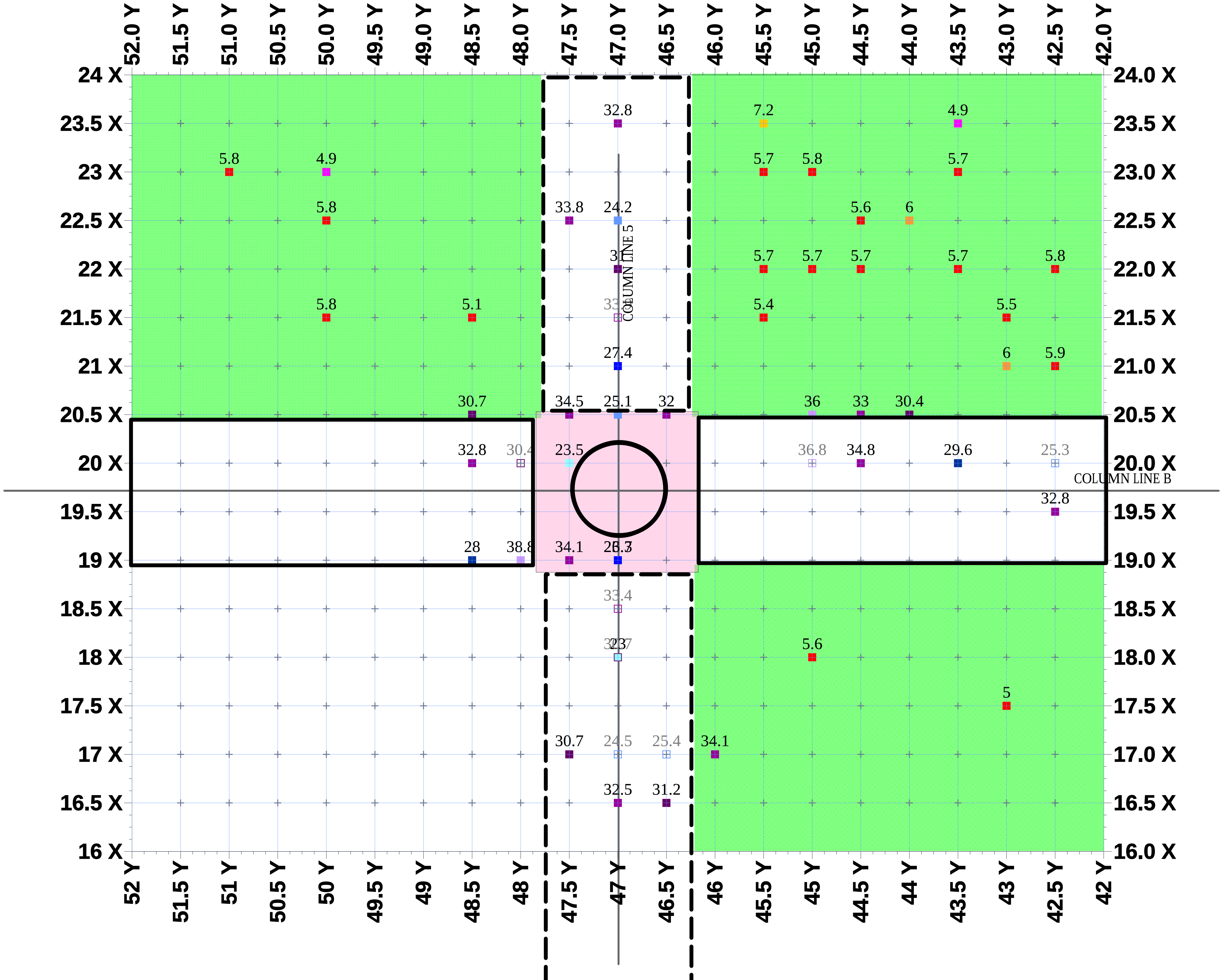

LEGEND

- Pedistal
- Location of Column 5B
- Location of Impulse Echo Measurement
- 5.8 Spot Depth (in.) of IE Measurement; Solid Square Denotes Good Confidence, while Hollow Square Denotes Tentative Value; Color (below) and noted Value Denote Depths and Approx. Depths (in.)
- Thin Concrete, Nominally 6" in Total Concrete Thickness
- Approx. Location of Grade Beam

DEPTH (in.)

- 4 to 5
- 5 to 6
- 6 to 7
- 7 to 8
- 8 to 12
- 12 to 18
- 18 to 20
- 20 to 22
- 22 to 24
- 24 to 26
- 26 to 28
- 28 to 30
- 30 to 32
- 32 to 36
- 36 to 42

NOTE: Poor surface condition of concrete and the presence of nearby electrical conduits produced significant noise in data.

SCALE: 1 Inch = 1 Foot

1 0 1 Foot

FIGURE 2
CONCRETE THICKNESS FROM IMPACT ECHO
INLCUDING PEDISTAL AND
GRADE BEAM THICKNESSES
EBSCO
10 ESTES STREET
IPSWICH, MASSACHUSETTS
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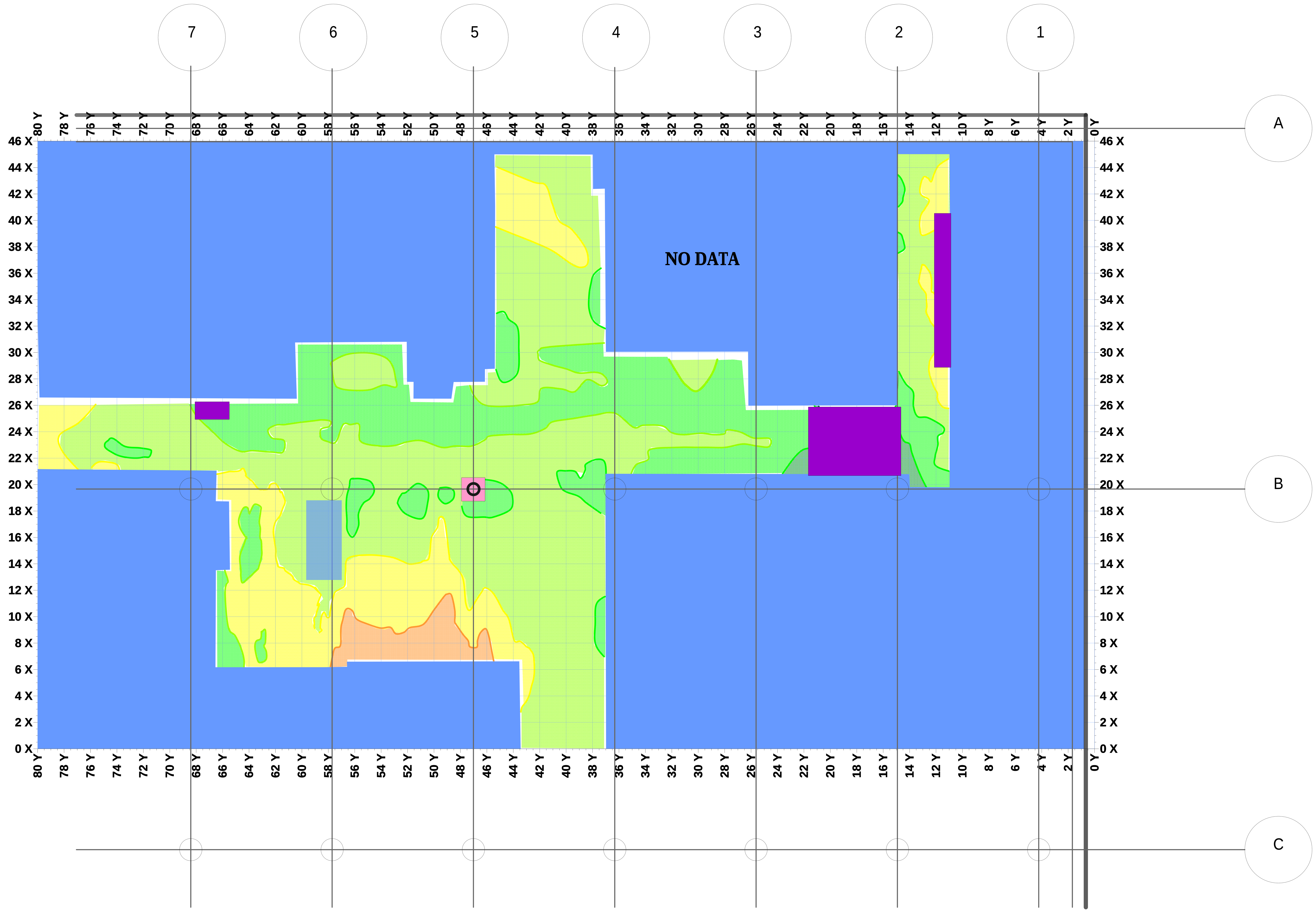

LEGEND

Pedistal

Location of Column 5B

GPR VISUAL INSPECTION LEGEND

Location of THICK CONCRETE

Approximate Structural Slab Thickness from GPR Visual Inspection of 1500 MHz Data. Color Indicates Depth (in.) as noted below:

DEPTH (in.)

- 4.5 to 5.4
- 5.4 to 6.3
- 6.3 to 7.2
- 7.2 to 8.1
- 8.1 to 9.0

SCALE: 1 Inch = 4 Feet

4 0 4 Feet

FIGURE 3A
APPROXIMATE STRUCTURAL SLAB THICKNESS FROM VISUAL INSPECTION OF 1500 MHz GPR DATA
EBSCO
10 ESTES STREET
IPSWICH, MASSACHUSETTS

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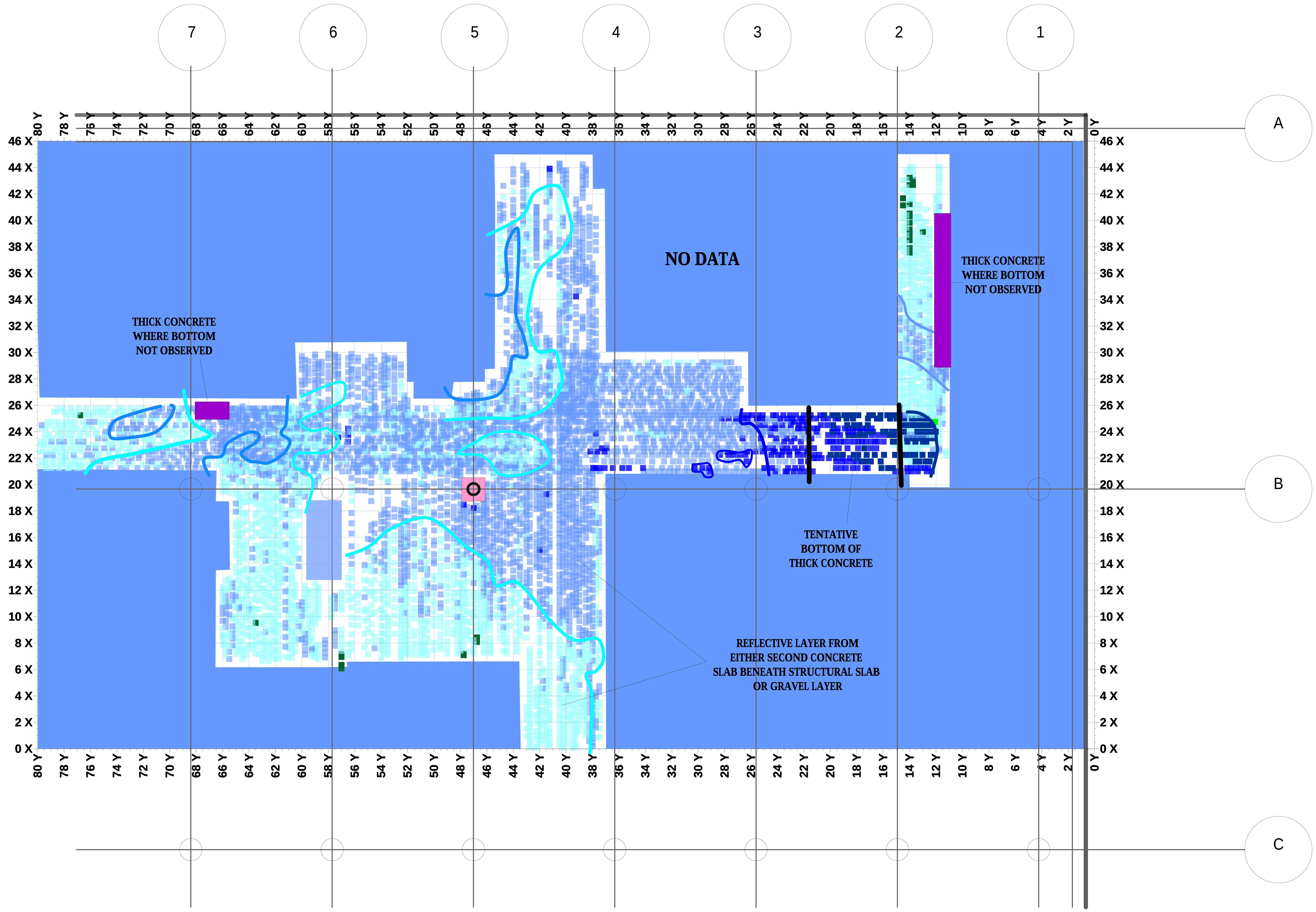

LEGEND

Pedistal

Location of Column 5B

GPR VISUAL INSPECTION LEGEND

Location of THICK CONCRETE

Approximate Structural Slab Thickness from GPR Visual Inspection of 1500 MHz Data.

Approximate Depth to Bottom of Second Reflective Layer; Depth (in.) to Likely Gravel Layer or Tentative Second Unreinforced Concrete Slab. Color Indicates Depth (in.) as noted below:

DEPTH (in.)

- 0 to 1
- 1 to 2
- 2 to 3
- 3 to 4
- 4 to 5
- 5 to 6
- 6 to 7
- 7 to 8
- 8 to 9
- 9 to 10
- 10 to 12
- 12 to 14
- 14 to 16
- 16 to 18
- 18 to 20
- 20 to 24

SCALE: 1 Inch = 4 Feet

4 0 4 Feet

FIGURE 3B
APPROXIMATE DEPTH TO BOTTOM OF
LIKELY GRAVEL LAYER OR TENTATIVE SECOND SLAB
FROM GPR VISUAL INSPECTION OF
1500 MHz GPR DATA
EBSCO
10 ESTES STREET
IPSWICH, MASSACHUSETTS
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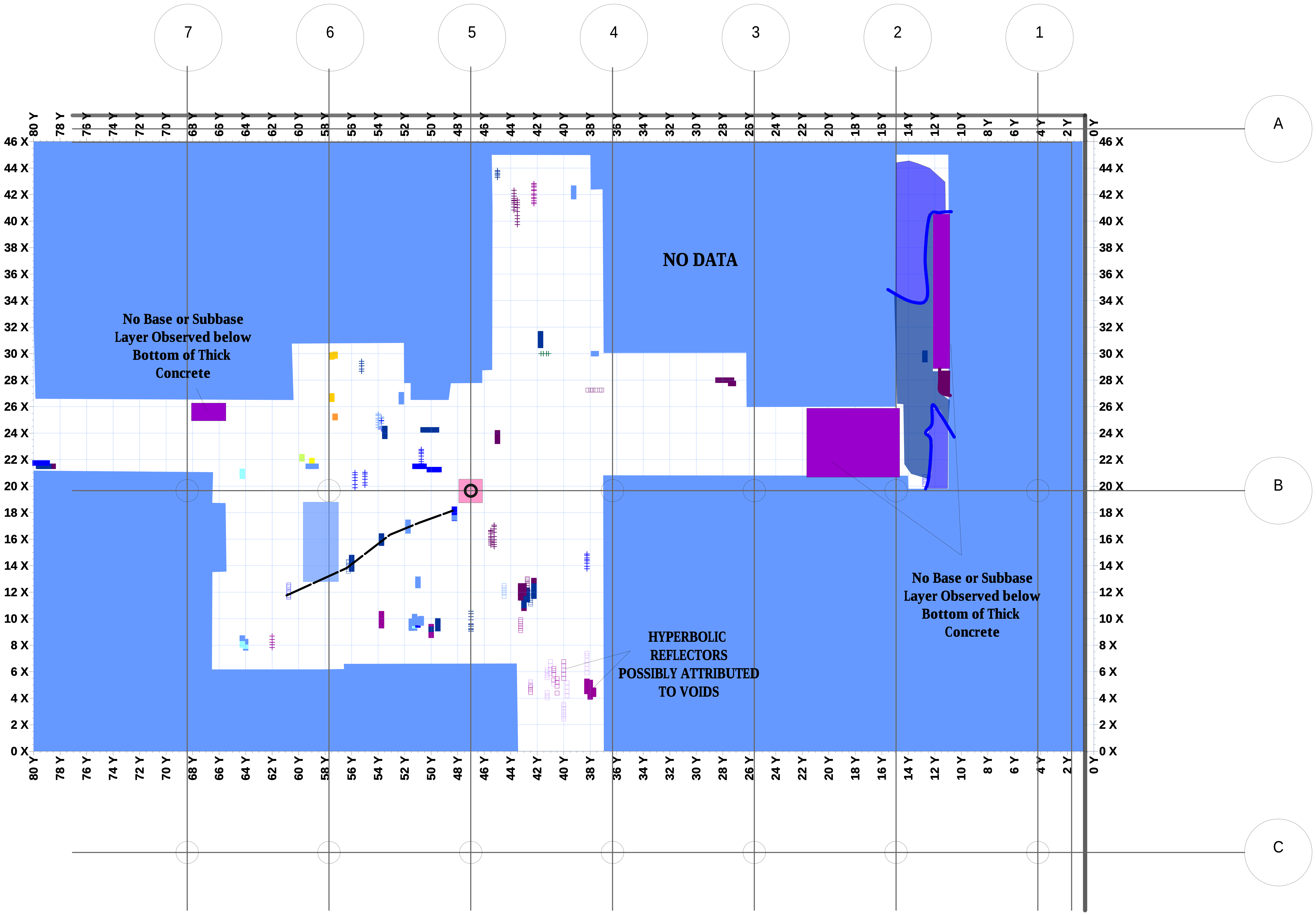

LEGEND

Pedistal

Location of Column 5B

GPR VISUAL INSPECTION LEGEND

Location of THICK CONCRETE

Approximate Structural Slab Thickness from GPR Visual Inspection of 1500 MHz Data.

Approximate Depth to Third Reflective Layer; Depth (in.) to Likely Top of Subbase or Native Material Color Indicates Depth (in.) as noted below

Large, High-Amplitude Hyperbolic Reflection; Reflection Attributed to Conduit, Chimney-Type Void, Cobble

Large, Weak-Amplitude Hyperbolic Reflection; Reflection Attributed to Chimney-Type Void or Cobble

Flat GPR Reflector

DEPTH (in.)

