100802-7-03
October 3, 2014

Mr. Ron Coler
Town of Ashfield Selectboard
412 Main Street
P.O. Box 560
Ashfield, MA 01330

Re: **Hydrology and Hydraulics Analysis**
Ashfield Lake Dam, MA00523

Dear Ron:

This letter summarizes the results of Tighe & Bond's Hydrology and Hydraulics (H&H) analysis of Ashfield Lake Dam, to assess the performance of the dam during its Spillway Design Flood (SDF), which is the one half of the probable maximum flood (1/2 PMF) as defined in the current Massachusetts dam safety regulations based on the dam's High Hazard and Intermediate Size classifications. A USGS site location map of the dam is shown as Figure 1. Figure 2 shows an orthophotograph of the dam and lake.

The results of the analysis show that the spillway can pass the Spillway Design Flood without overtopping the dam, although the available freeboard between the peak water surface elevation and the top of the dam is not adequate. Typical dam design practice is to provide at least 1 to 3 feet of available freeboard between the peak flood water surface elevation and the crest of the dam. Our recommendation is to raise the dam crest as part of the proposed dam repair project to increase the available freeboard, to 1.6 feet, although reasonable alternatives are available that may be considered by the Town.

Elevations in this report are in units of feet using North American Vertical Datum (NAVD) of 1988 except where otherwise noted. Reference to left and right directions assume an orientation looking in the downstream direction.

Existing Dam Geometry

Ashfield Lake Dam is an earthen dam approximately 775 feet long with a maximum height of 16 feet and a typical crest width of 14 feet. A ground survey performed in 2014 (see Appendix A) indicates that the vertical alignment of the crest is generally level with minor variations, ranging from a low point of elevation 1255.7 feet, located approximately 160 feet to the right of the spillway structure, to a higher area above 1256.0 located approximately 210 feet to the right of the spillway.

The dam's spillway is approximately 30 feet long and concrete capped, with a crest elevation of approximately 1251.2 feet. The spillway crest is a broad crested weir, with the downstream face consisting of stepped stone masonry. The upstream side of the spillway is covered by sand-sized sediment material that slopes upward to the crest at approximately 5 units horizontal run to 1 unit vertical rise (5H:1V). The spillway is confined by mortared stone training walls, the crests of which are capped with concrete to support a steel footbridge. The elevation of the top of training walls is approximately 1256.4 feet.

A dike is located east of the dam along Buckland Road. We understand that the dike was newly constructed in 1990 to prevent the road from overtopping during large storms. It is approximately 160 feet in length, has a crest elevation of approximately 1255.2 feet, which is approximately 0.5 feet below the crest of the dam, and has a low point on the downstream side of 1251.6 feet. The dike is not registered separately from the dam with the Office of Dam Safety. Design plans, dated 1988, from the dam's last rehabilitation project called for the crests of the dike and dam to be the same elevation (1256.7 NGVD 1929), so there is a discrepancy between the design and existing conditions. As-built or record drawings have not been located, and may not exist.

The low area on the downstream side of the dike slopes upward to Buckland Road and collects drainage from Buckland Road. A 6-inch corrugated metal pipe (CMP) drains from the low area into the lake (flow in the pipe will reverse when the water surface of the lake rises above the level in the low area).

A low level outlet consisting of a 4 foot wide, 2.25 foot high rectangular concrete conduit, lined with a 15-inch diameter pipe, is located at the east end of the spillway, discharging at the base of the lowest step (discharge elevation 1240.4 feet±). Discharges through the 40 foot long conduit are controlled by a manually operated sluice gate. The gate is located on the downstream wall of a concrete capped, 3 feet by 4.5 feet stone masonry gate shaft that is located adjacent to the left training wall. The inlet to the low level conduit is a riser-pipe just outside the base of the spillway structure, about 30 feet from the upstream wall of the gate shaft.