0 to 1
1 to 2
2 to 3
3 to 4
4 to 5
5 to 6
6 to 7
7 to 8
8 to 9
9 to 10
10 to 12
12 to 14
14 to 16
16 to 18
18 to 20
20 to 24

SCALE: 1 Inch = 4 Feet

4 0 4 Feet

FIGURE 3C
APPROXIMATE DEPTH TO LIKELY BASE LAYER AND OTHER FEATURES FROM GPR VISUAL INSPECTION OF 1500 MHz GPR DATA
EBSCO
10 ESTES STREET
IPSWICH, MASSACHUSETTS
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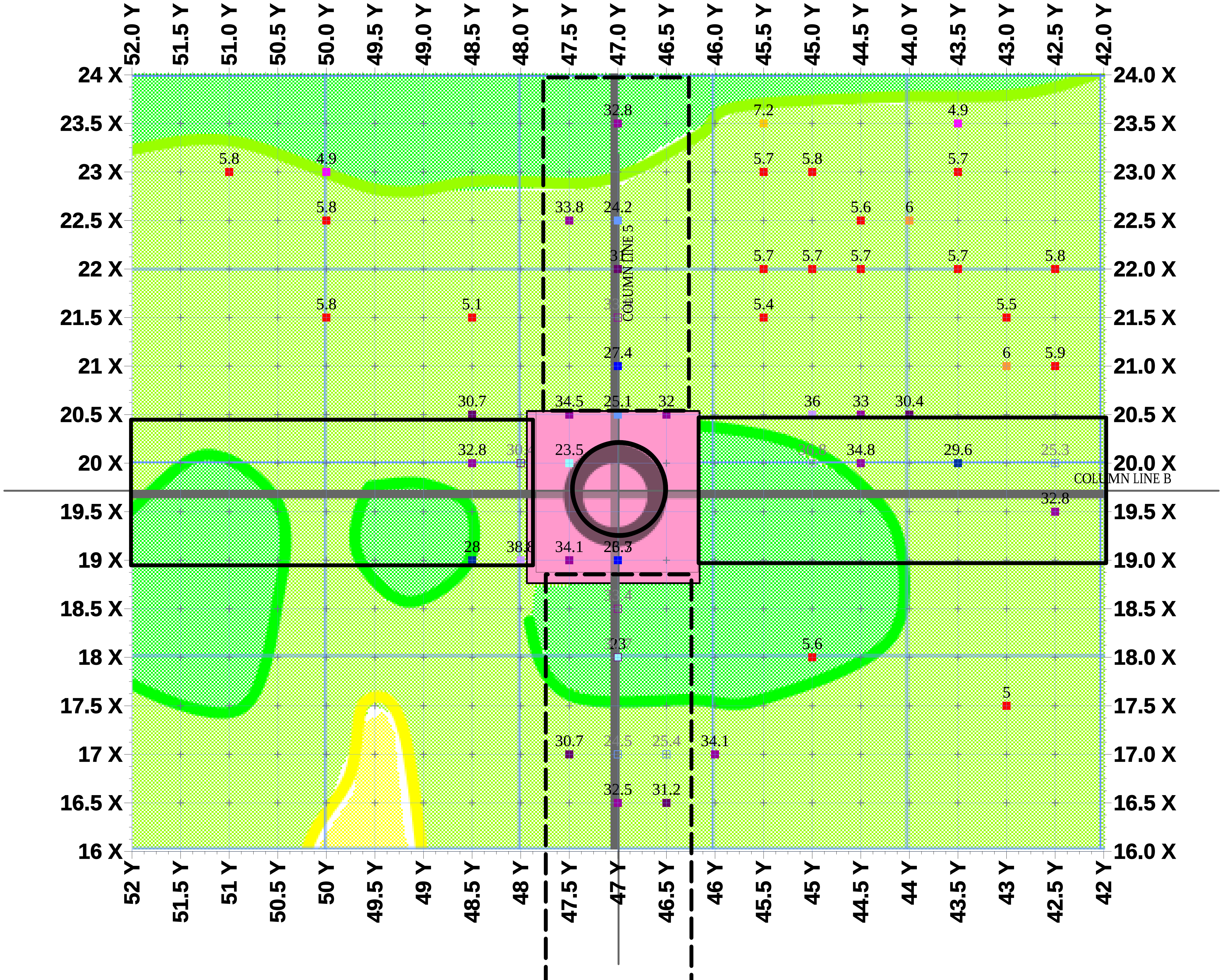

LEGEND

Pedistal

Location of Column 5B

Location of Impulse Echo Measurement

5.8 Spot Depth (in.) of IE Measurement; Solid Square Denotes Good Confidence, while Hollow Square Denotes Tentative Value; Color (below) and noted Value Denote Depths and Approx. Depths (in.)

Thin Concrete, Nominally 6" in Total Concrete Thickness

Approx. Location of Grade Beam from IE

Thickness of Structural Slab from GPR; Depth (in.) as denoted below:

DEPTH (in.)
- 4.5 to 5.4
- 5.4 to 6.3
- 6.3 to 7.2
- 7.2 to 8.1
- 8.1 to 9.0

NOTE: Grade beams were not visible below the structural slab in the GPR Visual Inspection.

Differences in structural slab thickness between GPR and IE are attributed to noise in IE data due to proximity to conduits and other electrical noise, and likely over-estimation of acoustic velocities in defective concrete, and/or error associated with identifying bottom of slab reflector with interference from nearby and overlying mesh reflector superimposed on it.

SCALE: 1 Inch = 1 Foot

1 0 1 Foot

FIGURE 3D
COMPARISON OF IE AND GPR THICKNESSES OF CONCRETE INLCUDING PEDISTAL AND GRADE BEAM THICKNESSES
EBASCO
10 ESTES STREET
IPSWICH, MASSACHUSETTS
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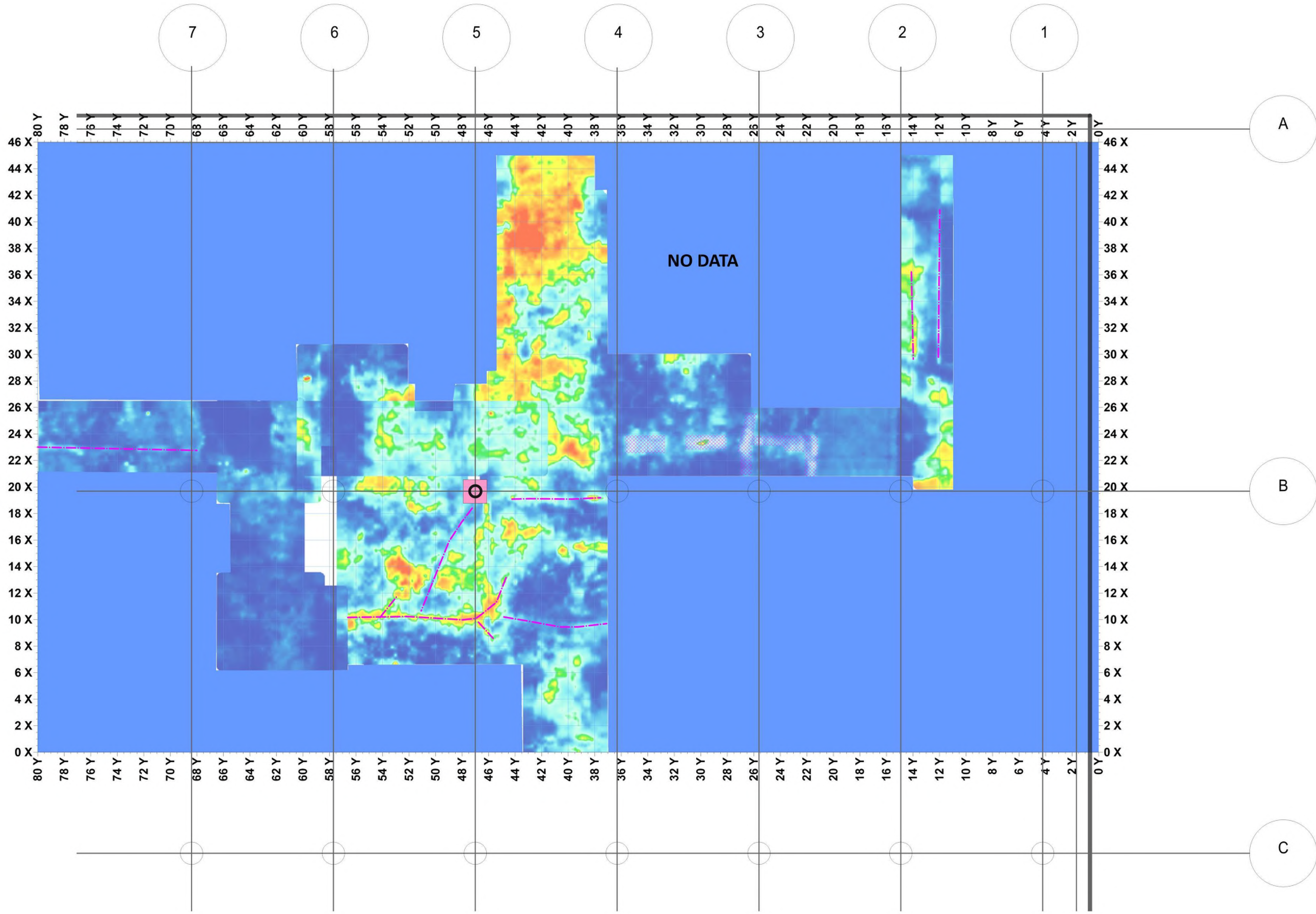

LEGEND

Pedestal

Location of Column 5B

1500 MHz DEPTH SLICE LEGEND

Linear, High-Amplitude Reflection that may indicate rebar or seam in Mesh, or Conduit; Depth (ft.) as indicated on slice at which lineation first appears

Zone of High-Amplitude GPR Reflections that could indicate an excavation, voiding, boulder, or cross-sectional reflection from a vertically oriented timber pile. High-amplitude reflections can also indicate a localized area of increased moisture

Zone of Attenuation that may indicate an area of thick concrete, increased soil moisture, and/or ground-water conductivity

DEPTH: 0.7 in. - 1.4 in.

increasing amplitude

low (weak) amplitude

high (strong) amplitude

SCALE: 1 Inch = 4 Feet

4 0 4 Feet

FIGURE 4A
REPRESENTATIVE DEPTH SLICE IMAGE
OF 1500 MHz GPR DATA
FOR VOID DETECTION
EBASCO
10 ESTES STREET
IPSWICH, MASSACHUSETTS
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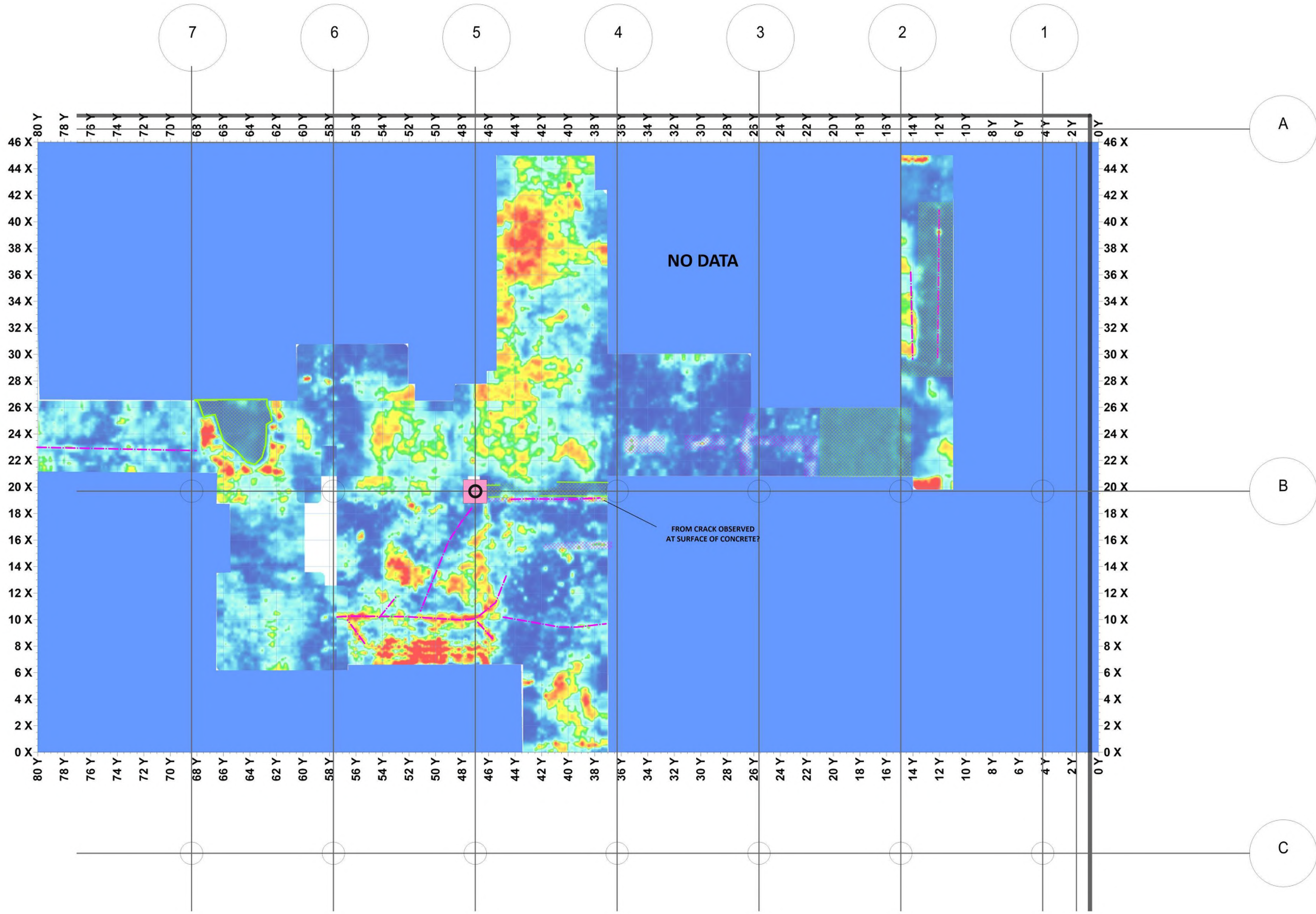


LEGEND

Pedestal

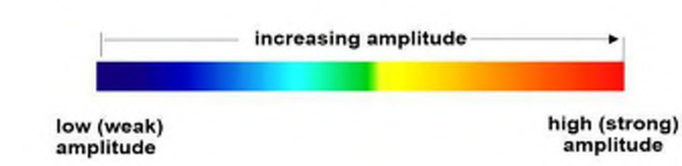
Location of Column 5B

1500 MHz DEPTH SLICE LEGEND

Zone of High-Amplitude GPR Reflections that could indicate an excavation or voiding, boulder, localized area of increased moisture

Zone of Attenuation that may indicate an area of increased soil and/or ground-water conductivity

DEPTH: 1.8 in. - 2.4 in.



SCALE: 1 Inch = 4 Feet

4 0 4 Feet

FIGURE 4B
REPRESENTATIVE DEPTH SLICE IMAGE
OF 1500 MHz GPR DATA
FOR VOID DETECTION
EBASCO
10 ESTES STREET
IPSWICH, MASSACHUSETTS
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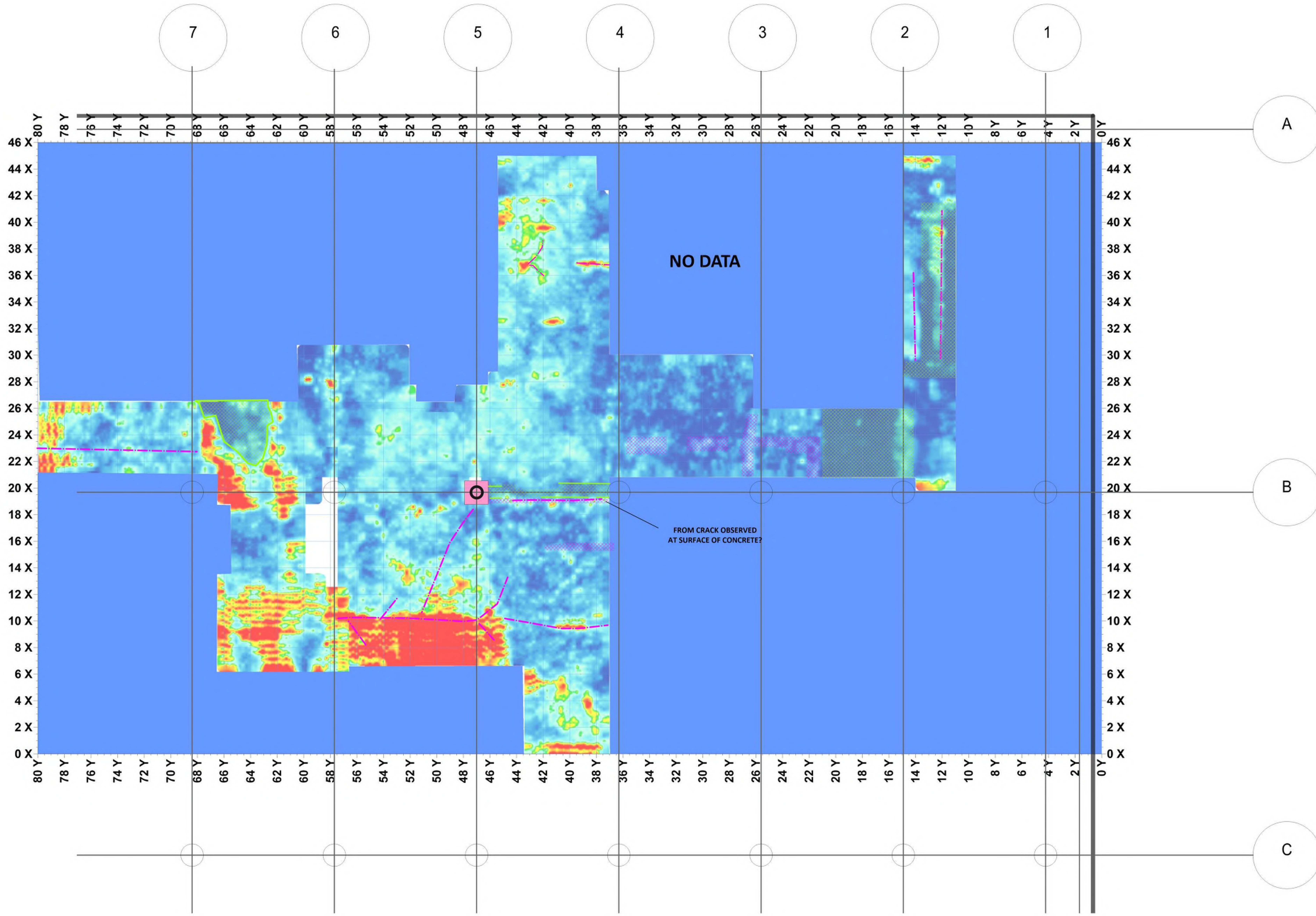

LEGEND

Pedestal

Location of Column 5B

1500 MHz DEPTH SLICE LEGEND

Zone of High-Amplitude GPR Reflections that could indicate an excavation or voiding, boulder, localized area of increased moisture

Zone of Attenuation that may indicate an area of increased soil and/or ground-water conductivity

DEPTH: 2.4 in. - 3.1 in.

increasing amplitude

low (weak) amplitude

high (strong) amplitude

SCALE: 1 Inch = 4 Feet

4 0 4 Feet

FIGURE 4C
REPRESENTATIVE DEPTH SLICE IMAGE
OF 1500 MHz GPR DATA
FOR VOID DETECTION
EBASCO
10 ESTES STREET
IPSWICH, MASSACHUSETTS
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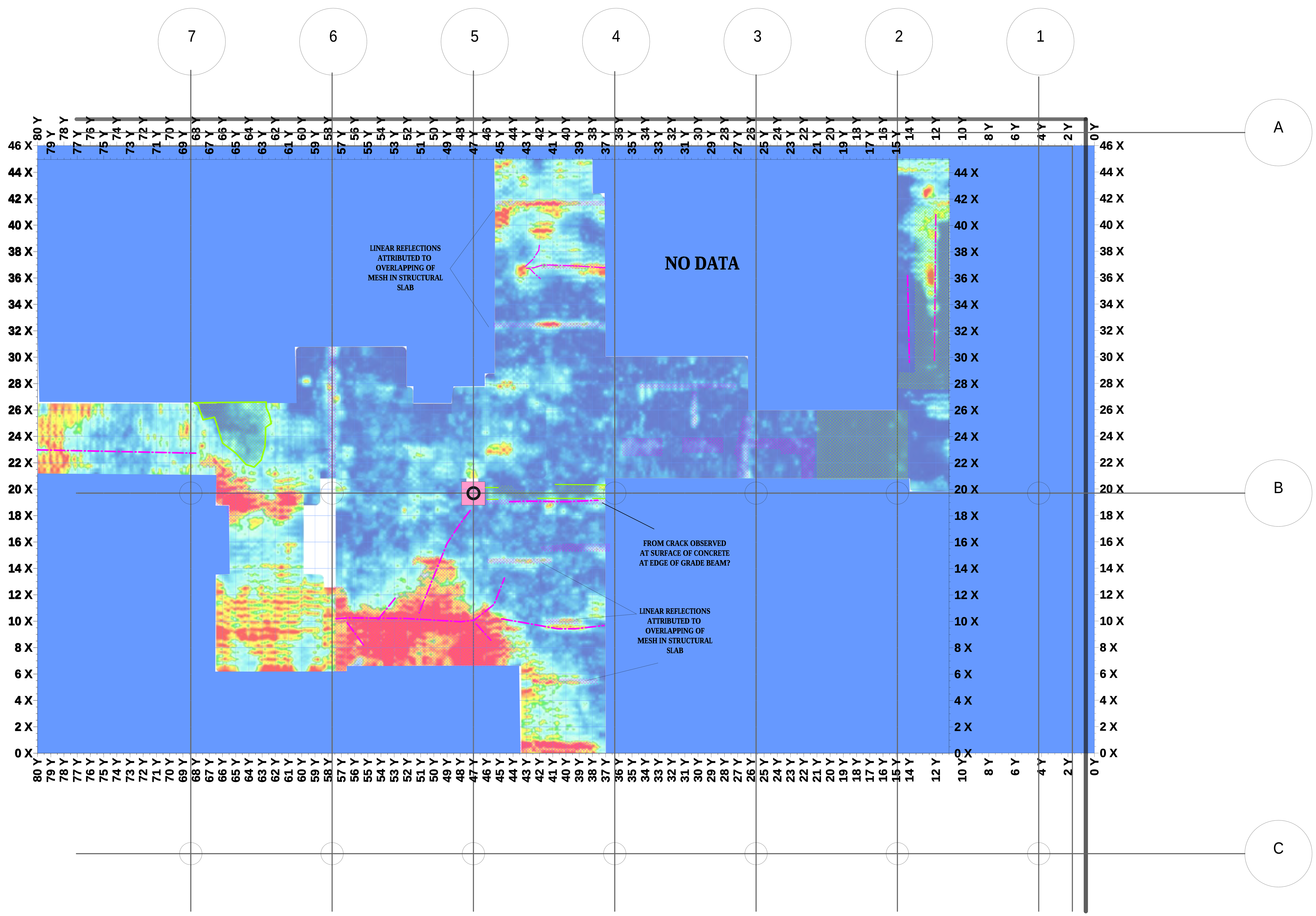

LEGEND

Pedestal

Location of Column 5B

1500 MHz DEPTH SLICE LEGEND

Zone of High-Amplitude GPR Reflections that could indicate an excavation or voiding, boulder, localized area of increased moisture

Zone of Attenuation that may indicate an area of increased soil and/or ground-water conductivity

DEPTH: 3.2 in. - 3.9 in.

increasing amplitude

low (weak) amplitude

high (strong) amplitude

SCALE: 1 Inch = 4 Feet

4 0 4 Feet

FIGURE 4D
REPRESENTATIVE DEPTH SLICE IMAGE
OF 1500 MHz GPR DATA
FOR VOID DETECTION
EBASCO
10 ESTES STREET
IPSWICH, MASSACHUSETTS
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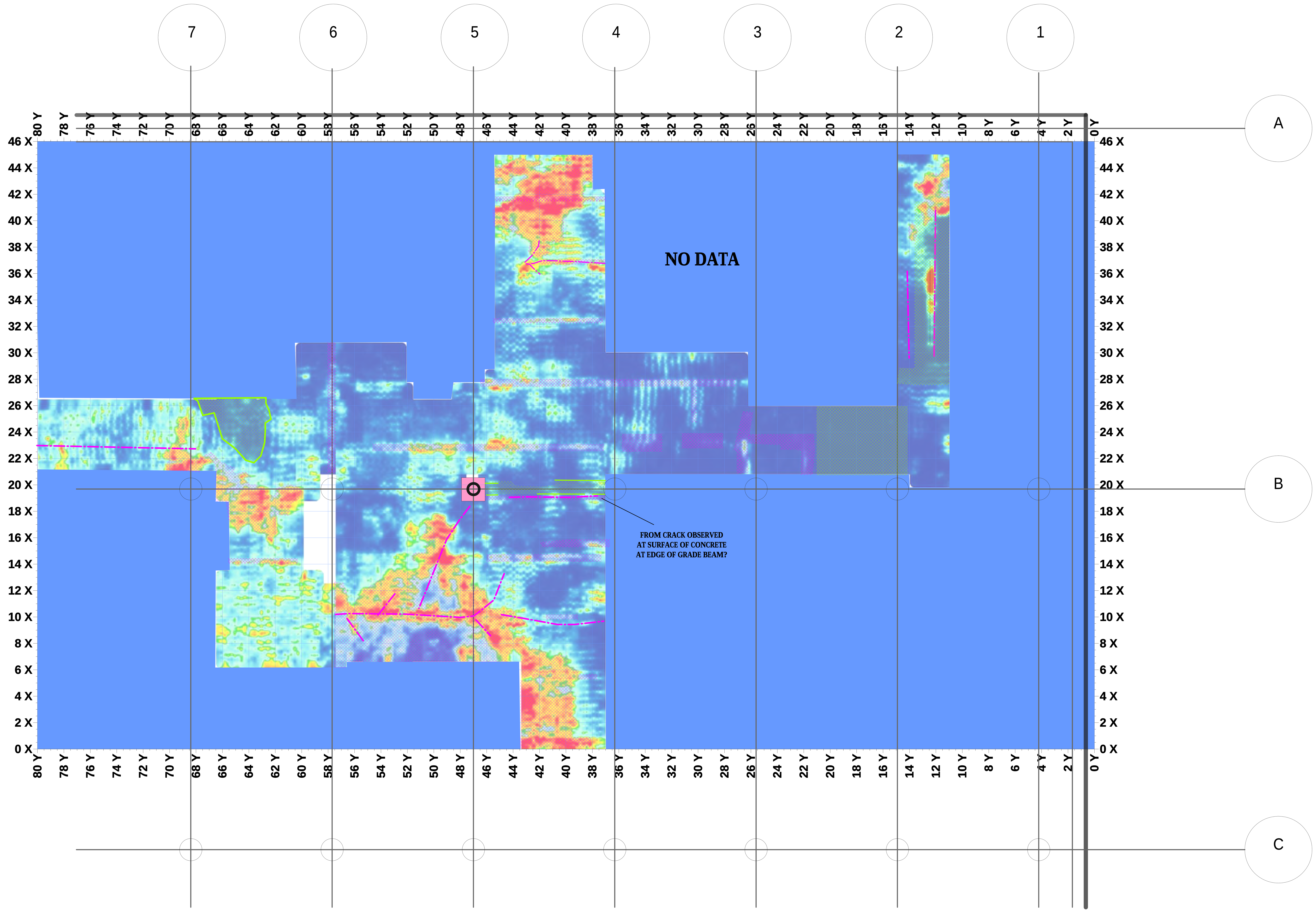

LEGEND

Pedestal

Location of Column 5B

1500 MHz DEPTH SLICE LEGEND

Zone of High-Amplitude GPR Reflections that could indicate an excavation or voiding, boulder, localized area of increased moisture

Zone of Attenuation that may indicate an area of increased soil and/or ground-water conductivity

DEPTH: 3.9 in. - 4.6 in.

increasing amplitude

low (weak) amplitude

high (strong) amplitude

SCALE: 1 Inch = 4 Feet

4 0 4 Feet

FIGURE 4E
REPRESENTATIVE DEPTH SLICE IMAGE
OF 1500 MHz GPR DATA
FOR VOID DETECTION
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10 ESTES STREET
IPSWICH, MASSACHUSETTS
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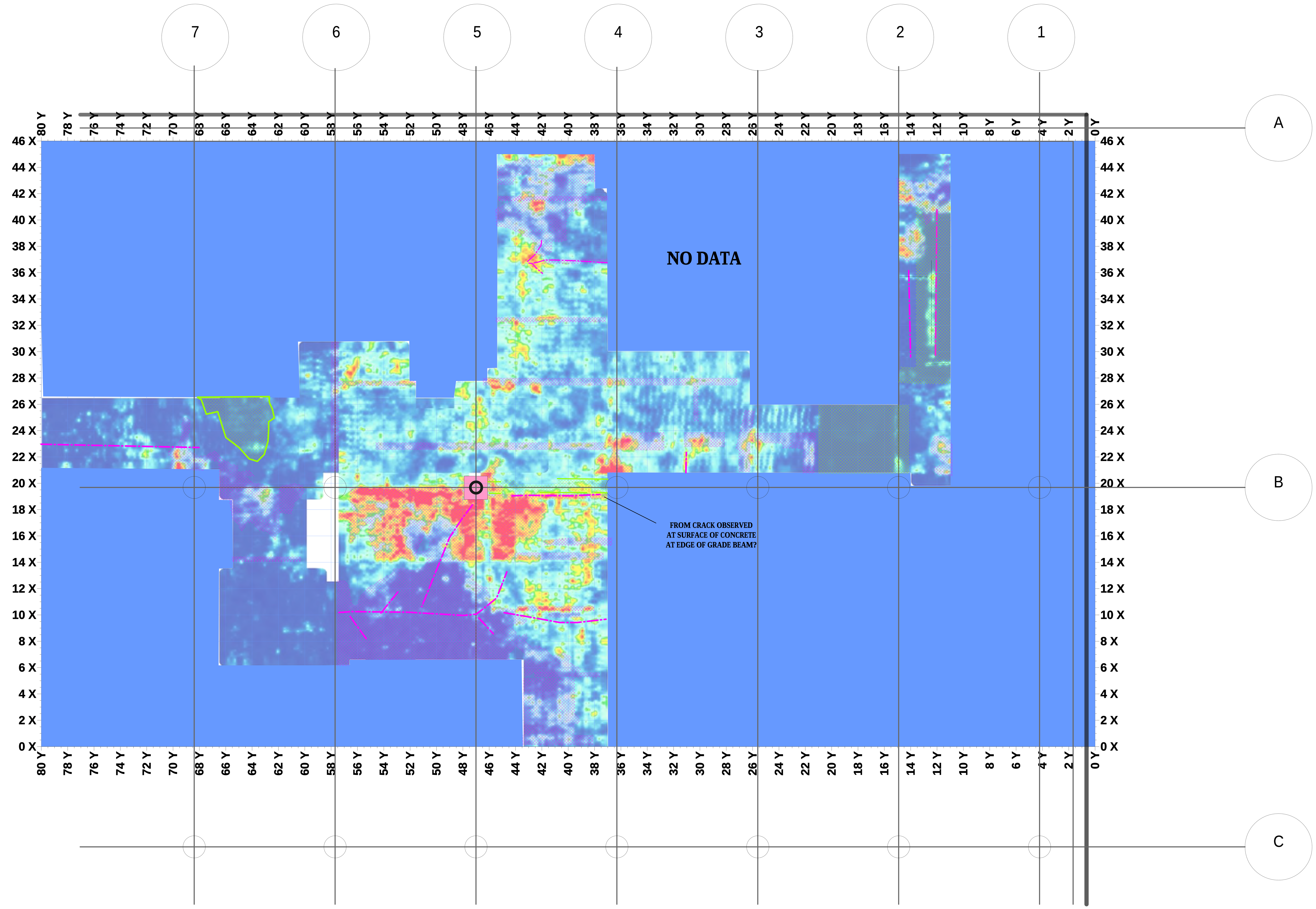

LEGEND

Pedestal

Location of Column 5B

1500 MHz DEPTH SLICE LEGEND

Zone of High-Amplitude GPR Reflections that could indicate an excavation or voiding, boulder, localized area of increased moisture

Zone of Attenuation that may indicate an area of increased soil and/or ground-water conductivity

DEPTH: 5.3 in. - 6.0 in.

increasing amplitude

low (weak) amplitude

high (strong) amplitude

SCALE: 1 Inch = 4 Feet

4 0 4 Feet

FIGURE 4F
REPRESENTATIVE DEPTH SLICE IMAGE
OF 1500 MHz GPR DATA
FOR VOID DETECTION
EBASCO
10 ESTES STREET
IPSWICH, MASSACHUSETTS
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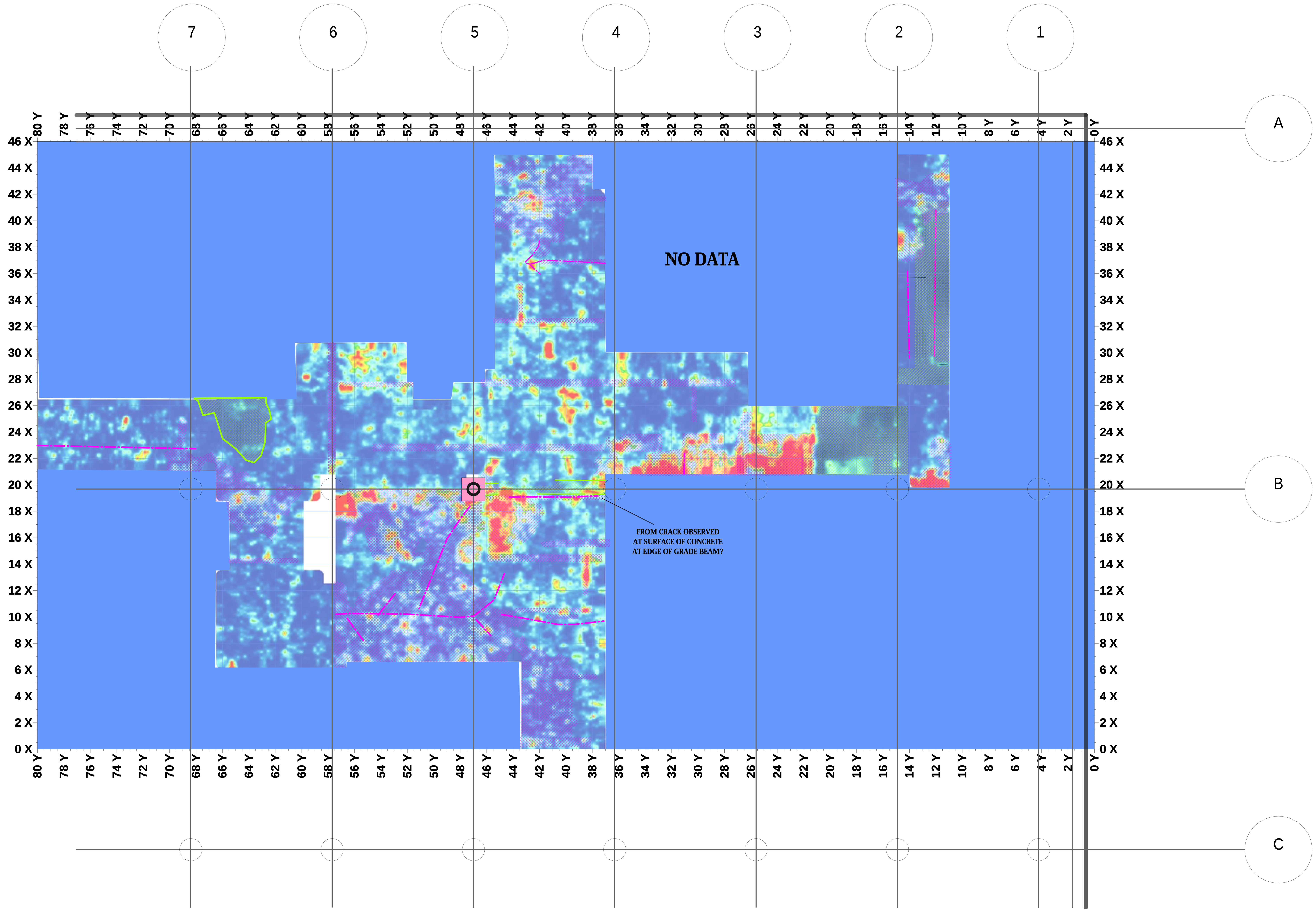

LEGEND

Pedestal

Location of Column 5B

1500 MHz DEPTH SLICE LEGEND

Zone of High-Amplitude GPR Reflections that could indicate an excavation or voiding, boulder, localized area of increased moisture

Zone of Attenuation that may indicate an area of increased soil and/or ground-water conductivity

DEPTH: 6.7 in. - 7.4 in.

increasing amplitude

low (weak) amplitude

high (strong) amplitude

SCALE: 1 Inch = 4 Feet

4 0 4 Feet

FIGURE 4G
REPRESENTATIVE DEPTH SLICE IMAGE
OF 1500 MHz GPR DATA
FOR VOID DETECTION
EBASCO
10 ESTES STREET
IPSWICH, MASSACHUSETTS
Prepared for
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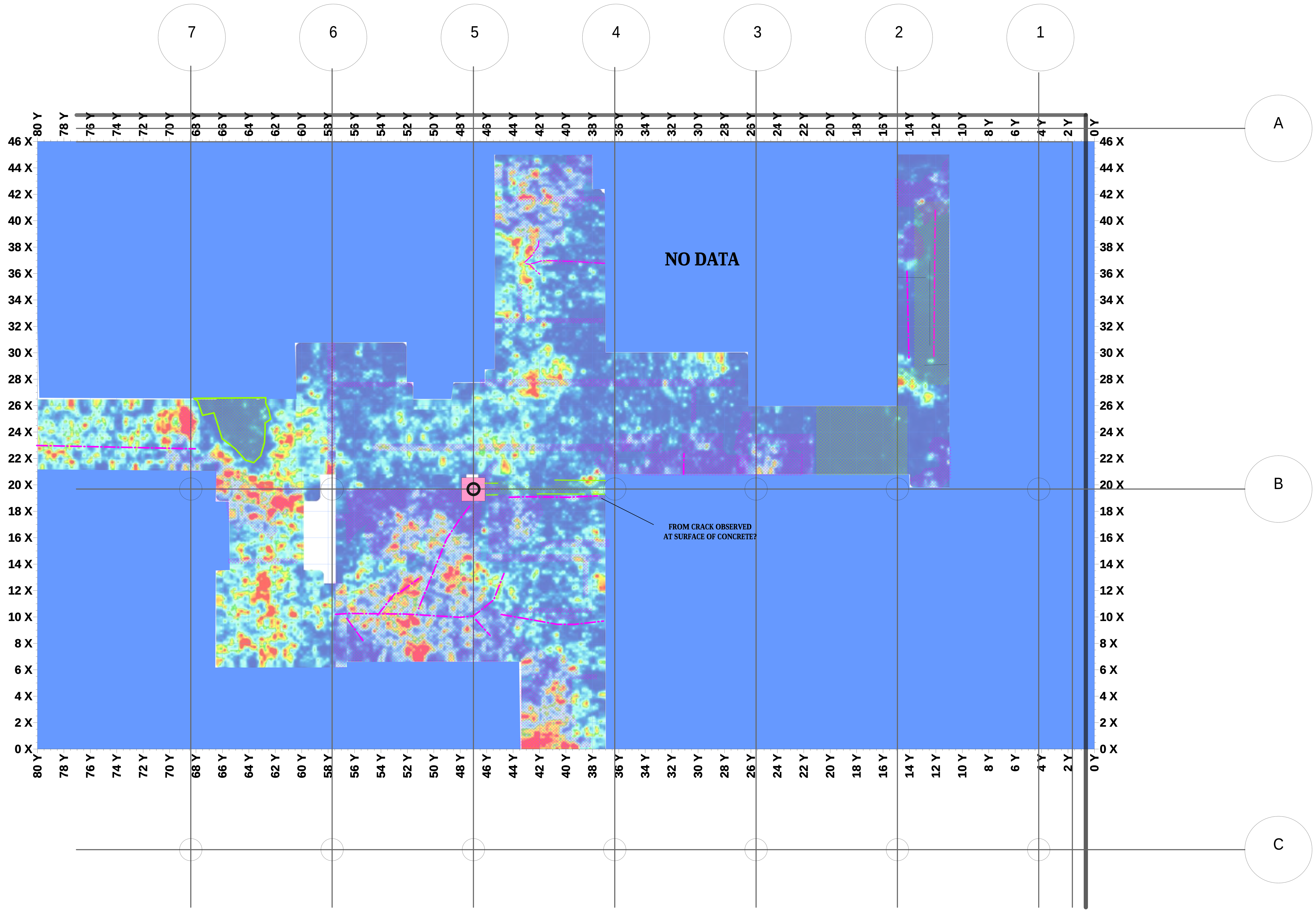

LEGEND

Pedestal

Location of Column 5B

1500 MHz DEPTH SLICE LEGEND

Zone of High-Amplitude GPR Reflections that could indicate an excavation or voiding, boulder, localized area of increased moisture

Zone of Attenuation that may indicate an area of increased soil and/or ground-water conductivity

DEPTH: 8.4 in. - 9.1 in.