Discharges from the spillway and the low level outlet are carried by two stream channels. The primary channel, carrying low flows, follows the natural brook and is 10 feet wide and 3 feet deep. At approximately 500 feet downstream it continues in an underground 48-inch diameter steel pipe for approximately 500 additional feet. The secondary channel, which carries the high flows, is lined with stone masonry and is about 1,000 feet long, 10 feet wide and 6 feet deep. Both channels converge upstream of Buckland Road to form the South River which flows into the Deerfield River, a tributary of the Connecticut River.

Past Analyses

A number of past hydrologic and hydraulic analyses have been performed for the dam and are on-file with the Office of Dam Safety. These analyses were each prepared using older methods, assumed different design storm rainfalls and durations from each other and from current generally-accepted dam safety practice in Massachusetts, and were not based on the current dam geometry or watershed and rainfall characteristics. However, one of these analyses, prepared by GZA in 1987 and that was the basis for the 1990 repairs, included a wave run-up analysis, the results of which are in a report on-file with the Office of Dam Safety and is discussed later in this report.

Hydrology

Tighe & Bond performed a hydrology and hydraulic (H&H) analysis using the HydroCAD stormwater modeling program which is based on the United States Department of Agriculture's Technical Release 20 program (TR-20). The model was developed using information from USGS and GIS mapping, soil characteristics, watershed characteristics and ground cover types within the watershed. Principal hydrology input values for the modeling program include: rainfall amount, the total contributing watershed area, land use, hydrologic soil types based on USDA Natural Resources Conservation Service (NRCS) soil surveys, and time of concentration (t_c).

Ashfield Lake Dam is an Intermediate size, High (Class I) hazard potential structure. In accordance with the current dam safety regulations, the SDF for the dam is one half of the probable maximum flood (1/2 PMF), which is the runoff hydrograph that corresponds to one half of the probable maximum precipitation (1/2 PMP) falling on the watershed. The reservoir and dam were evaluated for the ½ PMF, as well as the 100-year and 500-year frequency storms for informational purposes. Table 1 provides the precipitation amounts used for the various storms analyzed.

Table 1
24-hr Precipitation Values Used in H&H Analysis

Storm Return Frequency	Precipitation Values (inches)
100-year	7.41
500-year	10.76
½ PMF	15.5

A full 24-hour PMP value of 31 inches was obtained from the 1988 Massachusetts Department of Environmental Management, Division of Planning & Development, Office of Dam Safety "Dam Operations and Maintenance Manual." The precipitation value for the ½ PMP used in this analysis is half of the full PMP, corresponding to 15.5 inches of rainfall. The 24-hour precipitation for the 100-year and 500-year frequency storms were estimated for this location based on the Northeast Regional Climate Center's (NRCC) Extreme Precipitation Tables.

The reservoir's watershed area is shown in Figure 3. It is approximately 684 acres, or 1.07 square miles in area. The overall watershed area consists of wooded, hilly terrain with limited areas of residential and agricultural land use, and is located entirely within the Town of Ashfield. Soils within the drainage area are dominated by those characterized by NRCS as Hydrologic Soil Group Type B soils (moderate infiltration rate when thoroughly wet) and Type D soils (very low infiltration rate and high runoff potential when thoroughly wet).

The area was divided into three subsidiary drainage areas, including:

- Drainage Area 1, which 451.6 acres in area and is located west of Route 112. The runoff from this area collects in an unnamed stream and flows below Route 112 in a 72-inch diameter culvert.

- Drainage Area 2, which is 194.3 acres in area and is characterized by direct inflow into Ashfield Lake without collection into a major defined tributary.

- Direct precipitation falling on the surface of the lake, which is 38.3 acres.

Drainage Area 3 shown on Figure 3 does not contribute to the lake, but instead collects in a low area between the dam and Route 116, eventually draining to the South River immediately downstream of the dam. It was not included in this model, but does contribute to the drainage issue on the downstream side of the dam and is being evaluated through other tasks of the dam repair project.