increasing amplitude

low (weak) amplitude

high (strong) amplitude

SCALE: 1 Inch = 4 Feet

4 0 4 Feet

FIGURE 4H
REPRESENTATIVE DEPTH SLICE IMAGE OF 1500 MHz GPR DATA FOR VOID DETECTION
EBASCO
10 ESTES STREET
IPSWICH, MASSACHUSETTS
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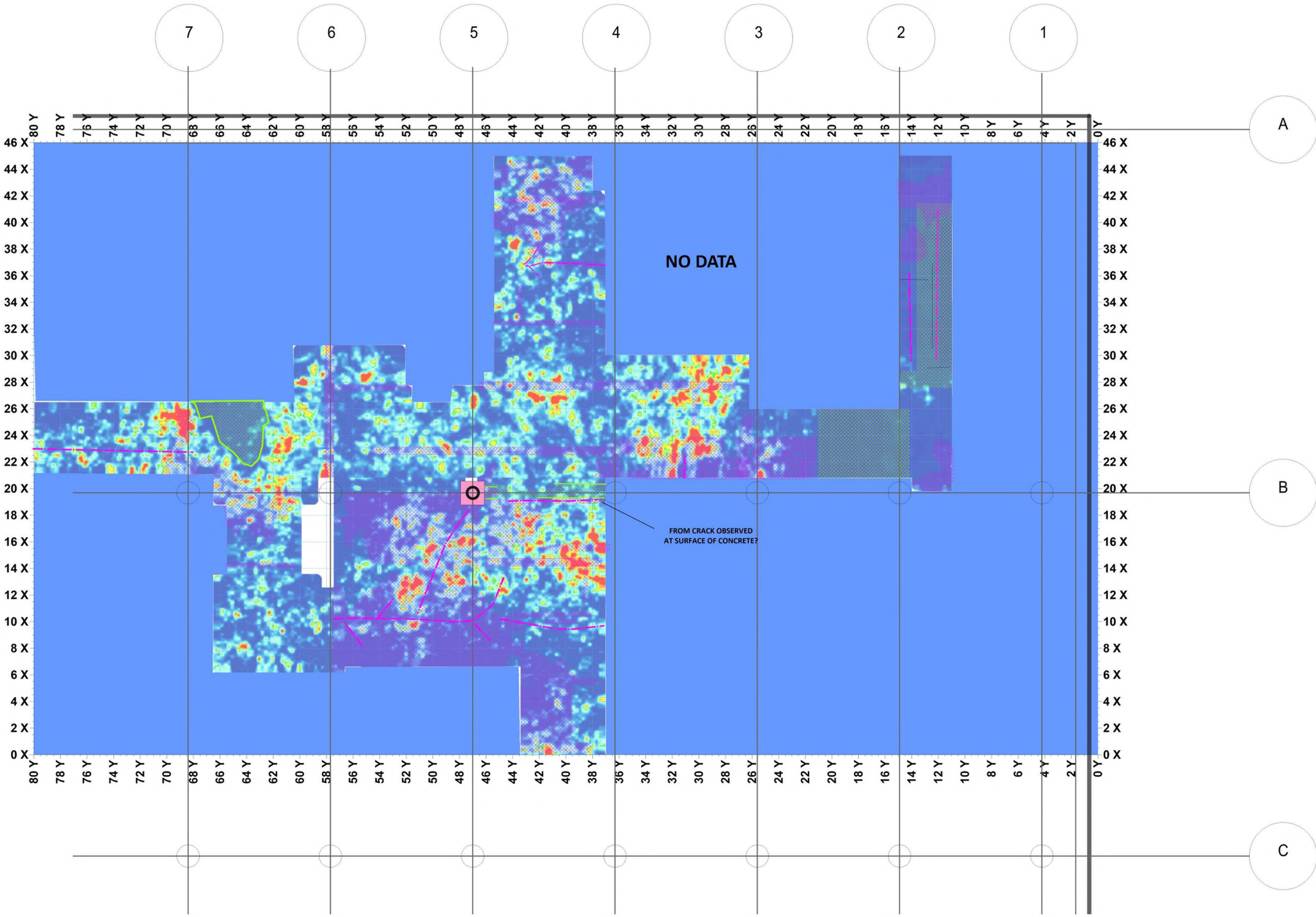

LEGEND

Pedestal

Location of Column 5B

1500 MHz DEPTH SLICE LEGEND

Zone of High-Amplitude GPR Reflections that could indicate an excavation or voiding, boulder, localized area of increased moisture

Zone of Attenuation that may indicate an area of increased soil and/or ground-water conductivity

DEPTH: 9.5 in. - 10.2 in.

increasing amplitude

low (weak) amplitude

high (strong) amplitude

SCALE: 1 Inch = 4 Feet

4 0 4 Feet

FIGURE 4I
REPRESENTATIVE DEPTH SLICE IMAGE
OF 1500 MHz GPR DATA
FOR VOID DETECTION
EBASCO
10 ESTES STREET
IPSWICH, MASSACHUSETTS
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LEGEND

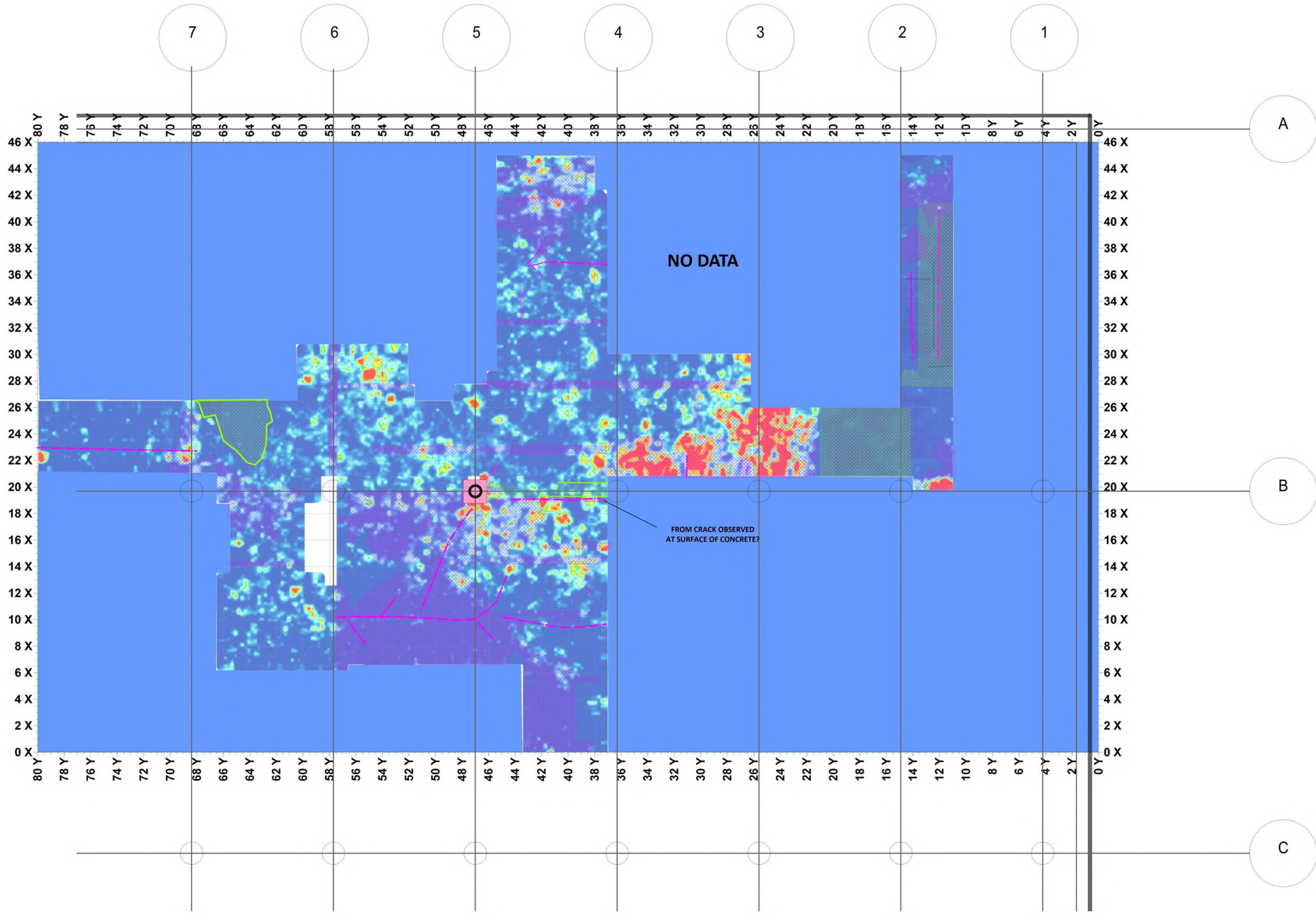
Pedestal

Location of Column 5B

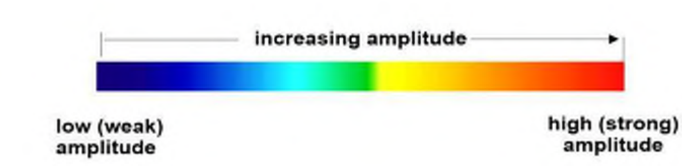
1500 MHz DEPTH SLICE LEGEND

Zone of High-Amplitude GPR Reflections that could indicate an excavation or voiding, boulder, localized area of increased moisture

Zone of Attenuation that may indicate an area of increased soil and/or ground-water conductivity



DEPTH: 11.3 in. - 11.9 in.



SCALE: 1 Inch = 4 Feet

4 0 4 Feet

FIGURE 4J
REPRESENTATIVE DEPTH SLICE IMAGE
OF 1500 MHz GPR DATA
FOR VOID DETECTION
EBASCO
10 ESTES STREET
IPSWICH, MASSACHUSETTS
Prepared for
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LEGEND

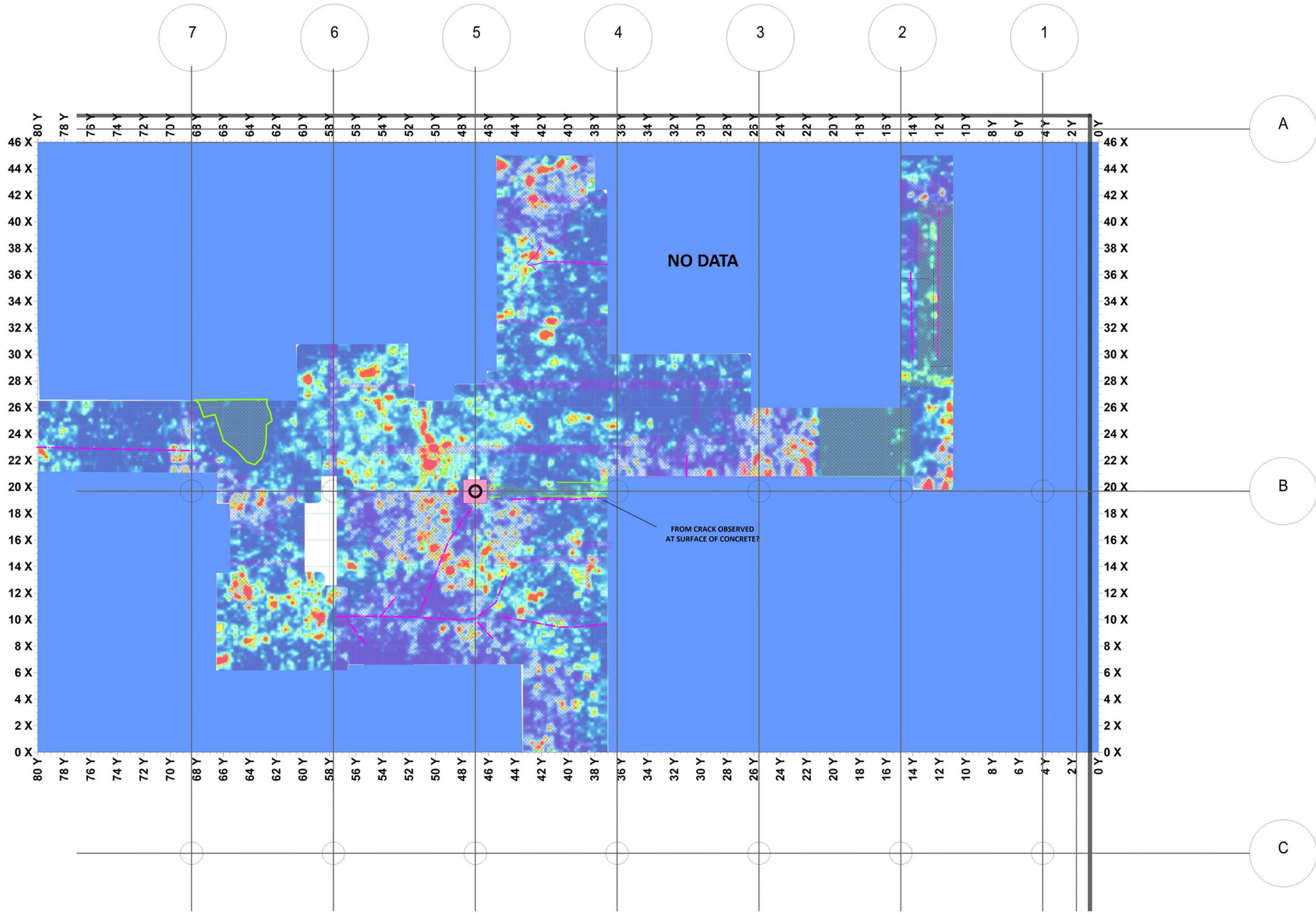
Pedestal

Location of Column 5B

1500 MHz DEPTH SLICE LEGEND

Zone of High-Amplitude GPR Reflections that could indicate an excavation or voiding, boulder, localized area of increased moisture

Zone of Attenuation that may indicate an area of increased soil and/or ground-water conductivity



DEPTH: 13.4 in. - 14.1 in.

increasing amplitude

low (weak) amplitude

high (strong) amplitude

SCALE: 1 Inch = 4 Feet

4 0 4 Feet

FIGURE 4K
REPRESENTATIVE DEPTH SLICE IMAGE
OF 1500 MHz GPR DATA
FOR VOID DETECTION
EBASCO
10 ESTES STREET
IPSWICH, MASSACHUSETTS
Prepared for
SIMPSON GUMPERTZ & HEGER
JUNE 2021

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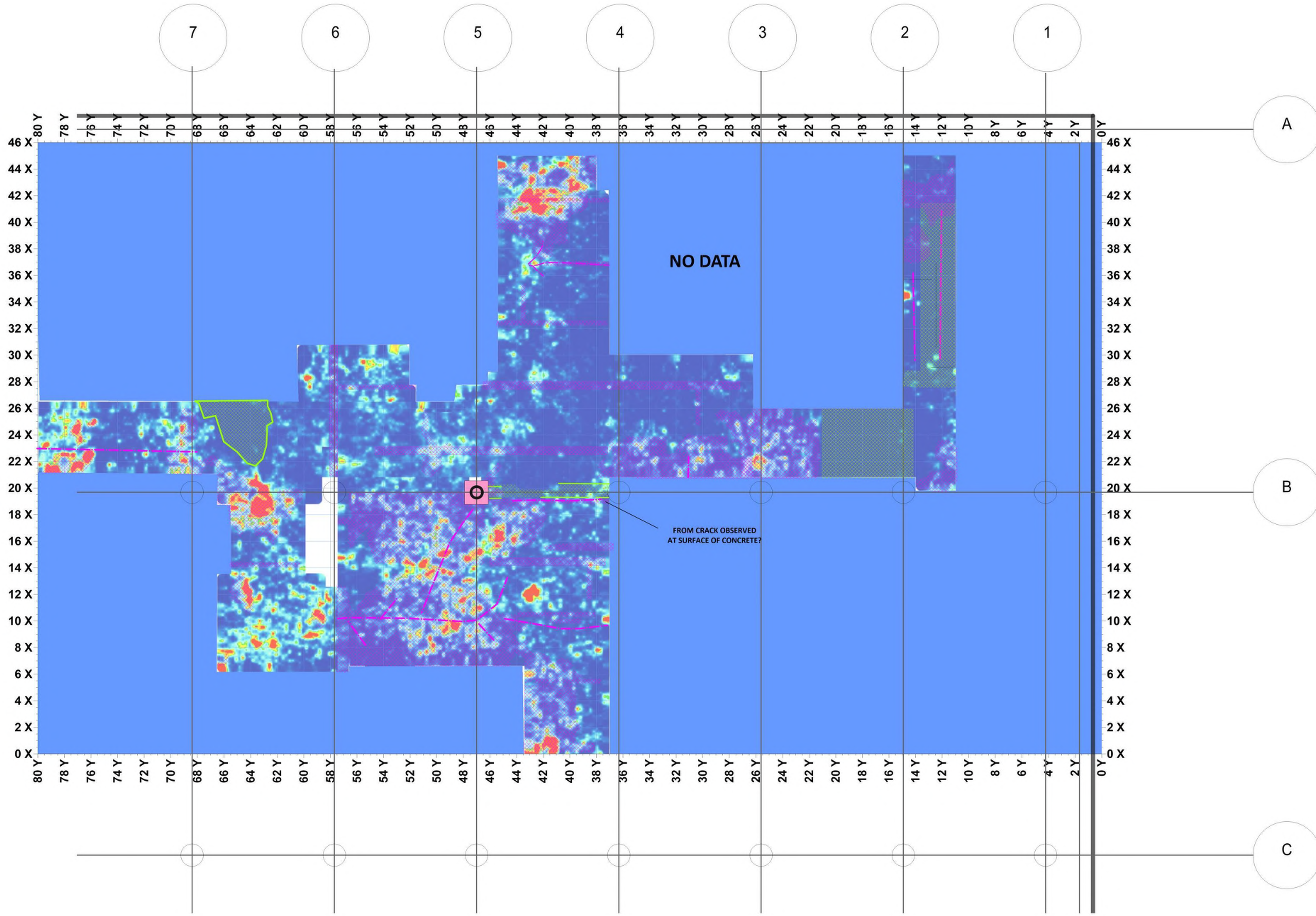

LEGEND

Pedestal

Location of Column 5B

1500 MHz DEPTH SLICE LEGEND

Zone of High-Amplitude GPR Reflections that could indicate an excavation or voiding, boulder, localized area of increased moisture

Zone of Attenuation that may indicate an area of increased soil and/or ground-water conductivity

DEPTH: 15.5 in. - 16.2 in.

increasing amplitude

low (weak) amplitude

high (strong) amplitude

SCALE: 1 Inch = 4 Feet

4 0 4 Feet

FIGURE 4L
REPRESENTATIVE DEPTH SLICE IMAGE
OF 1500 MHz GPR DATA
FOR VOID DETECTION
EBASCO
10 ESTES STREET
IPSWICH, MASSACHUSETTS
Prepared for
SIMPSON GUMPERTZ & HEGER
JUNE 2021

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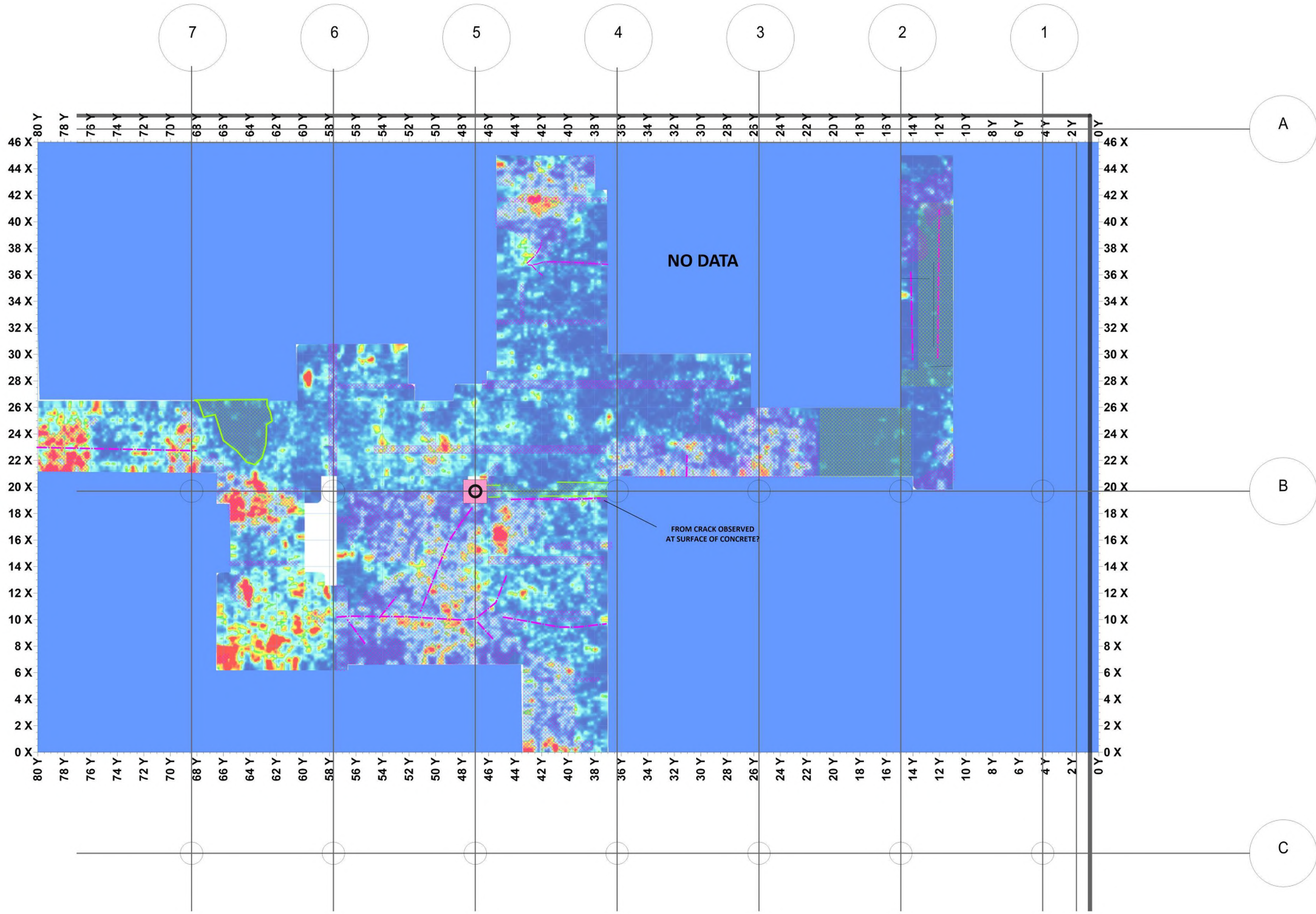

LEGEND

Pedestal

Location of Column 5B

1500 MHz DEPTH SLICE LEGEND

Zone of High-Amplitude GPR Reflections that could indicate an excavation or voiding, boulder, localized area of increased moisture

Zone of Attenuation that may indicate an area of increased soil and/or ground-water conductivity

DEPTH: 16.9 in. - 17.6 in.

increasing amplitude

low (weak) amplitude

high (strong) amplitude

SCALE: 1 Inch = 4 Feet

4 0 4 Feet

FIGURE 4M
REPRESENTATIVE DEPTH SLICE IMAGE
OF 1500 MHz GPR DATA
FOR VOID DETECTION
EBASCO
10 ESTES STREET
IPSWICH, MASSACHUSETTS
Prepared for
SIMPSON GUMPERTZ & HEGER
JUNE 2021

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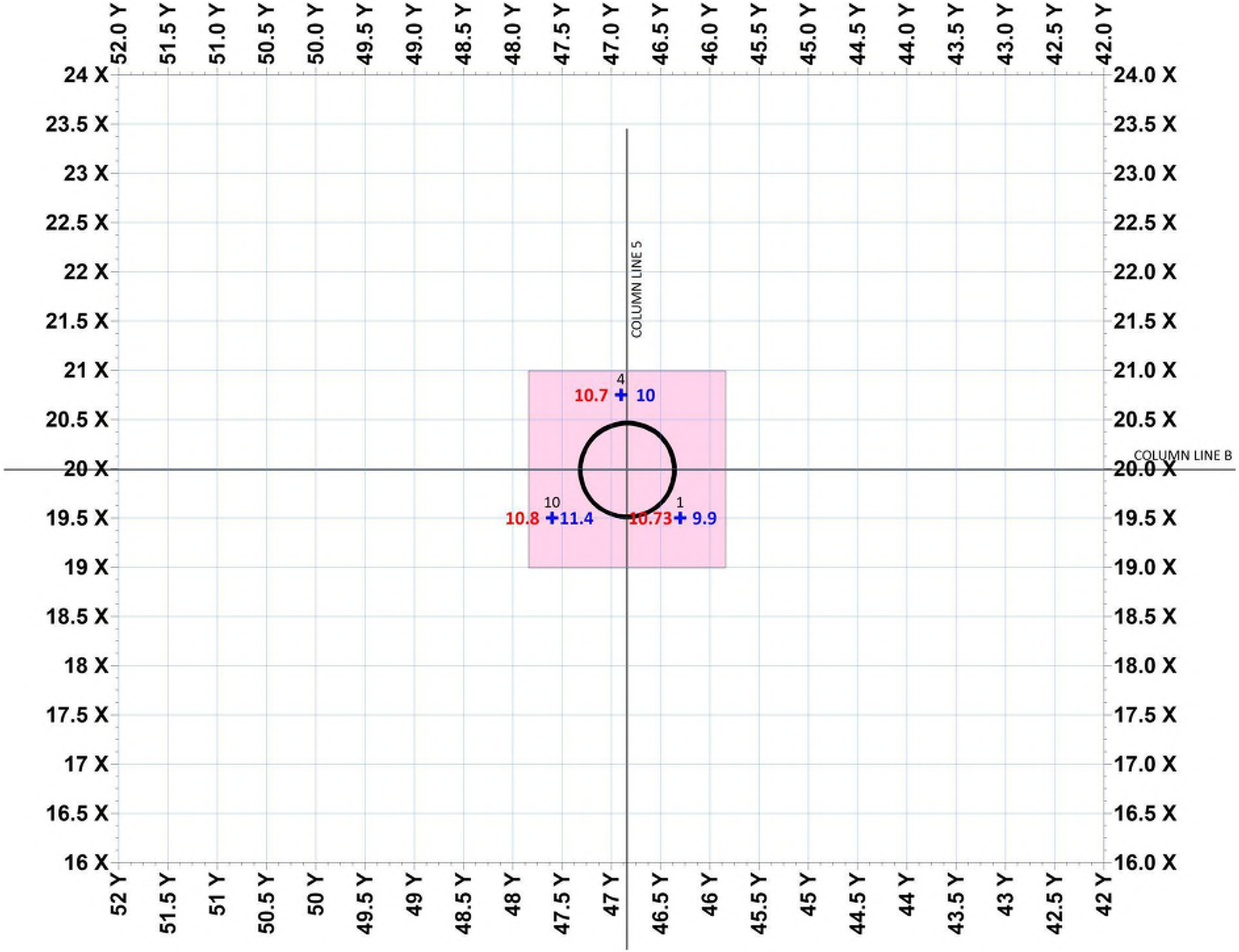

LEGEND

- Pedistal
- Location of Column 5B
- Location of Impulse Echo Measurement (not shown)
- Location of SEIR Measurement (FILE Number in black above location)
- 10.7 Depth to Bottom of Wooden Timber Pile from SEIR Time-Domain Amplitude Plot
- 10.7 Depth to Bottom of Wooden Timber Pile from SEIR Frequency-Domain IR Test

NOTE: Measurements Taken with Dytrans Instrumented Hammer using a 1,000 Hz Accelerometer.

Bell-Response in Time-Domain Indicates Wave Impeding onto a Higher Acoustic Material at Bottom of Wooden Timber. This is likely to be Glacial Till, rather than Concrete

SCALE: 1 Inch = 1 Foot

1 0 1 Foot

FIGURE 5
DEPTH TO BOTTOM OF WOODEN TIMBER PILE USING SEIR
EBASCO
10 ESTES STREET
IPSWICH, MASSACHUSETTS
Prepared for
SIMPSON GUMPERTZ & HEGER
JUNE 2021

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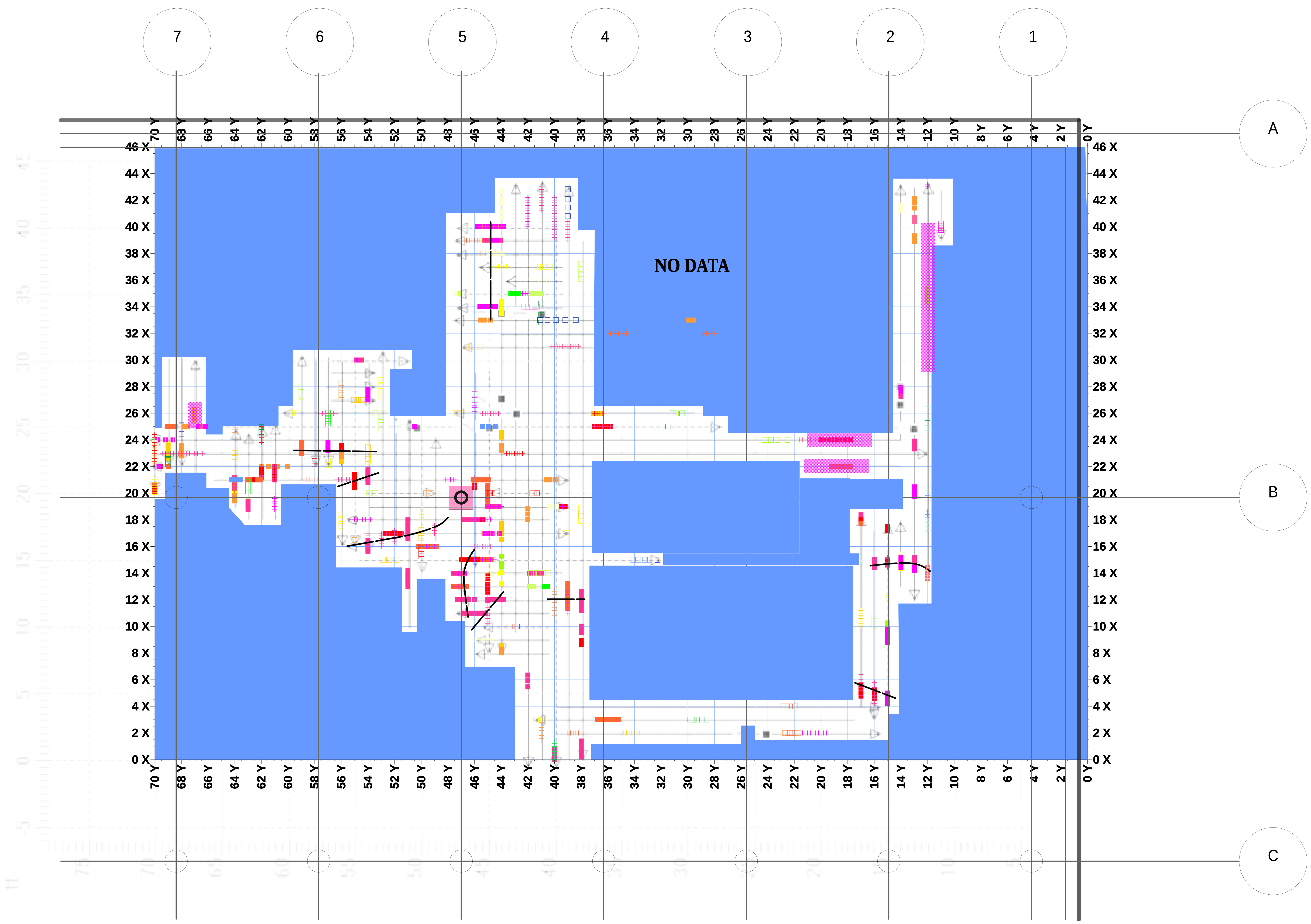

LEGEND

Pedistal Around Column 5B

Column 5B, Used for Sonic Echo/ Impulse Response Tests

Location and Orientation of GPR Traverse - 500 MHz Antenna

GPR VISUAL INSPECTION LEGEND

Area of Deep Concrete

High-Amplitude Hyperbolic Reflector (possible reflection from utility, boulder, and/or tentative wooden timber pile, if the latter is repetitive (from GPR Visual Inspection); Depth (ft.) as noted BELOW

Flat GPR Reflector (reflection possibly from buried slab, multiple closely spaced utilities or conduits, or tentatively from top of wooden pile)

Interpreted Trend of Possible Conduits or Utility (from the alignment of similar reflectors along adjacent lines)

Depth (ft.)

0.00 to 1.00
1.00 to 1.25
1.25 to 1.50
1.50 to 1.75
1.75 to 2.00
2.00 to 2.25
2.25 to 2.50
2.50 to 2.75
2.75 to 3.00
3.00 to 3.25
3.25 to 3.50
3.50 to 3.75
3.75 to 4.00
4.00 to 4.25
4.25 to 4.50
4.50 to 4.75
4.75 to 5.00
5.00 to 5.50
5.50 to 6.00
6.00 to 6.50
6.50 to 7.00
7.00 to 7.50
7.50 to 8.00
8.00 to 9.00

SCALE: 1 Inch = 4 Feet

4 0 4 Feet

FIGURE 6
GPR VISUAL INSPECTION OF
500 MHz DATA FOR TIMBER PILES
EBSCO
10 ESTES STREET
IPSWICH, MASSACHUSETTS

Prepared for
SIMPSON GUMPERTZ & HEGER
JUNE 2021

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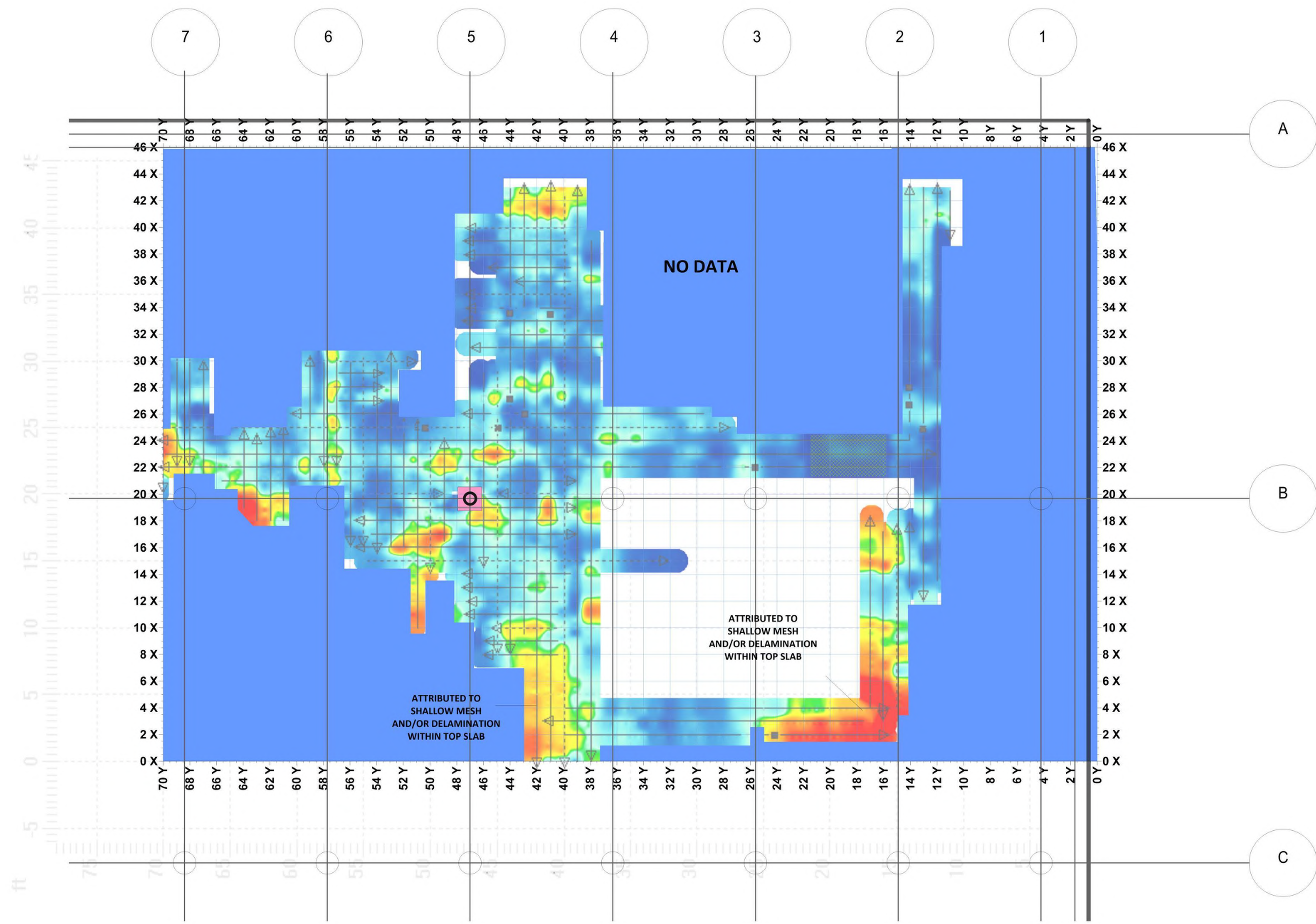

LEGEND

Pedestal Around Column 5B

Column 5B, Used for Sonic Echo/ Impulse Response Tests

Location and Orientation of GPR Traverse - 500 MHz Antenna

1500 MHz DEPTH SLICE LEGEND

Linear, High-Amplitude Reflection that may indicate rebar or seam in Mesh, or Conduit; Depth (ft.) as indicated on slice at which lineation first appears

Zone of High-Amplitude GPR Reflections that could indicate an excavation, voiding, boulder, or cross-sectional reflection from a vertically oriented timber pile. High-amplitude reflections can also indicate a localized area of increased moisture

Zone of Attenuation that may indicate an area of thick concrete, increased soil moisture, and/or ground-water conductivity

DEPTH: 0.1 ft. - 0.4 ft.

increasing amplitude

low (weak) amplitude

high (strong) amplitude

SCALE: 1 Inch = 4 Feet

4 0 4 Feet

FIGURE 7A
REPRESENTATIVE DEPTH SLICE IMAGE OF 500 MHz GPR DATA TO IDENTIFY TOP OF TIMBER PILES
EBSCO
10 ESTES STREET
IPSWICH, MASSACHUSETTS
Prepared for
SIMPSON GUMPERTZ & HEGER
JUNE 2021

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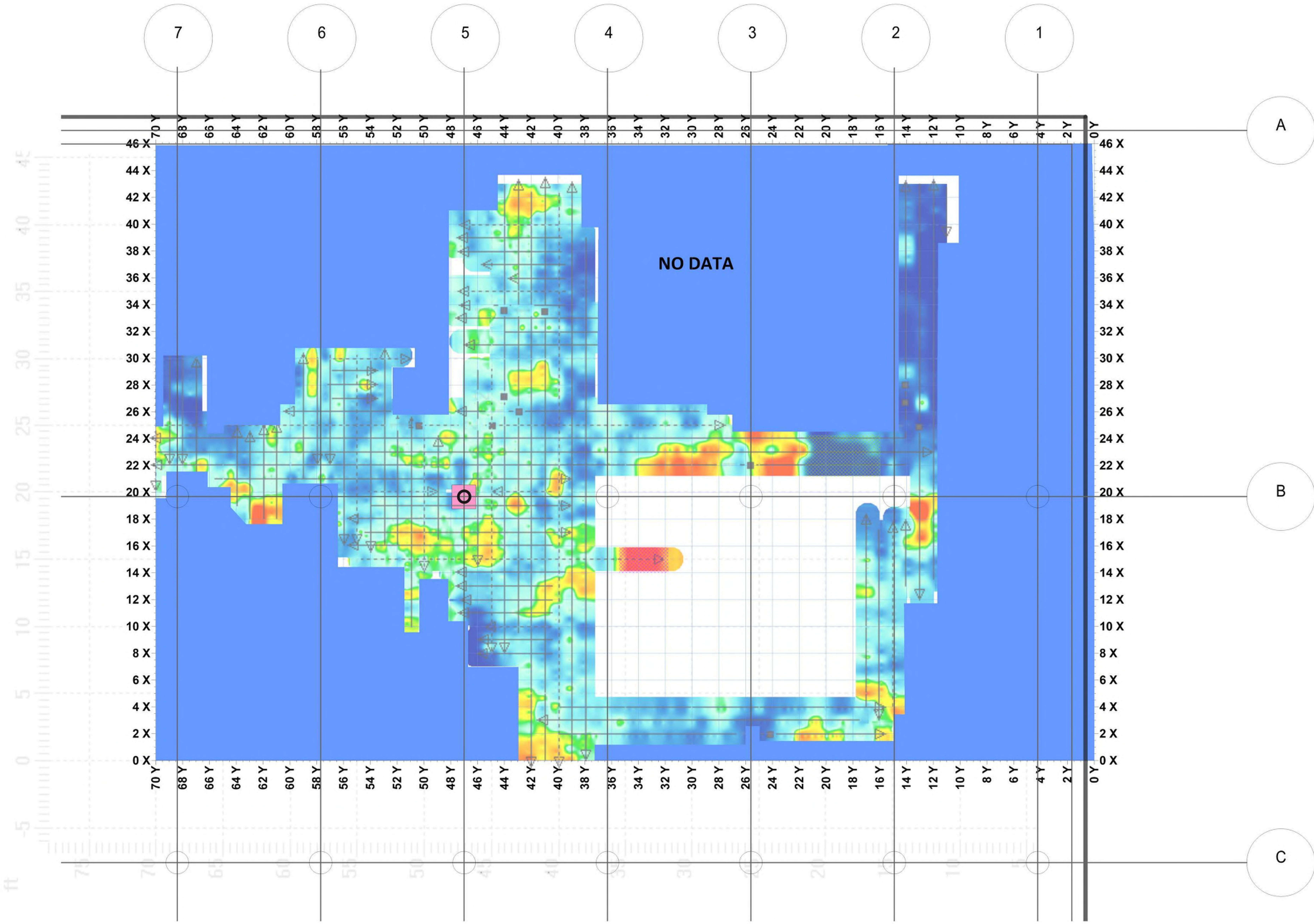