Time of concentration (t_c) was determined using a time lag method, which was developed to allow calculation for t_c over a wide range of conditions. The average land slope used in the t_c calculations was obtained from LiDAR data available from MassGIS. The average land slope is approximately 22% in both Drainage Areas 1 and 2. The hydraulic length of each drainage area was estimated using the USGS streamstats application.

Hydraulics

Principal input values for the hydraulic module of the program include the elevation, length, and height of the spillway, the available storage volume behind the dam, the size and invert elevation of the culvert below Route 112 that connects Drainage Area 1 to Ashfield Lake, and the available storage volume on the upstream side of the road.

The dimensions and elevation of the dam and spillway were obtained from topographic survey. The dimensions of the culvert through Route 112 were measured in the field. Stage-storage curves representing impoundment storage volumes in the lake and upstream of Route 112 were prepared using the MassGIS LiDAR data.

The size and slope of the downstream channels are expected to convey flows away from the dam without resulting in a tailwater condition that could reduce the spillway's efficiency. The culverts below Buckland Road would serve as hydraulic restrictions, but the crest of the road is approximately 10 feet below the crest of the dam, such that, during large storms, high flows would pass over the road without subjecting the spillway to tailwater effects.

A summary of hydraulic information used for the HydroCAD model input values are as follows:

- Ashfield Lake Dam
  a) Spillway Length – 30 feet
  b) Spillway Crest Elevation – 1251.2 feet
  c) Top of Spillway Training Walls – 1256.4 feet
  d) Low Point of Dam Crest Elevation – 1255.7 feet
  e) Low Point of Dike Crest Elevation – 1255.2 feet
- Route 112 Culvert (upstream of dam)
  a) Culvert Diameter – 72 inches
  b) Culvert Inlet invert – 1262.0 feet
  c) Roadway Crest Elevation – 1288.0 feet

Analysis Results

The HydroCAD model was analyzed using the above described hydrology and hydraulic values. The model results for the 100 and 500-year return frequency storms, and the ½ PMF design flood are presented in Table 2 below. Appendix B contains the model output summary sheet for each of the storm events.

Table 2
Hydraulic/Hydrologic Analysis Results

	Storm Recurrence Frequency		
	100-Year	**500-Year**	**½ PMF**
Peak Inflow into Ashfield Lake (CFS)	641	1,134	1,814
Peak Discharge Through Ashfield Lake Dam Spillway (CFS)	234	408	619
Peak Water Surface Elevation (FT)	1,253.30	1254.19	1255.15
Freeboard at Low Point of Dam Crest (FT)	2.40	1.51	0.55
Freeboard at Low Point of Dike Crest (FT)	1.90	1.01	0.05

As shown in Table 2, Ashfield Reservoir Dam can pass the ½ PMF without overtopping the dam, although the available freeboard at the peak water surface elevation is less than 1 foot and is thus inadequate, since wind-induced waves could overtop the dam, potentially causing erosion that could lead to a failure of the dam.

Similarly, almost no freeboard is available between the peak water surface elevation and the crest of the dike since the dike crest is lower than the dam crest, resulting in a higher potential for failure of the dike than for the dam during the SDF. However, the potential consequences of a failure of a dike appear to be lower than that of the dam. As such, there is the potential that the Office of Dam Safety would consider a lower hazard classification for the dike, such as Significant or Low if the dike were to be registered separately from the dam.

If this were the case, and the hazard classification of the dike were lowered, than the design storm of the dike could potentially be lowered. For example, if the dike were registered as a Significant-hazard structure, its spillway design flood would be the 100-year storm, during which approximately 1.9 feet of freeboard is currently available, which would be adequate. However, if the dike were registered separately, then separate Phase I inspections would need to be performed at regular intervals

It is important to note that significant, temporary flood storage is provided by the 72-inch culvert below Route 112, which serves as a hydraulic restriction. If, for some reason, the culvert were replaced with a larger one in the future, peak flows into Ashfield Lake are likely to increase, reducing the available freeboard at the dam during the SDF. For example, the model shows that replacement of the culvert with a free flowing channel would result in the dam embankment overtopping by approximately 10 inches during the SDF. To compensate, the dam and dike would have to be raised by 1.71 feet, and 2.21 feet, respectively, to meet the increased water surface elevation, not including any freeboard.