LEGEND

Pedestal Around Column 5B

Column 5B, Used for Sonic Echo/ Impulse Response Tests

Location and Orientation of GPR Traverse - 500 MHz Antenna

<u>1500 MHz DEPTH SLICE LEGEND</u>

Linear, High-Amplitude Reflection that may indicate rebar or seam in Mesh, or Conduit; Depth (ft.) as indicated on slice at which lineation first appears

Zone of High-Amplitude GPR Reflections that could indicate an excavation, voiding, boulder, or cross-sectional reflection from a vertically oriented timber pile. High-amplitude reflections can also indicate a localized area of increased moisture

Zone of Attenuation that may indicate an area of thick concrete, increased soil moisture, and/or ground-water conductivity

DEPTH: 0.5 ft. - 0.8 ft.

increasing amplitude

low (weak) amplitude

high (strong) amplitude

SCALE: 1 Inch = 4 Feet

4 0 4 Feet

FIGURE 7B
REPRESENTATIVE DEPTH SLICE IMAGE OF 500 MHz GPR DATA TO IDENTIFY TOP OF TIMBER PILES
EBSCO
10 ESTES STREET
IPSWICH, MASSACHUSETTS
Prepared for
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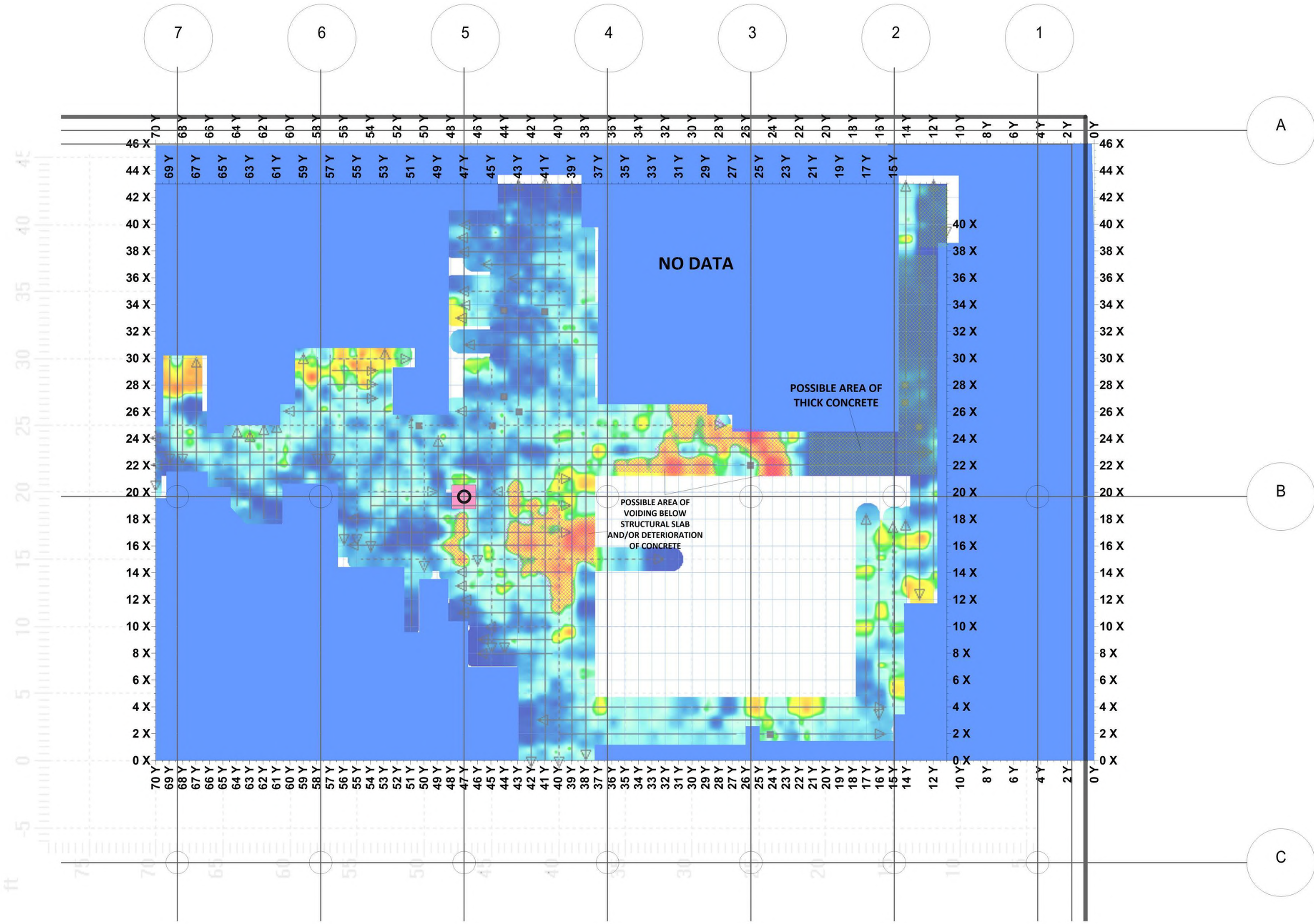

LEGEND

Pedestal Around Column 5B

Column 5B, Used for Sonic Echo/ Impulse Response Tests

Location and Orientation of GPR Traverse - 500 MHz Antenna

1500 MHz DEPTH SLICE LEGEND

Linear, High-Amplitude Reflection that may indicate rebar or seam in Mesh, or Conduit; Depth (ft.) as indicated on slice at which lineation first appears

Zone of High-Amplitude GPR Reflections that could indicate an excavation, voiding, boulder, or cross-sectional reflection from a vertically oriented timber pile. High-amplitude reflections can also indicate a localized area of increased moisture

Zone of Attenuation that may indicate an area of thick concrete, increased soil moisture, and/or ground-water conductivity

DEPTH: 0.9 ft. - 1.2 ft.

increasing amplitude

low (weak) amplitude

high (strong) amplitude

SCALE: 1 Inch = 4 Feet

4 0 4 Feet

FIGURE 7C
REPRESENTATIVE DEPTH SLICE IMAGE OF 500 MHz GPR DATA
TO IDENTIFY TOP OF TIMBER PILES
EBSCO
10 ESTES STREET
IPSWICH, MASSACHUSETTS
Prepared for
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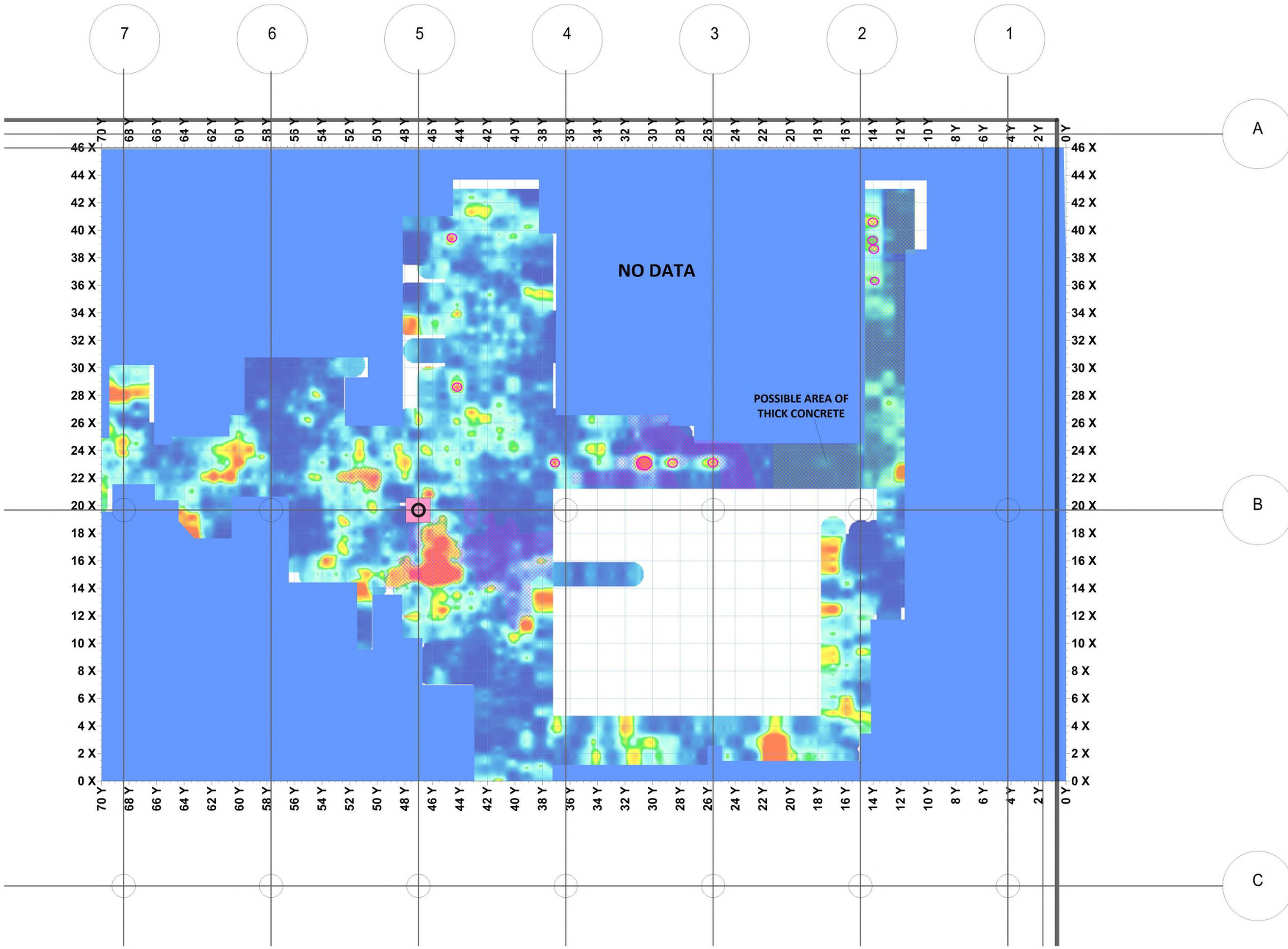

LEGEND

- Pedestal Around Column 5B
- Column 5B, Used for Sonic Echo/ Impulse Response Tests
- Location and Orientation of GPR Traverse - 500 MHz Antenna

1500 MHz DEPTH SLICE LEGEND

- Linear, High-Amplitude Reflection that may indicate rebar or seam in Mesh, or Conduit; Depth (ft.) as indicated on slice at which lineation first appears
- Zone of High-Amplitude GPR Reflections that could indicate an excavation, voiding, boulder, or cross-sectional reflection from a vertically oriented timber pile. High-amplitude reflections can also indicate a localized area of increased moisture
- Zone of Attenuation that may indicate an area of thick concrete, increased soil moisture, and/or ground-water conductivity

DEPTH: 1.2 ft. - 1.4 ft.

increasing amplitude

low (weak) amplitude — high (strong) amplitude

SCALE: 1 Inch = 4 Feet

4 0 4 Feet

FIGURE 7D
REPRESENTATIVE DEPTH SLICE IMAGE OF 500 MHz GPR DATA TO IDENTIFY TOP OF TIMBER PILES
EBSCO
10 ESTES STREET
IPSWICH, MASSACHUSETTS
Prepared for
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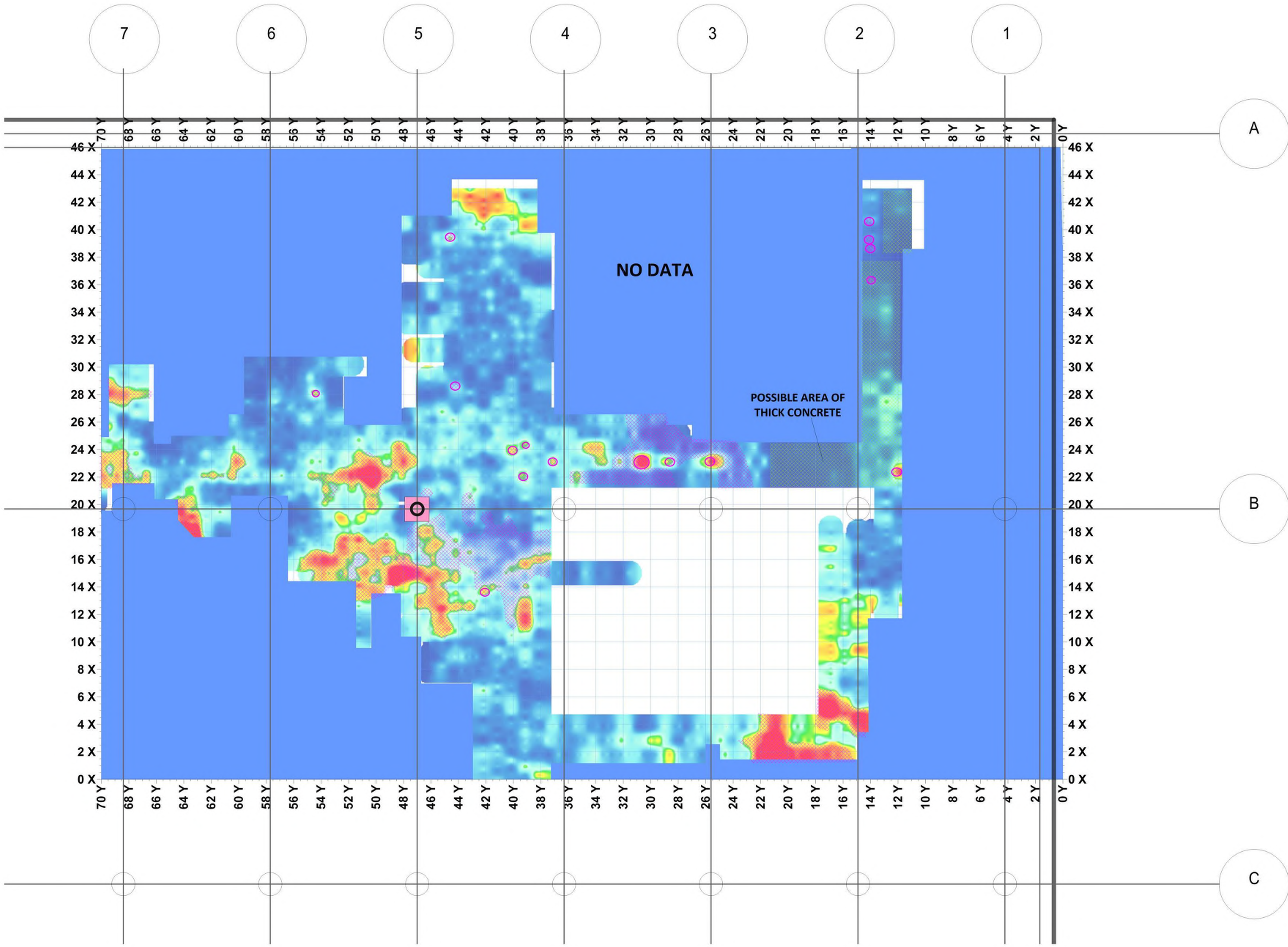

LEGEND

Pedestal Around Column 5B

Column 5B, Used for Sonic Echo/ Impulse Response Tests

Location and Orientation of GPR Traverse - 500 MHz Antenna

1500 MHz DEPTH SLICE LEGEND

Linear, High-Amplitude Reflection that may indicate rebar or seam in Mesh, or Conduit; Depth (ft.) as indicated on slice at which lineation first appears

Zone of High-Amplitude GPR Reflections that could indicate an excavation, voiding, boulder, or cross-sectional reflection from a vertically oriented timber pile. High-amplitude reflections can also indicate a localized area of increased moisture

Zone of Attenuation that may indicate an area of thick concrete, increased soil moisture, and/or ground-water conductivity

DEPTH: 1.3 ft. - 1.6 ft.

increasing amplitude

low (weak) amplitude

high (strong) amplitude

SCALE: 1 Inch = 4 Feet

4 0 4 Feet

FIGURE 7E
REPRESENTATIVE DEPTH SLICE IMAGE OF 500 MHz GPR DATA TO IDENTIFY TOP OF TIMBER PILES
EBSCO
10 ESTES STREET
IPSWICH, MASSACHUSETTS
Prepared for
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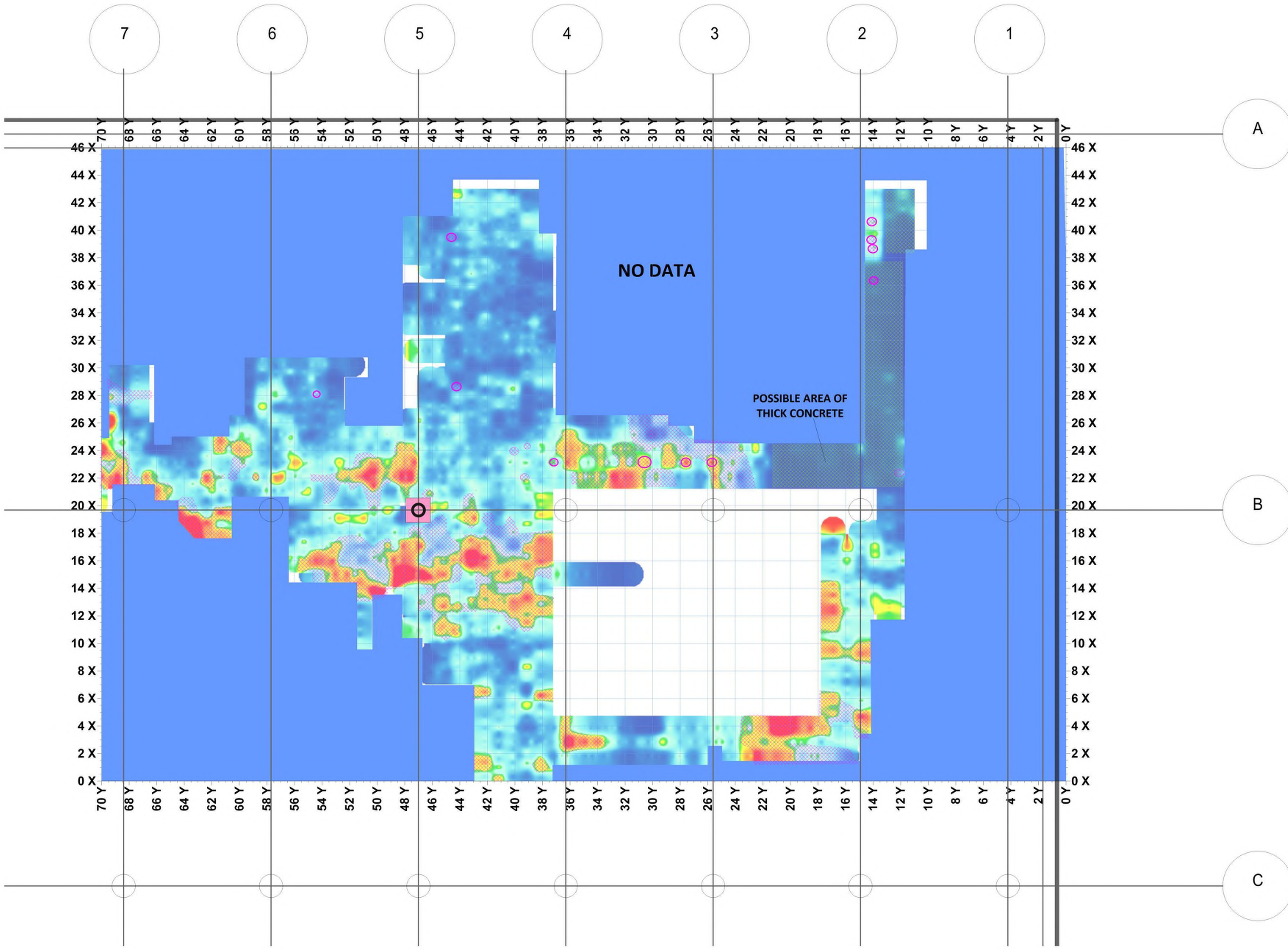

LEGEND

Pedestal Around Column 5B

Column 5B, Used for Sonic Echo/ Impulse Response Tests

Location and Orientation of GPR Traverse - 500 MHz Antenna

1500 MHz DEPTH SLICE LEGEND

Linear, High-Amplitude Reflection that may indicate rebar or seam in Mesh, or Conduit; Depth (ft.) as indicated on slice at which lineation first appears

Zone of High-Amplitude GPR Reflections that could indicate an excavation, voiding, boulder, or cross-sectional reflection from a vertically oriented timber pile. High-amplitude reflections can also indicate a localized area of increased moisture

Zone of Attenuation that may indicate an area of thick concrete, increased soil moisture, and/or ground-water conductivity

DEPTH: 1.6 ft. - 1.8 ft.

increasing amplitude

low (weak) amplitude

high (strong) amplitude

SCALE: 1 Inch = 4 Feet

4 0 4 Feet

FIGURE 7F
REPRESENTATIVE DEPTH SLICE IMAGE OF 500 MHz GPR DATA TO IDENTIFY TOP OF TIMBER PILES
EBSCO
10 ESTES STREET
IPSWICH, MASSACHUSETTS
Prepared for
SIMPSON GUMPERTZ & HEGER
JUNE 2021

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LEGEND
Pedestal Around Column 5B
Column 5B, Used for Sonic Echo/ Impulse Response Tests
Location and Orientation of GPR Traverse - 500 MHz Antenna
1500 MHz DEPTH SLICE LEGEND
Linear, High-Amplitude Reflection that may indicate rebar or seam in Mesh, or Conduit; Depth (ft.) as indicated on slice at which lineation first appears
Zone of High-Amplitude GPR Reflections that could indicate an excavation, voiding, boulder, or cross-sectional reflection from a vertically oriented timber pile. High-amplitude reflections can also indicate a localized area of increased moisture
Zone of Attenuation that may indicate an area of thick concrete, increased soil moisture, and/or ground-water conductivity
DEPTH: 1.8 ft. - 2.1 ft.
increasing amplitude
low (weak) amplitude
high (strong) amplitude
SCALE: 1 Inch = 4 Feet
4 0 4 Feet
NO DATA
POSSIBLE AREA OF THICK CONCRETE
7 6 5 4 3 2 1
A
B
C
FIGURE 7G
REPRESENTATIVE DEPTH SLICE IMAGE OF 500 MHz GPR DATA TO IDENTIFY TOP OF TIMBER PILES
EBSCO
10 ESTES STREET
IPSWICH, MASSACHUSETTS
Prepared for
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JUNE 2021
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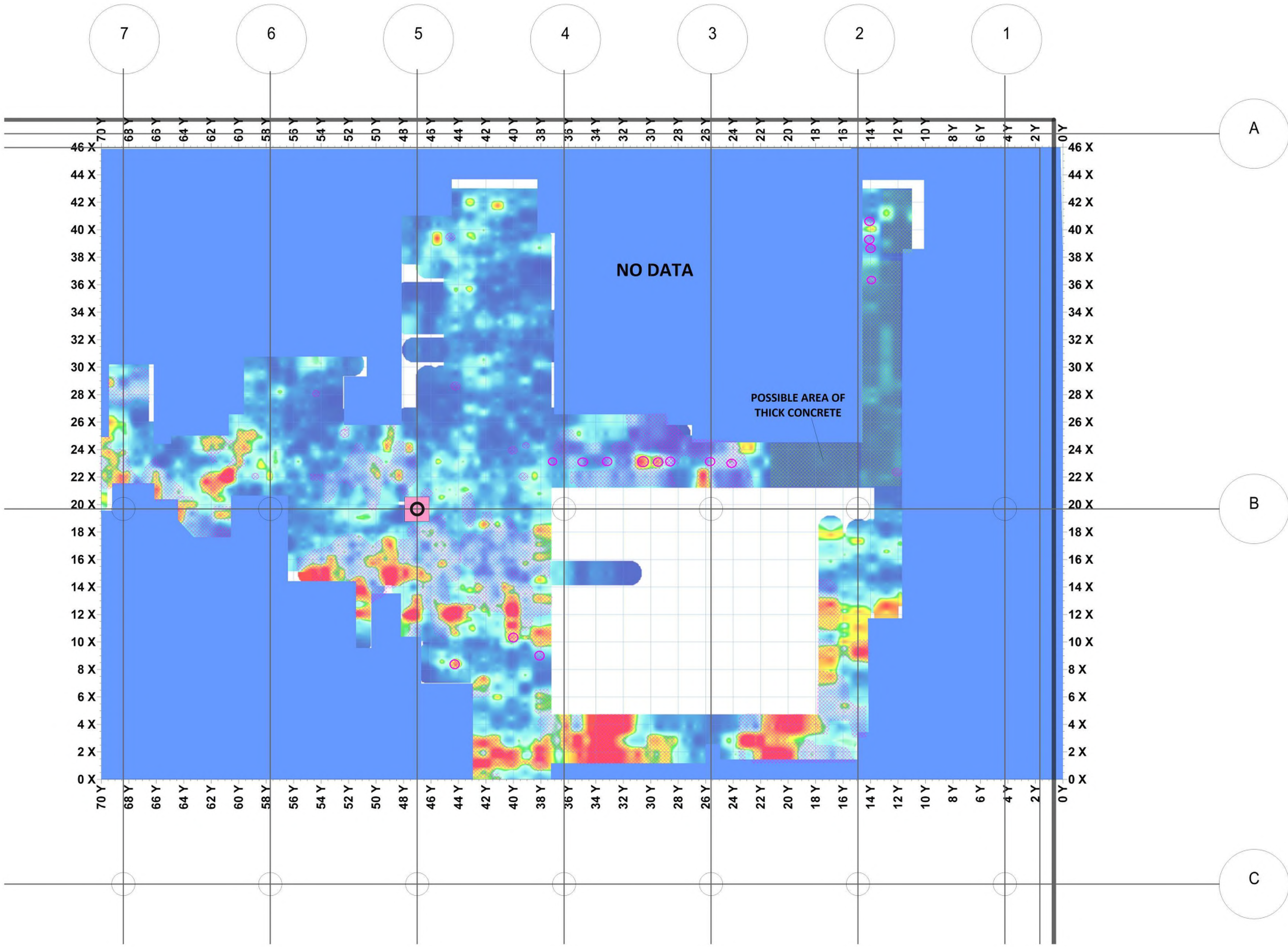

LEGEND

Pedestal Around Column 5B

Column 5B, Used for Sonic Echo/ Impulse Response Tests

Location and Orientation of GPR Traverse - 500 MHz Antenna

1500 MHz DEPTH SLICE LEGEND

Linear, High-Amplitude Reflection that may indicate rebar or seam in Mesh, or Conduit; Depth (ft.) as indicated on slice at which lineation first appears

Zone of High-Amplitude GPR Reflections that could indicate an excavation, voiding, boulder, or cross-sectional reflection from a vertically oriented timber pile. High-amplitude reflections can also indicate a localized area of increased moisture

Zone of Attenuation that may indicate an area of thick concrete, increased soil moisture, and/or ground-water conductivity

DEPTH: 2.2 ft. - 2.5 ft.

increasing amplitude

low (weak) amplitude

high (strong) amplitude

SCALE: 1 Inch = 4 Feet

4 0 4 Feet

FIGURE 7H
REPRESENTATIVE DEPTH SLICE IMAGE OF 500 MHz GPR DATA TO IDENTIFY TOP OF TIMBER PILES
EBSCO
10 ESTES STREET
IPSWICH, MASSACHUSETTS
Prepared for
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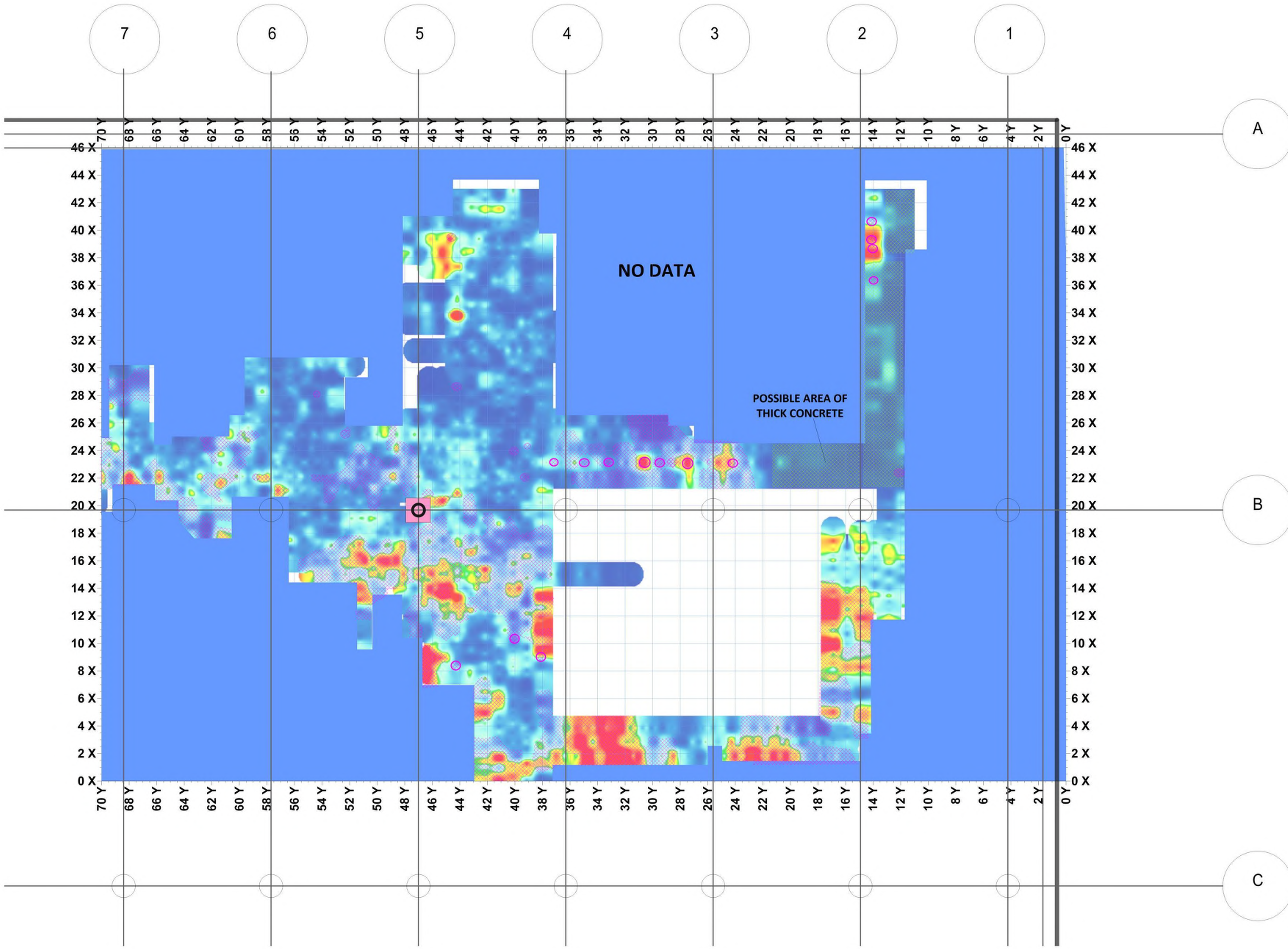

LEGEND

Pedestal Around Column 5B

Column 5B, Used for Sonic Echo/ Impulse Response Tests

Location and Orientation of GPR Traverse - 500 MHz Antenna

1500 MHz DEPTH SLICE LEGEND

Linear, High-Amplitude Reflection that may indicate rebar or seam in Mesh, or Conduit; Depth (ft.) as indicated on slice at which lineation first appears

Zone of High-Amplitude GPR Reflections that could indicate an excavation, voiding, boulder, or cross-sectional reflection from a vertically oriented timber pile. High-amplitude reflections can also indicate a localized area of increased moisture

Zone of Attenuation that may indicate an area of thick concrete, increased soil moisture, and/or ground-water conductivity

DEPTH: 2.6 ft. - 2.9 ft.

increasing amplitude

low (weak) amplitude

high (strong) amplitude

SCALE: 1 Inch = 4 Feet

4 0 4 Feet

FIGURE 7I
REPRESENTATIVE DEPTH SLICE IMAGE OF 500 MHz GPR DATA
TO IDENTIFY TOP OF TIMBER PILES
EBSCO
10 ESTES STREET
IPSWICH, MASSACHUSETTS
Prepared for
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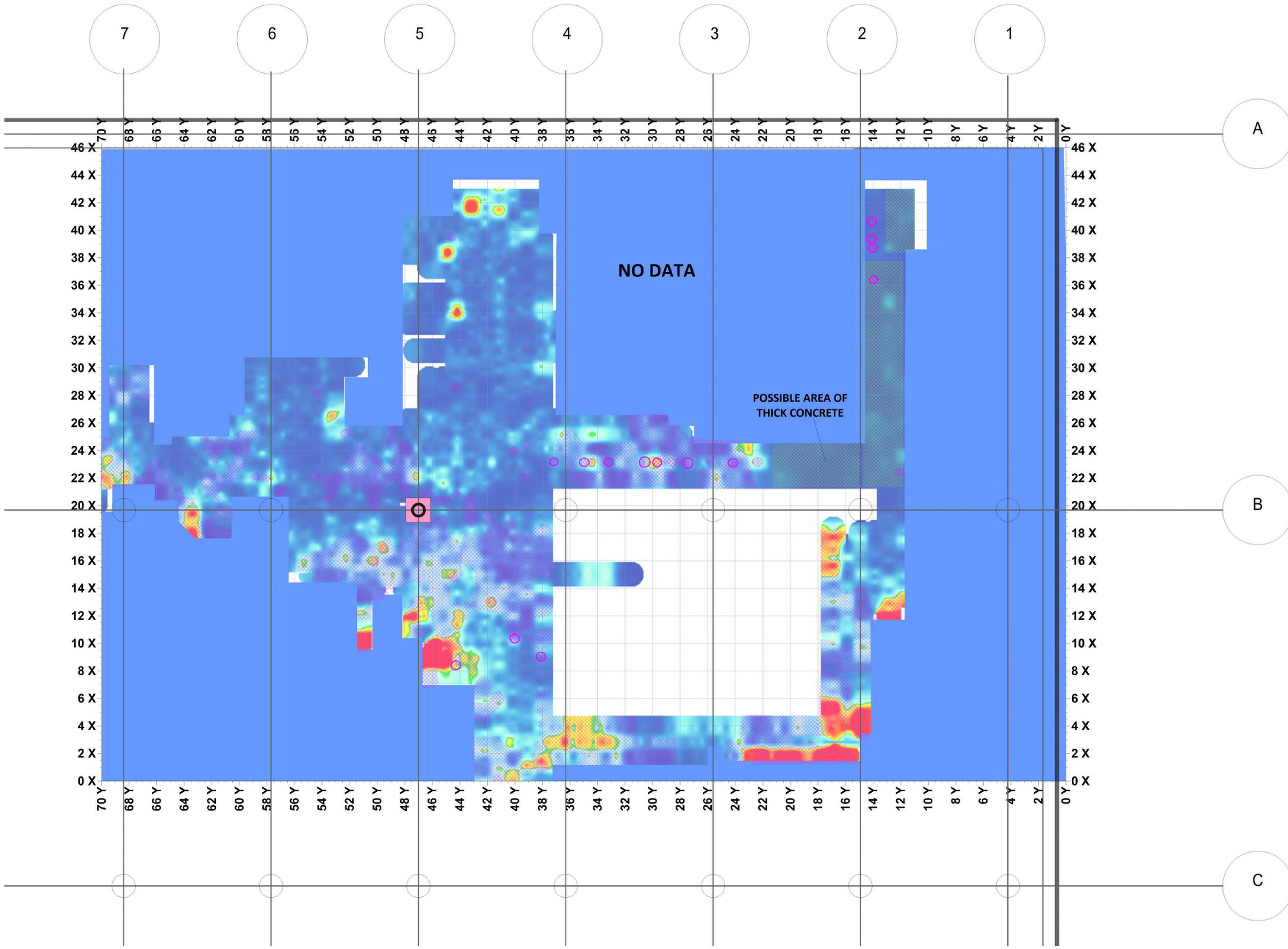

LEGEND

Pedestal Around Column 5B

Column 5B, Used for Sonic Echo/ Impulse Response Tests

Location and Orientation of GPR Traverse - 500 MHz Antenna

1500 MHz DEPTH SLICE LEGEND

Linear, High-Amplitude Reflection that may indicate rebar or seam in Mesh, or Conduit; Depth (ft.) as indicated on slice at which lineation first appears

Zone of High-Amplitude GPR Reflections that could indicate an excavation, voiding, boulder, or cross-sectional reflection from a vertically oriented timber pile. High-amplitude reflections can also indicate a localized area of increased moisture

Zone of Attenuation that may indicate an area of thick concrete, increased soil moisture, and/or ground-water conductivity

DEPTH: 3.1 ft. - 3.4 ft.

increasing amplitude

low (weak) amplitude

high (strong) amplitude

SCALE: 1 Inch = 4 Feet

4 0 4 Feet

FIGURE 7J
REPRESENTATIVE DEPTH SLICE IMAGE OF 500 MHz GPR DATA TO IDENTIFY TOP OF TIMBER PILES
EBSCO
10 ESTES STREET
IPSWICH, MASSACHUSETTS
Prepared for
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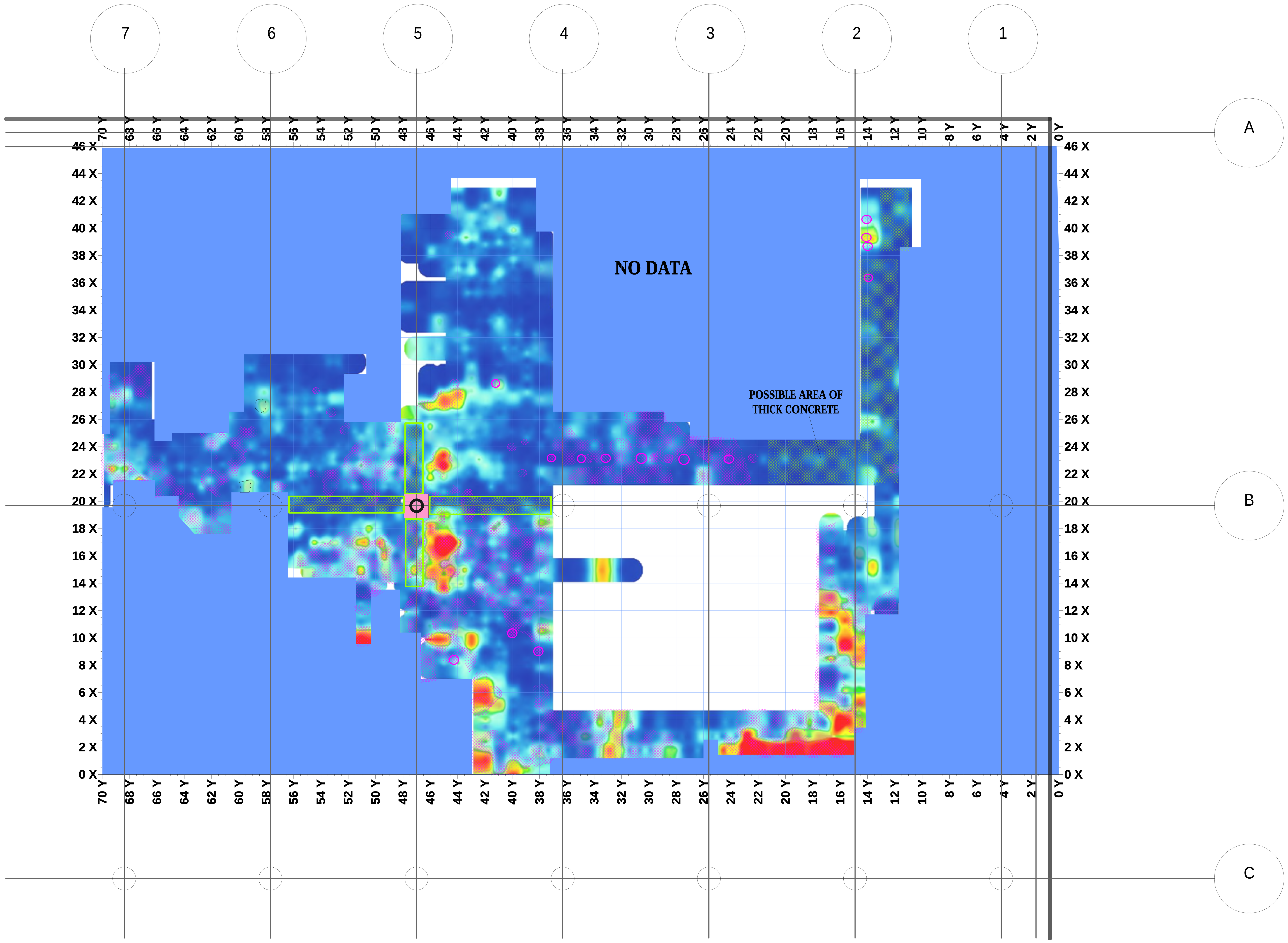

LEGEND

Pedestal Around Column 5B

Column 5B, Used for Sonic Echo/ Impulse Response Tests

Location and Orientation of GPR Traverse - 500 MHz Antenna

1500 MHz DEPTH SLICE LEGEND

Linear, High-Amplitude Reflection that may indicate rebar or seam in Mesh, or Conduit; Depth (ft.) as indicated on slice at which lineation first appears

Zone of High-Amplitude GPR Reflections that could indicate an excavation, voiding, boulder, or cross-sectional reflection from a vertically oriented timber pile. High-amplitude reflections can also indicate a localized area of increased moisture

Zone of Attenuation that may indicate an area of thick concrete, increased soil moisture, and/or ground-water conductivity

DEPTH: 3.8 ft. - 4.1 ft.

increasing amplitude

low (weak) amplitude

high (strong) amplitude

SCALE: 1 Inch = 4 Feet

4 0 4 Feet

FIGURE 7K
REPRESENTATIVE DEPTH SLICE IMAGE OF 500 MHz GPR DATA
TO IDENTIFY TOP OF TIMBER PILES
EBSCO
10 ESTES STREET
IPSWICH, MASSACHUSETTS
Prepared for
SIMPSON GUMPERTZ & HEGER
JUNE 2021

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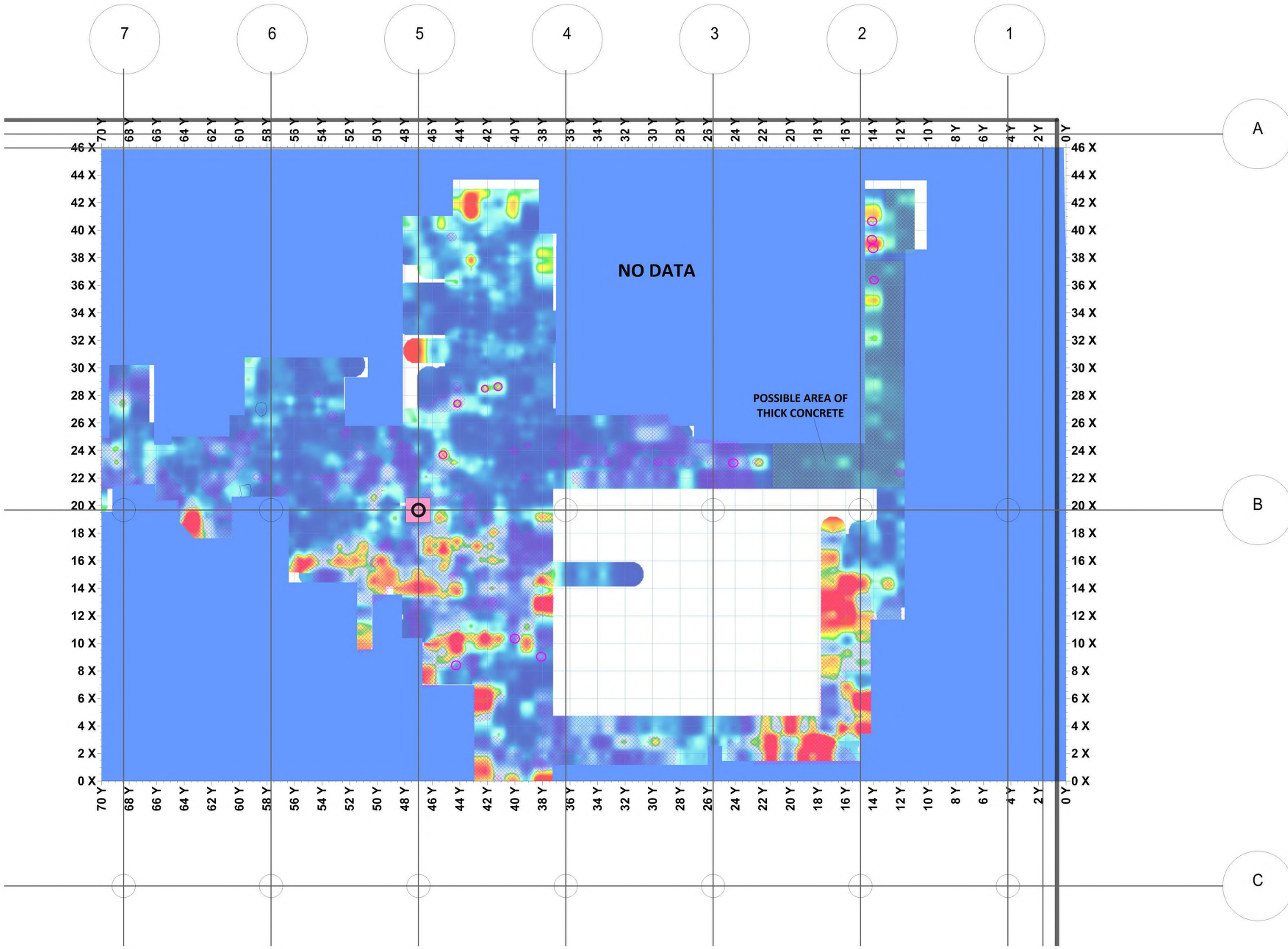

LEGEND

Pedestal Around Column 5B

Column 5B, Used for Sonic Echo/ Impulse Response Tests

Location and Orientation of GPR Traverse - 500 MHz Antenna

1500 MHz DEPTH SLICE LEGEND

Linear, High-Amplitude Reflection that may indicate rebar or seam in Mesh, or Conduit; Depth (ft.) as indicated on slice at which lineation first appears

Zone of High-Amplitude GPR Reflections that could indicate an excavation, voiding, boulder, or cross-sectional reflection from a vertically oriented timber pile. High-amplitude reflections can also indicate a localized area of increased moisture

Zone of Attenuation that may indicate an area of thick concrete, increased soil moisture, and/or ground-water conductivity

DEPTH: 4.7 ft. - 4.9 ft.

increasing amplitude

low (weak) amplitude

high (strong) amplitude

SCALE: 1 Inch = 4 Feet

4 0 4 Feet

FIGURE 7L
REPRESENTATIVE DEPTH SLICE IMAGE OF 500 MHz GPR DATA TO IDENTIFY TOP OF TIMBER PILES
EBSCO
10 ESTES STREET
IPSWICH, MASSACHUSETTS
Prepared for
SIMPSON GUMPERTZ & HEGER
JUNE 2021

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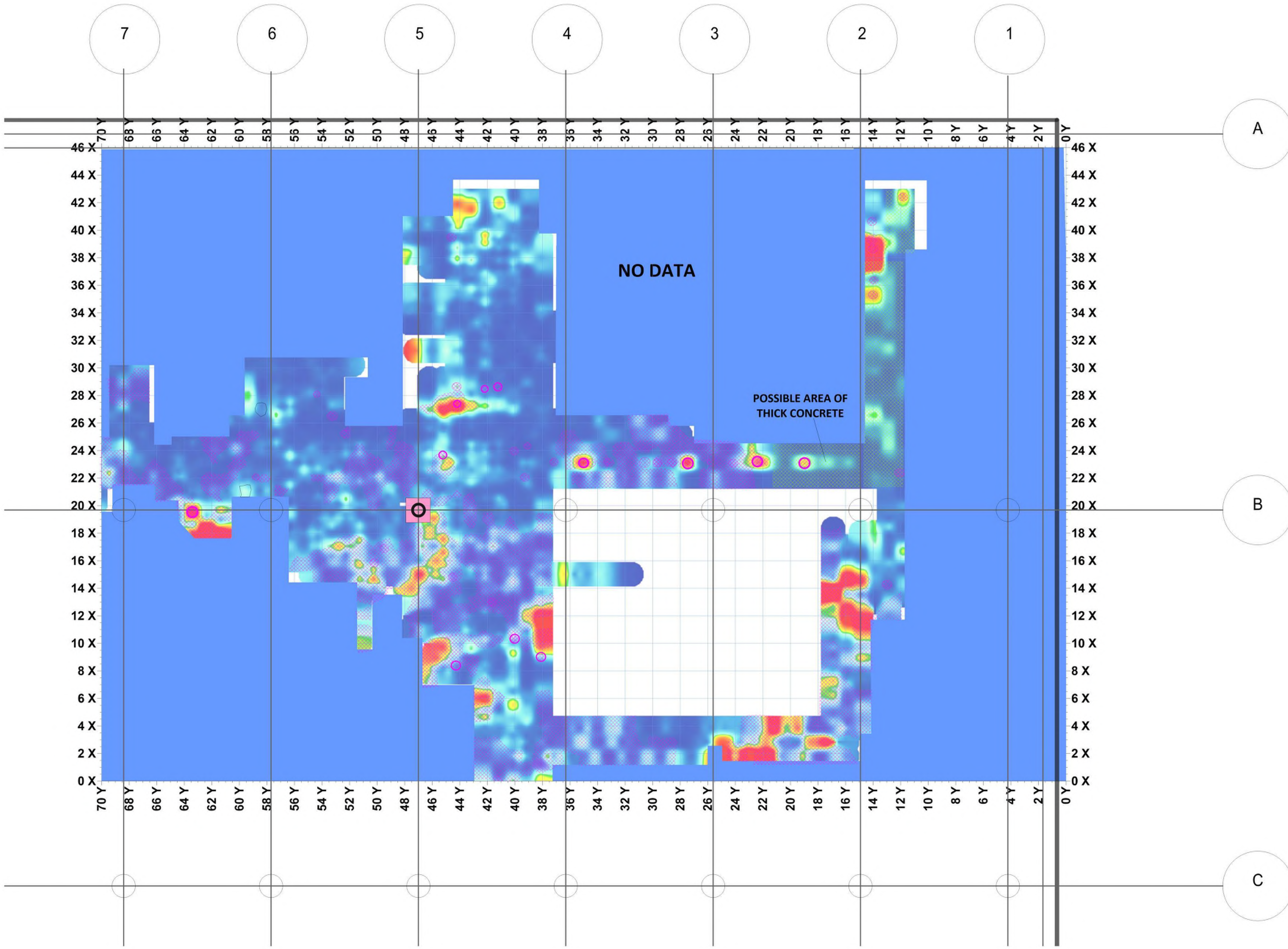

LEGEND

Pedestal Around Column 5B

Column 5B, Used for Sonic Echo/ Impulse Response Tests

Location and Orientation of GPR Traverse - 500 MHz Antenna

1500 MHz DEPTH SLICE LEGEND

Linear, High-Amplitude Reflection that may indicate rebar or seam in Mesh, or Conduit; Depth (ft.) as indicated on slice at which lineation first appears

Zone of High-Amplitude GPR Reflections that could indicate an excavation, voiding, boulder, or cross-sectional reflection from a vertically oriented timber pile. High-amplitude reflections can also indicate a localized area of increased moisture

Zone of Attenuation that may indicate an area of thick concrete, increased soil moisture, and/or ground-water conductivity

DEPTH: 5.5 ft. - 5.7 ft.

increasing amplitude

low (weak) amplitude

high (strong) amplitude

SCALE: 1 Inch = 4 Feet

4 0 4 Feet

FIGURE 7M
REPRESENTATIVE DEPTH SLICE IMAGE OF
500 MHz GPR DATA
TO IDENTIFY TOP OF TIMBER PILES
EBSCO
10 ESTES STREET
IPSWICH, MASSACHUSETTS
Prepared for
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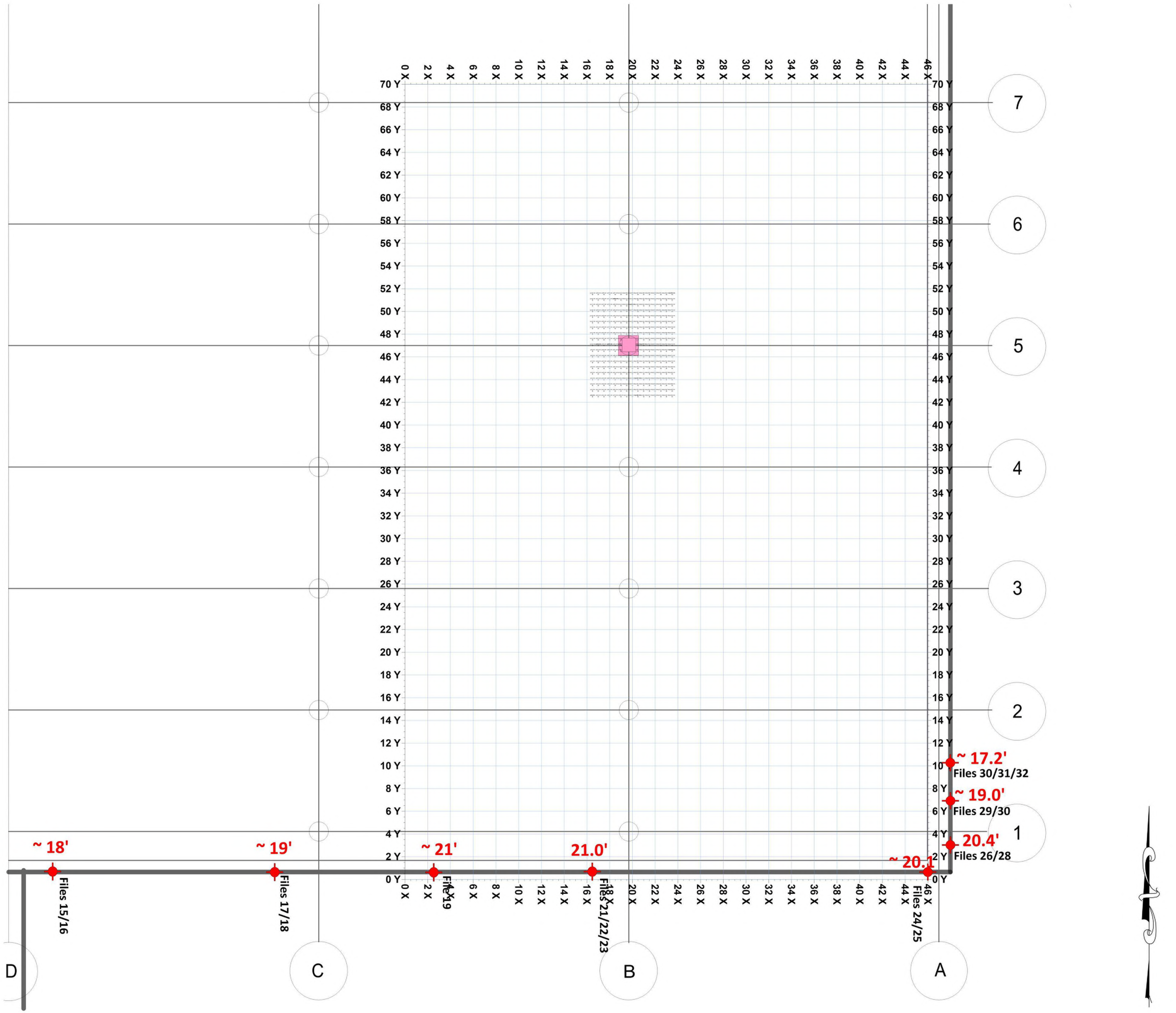

LEGEND

Pedistal

Location of Column 5B

Location of Impulse Echo Measurement (not shown)

Location of SEIR Measurement

10.7 Depth to Bottom of Wooden Timber Pile from SEIR Time-Domain Amplitude Plot

10.7 Depth to Bottom of Wooden Timber Pile from SEIR Frequency-Domain IR Test

NOTE: Measurements Taken with Dytrans Instrumented Hammer using a 1,000 Hz Accelerometer.

SCALE: 1 Inch = 4 Feet

4 0 4 Feet

FIGURE 8
FOUNDATION WALL DEPTH FROM SONIC ECHO/IMPULSE RESPONSE
EBSCO
10 ESTES STREET
IPSWICH, MASSACHUSETTS

Prepared for
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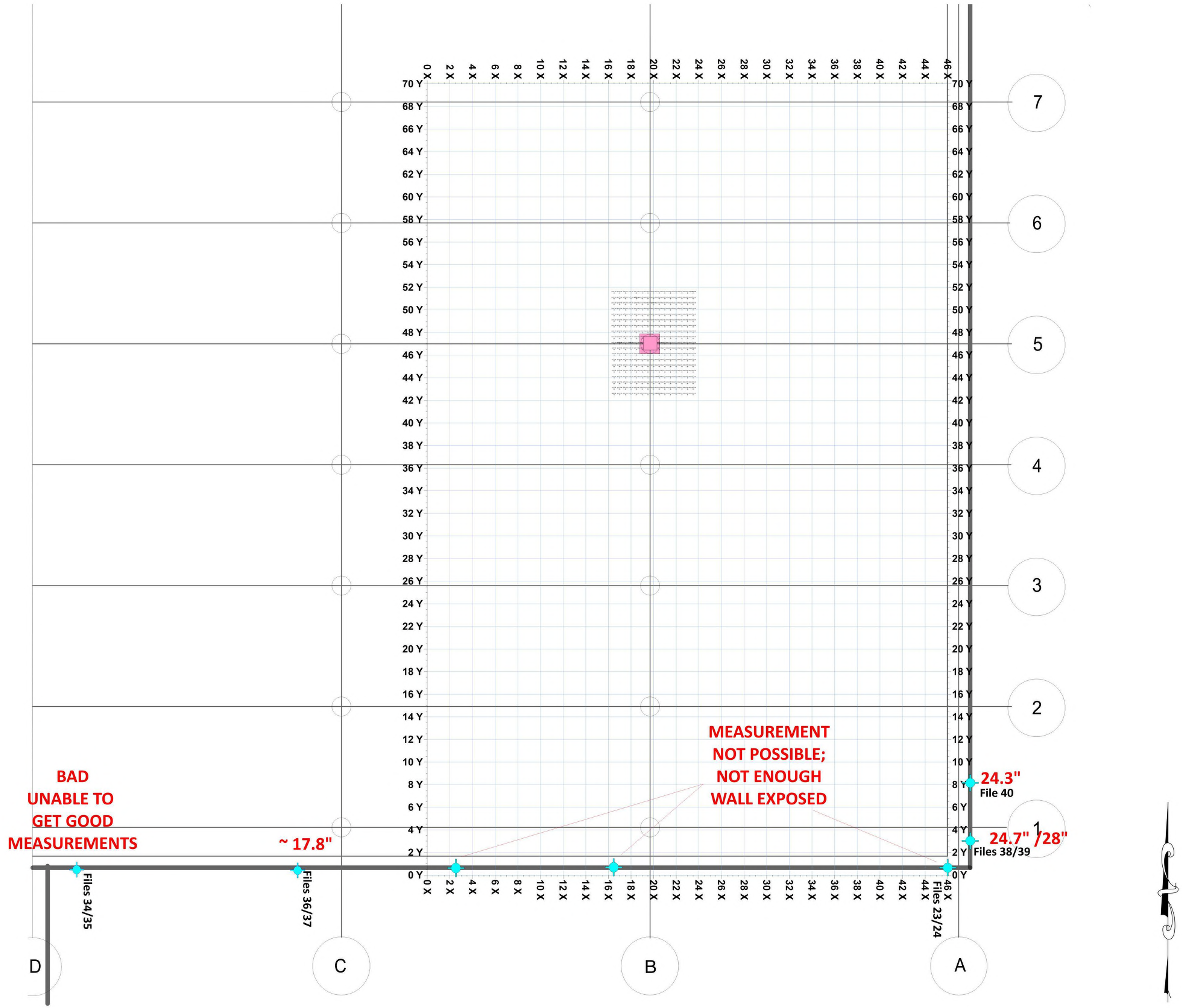

LEGEND

- Pedistal
- Location of Column 5B
- Location of Impact Echo Measurement (inside near Column 5B)
- Location of IE Measurement at Top of Foundation Wall (outside, south and east walls)
- ~18" Approximate Thickness of Foundation Wall at Top of Wall (inches)

NOTE: Measurements Taken with IE using a 12 oz ball pein hammer as its source, not the solenoid.

Proximity to the western wall prevented un-usable data due to high noise (ringing) relative to the signal amplitude.

SCALE: 1 Inch = 4 Feet

4 0 4 Feet

FIGURE 9
THICKNESS AT THE TOP OF FOUNDATION WALL FROM IMPACT ECHO
EBSCO
10 ESTES STREET
IPSWICH, MASSACHUSETTS

Prepared for
SIMPSON GUMPERTZ & HEGER
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RSI Geophysics for People and the Environment
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