Although the model does not suggest that the Route 112 culvert is inadequately sized, permitting programs incentivize increased culvert sizes to meet stream crossing standards for fish and wildlife passage, which generally results in increased hydraulic capacity as a side effect. Route 112 is a MassDOT-jurisdictional road, so the Town should maintain familiarity with proposed MassDOT projects in the area and coordinate with MassDOT if culvert modifications are proposed in the future.

Freeboard - While the methods used in this analysis determine peak flatwater elevations during storm events, they cannot be used to determine the freeboard allowance that should be provided for wave effects. No wave run-up analysis was performed as part of this analysis. However, a wave run-up analysis was performed as part of the 1987 hydrology and hydraulics analysis. That analysis indicated that 1.6 feet of freeboard should be provided from the peak water surface elevation to the crest of the dam, but noted that there is a conservative bias for small impoundments in general and for Ashfield Lake in particular since it is surrounded by hilly terrain that would tend to reduce wind speed across the lake.

Recommendations

In general, ways to improve the dam's spillway performance may include:

1. Raising the dam crest to increase impoundment flood storage.

2. Installing embankment overtopping protection, which in essence, turns the embankment into an auxiliary spillway.

3. Providing additional spillway capacity, either through widening the spillway or providing an auxiliary spillway.

4. Lowering the spillway crest elevation to increase impoundment storage, and increase the spillway's discharge capacity.

We have assumed when evaluating these alternatives that the planned dam repair project will include regrading the crest of the dam and dike to an even elevation of approximately 1256.0 feet, which is generally within 0.3 feet of the high and low areas along the dam embankment and is 0.8 feet above the dike embankment crest.

Alternative 1 is recommended; since regrading the crest is planned for inclusion in the proposed project anyway, it will likely be significantly more cost effective to raise the crest by an additional increment to achieve acceptable freeboard than to implement the other alternatives.

Several sub-alternatives for consideration by the Town were developed to meet SDF performance goals. These were compared to a base alternative that assumes the crests of the dam and dike will be regraded to 1256.0 feet at minimum as part of the repair project (opinions of probable cost for which have not yet been developed).

The sub-alternatives include:

- Alternative 1A - Raise the crests of the dam and dike to the top of the existing spillway training walls at elevation 1256.4 feet. This alternative provides the greatest freeboard that is allowed by the existing training walls, but it is 0.4 feet less than the full freeboard recommended by the 1987 wave run-up analysis. Our opinion of probable cost (OPC) for this alternative is an additional $13,000 compared to the base alternative. The Office of Dam Safety may be willing to consider 1.2 feet of freeboard since it will match the top of the training walls, which is convenient, and since the wave run-up analysis report indicates that the method used has a conservative bias.

- Alternative 1B – Raise the crests of the dam and dike to 1256.8 to provide the full freeboard depth recommended by past analyses, except surrounding the spillway, where the crest would be sloped downward to the existing top of the training walls. These sloping portions would be protected with permanent erosion protection, such as Turf Reinforcement Mats (TRMs), to provide resistance to wave action. Our opinion of probable cost associated with this alternative is $27,000 compared to the base alternative. Literature on a typical TRM product is included in Appendix C.

- Alternative 1C - Raise the entire dam and dike crest to 1256.8 feet, including raising the spillway training walls and the bridge supports to accommodate the higher embankment. This alternative would provide the full freeboard depth recommended by previous analyses. Our opinion of probable cost associated with this alternative is an additional $50,000 compared to the base alternative. Note that, if this alternative is selected, the Town should consider raising the walls by an additional increment (such as one foot) to provide reveal and to allow for future flexibility if dam safety regulations change, but that a structural evaluation of the walls in their existing and proposed condition may be needed to confirm that they have adequate structural capacity to accommodate the additional height, which could significantly increase costs.

Alternative 2, consisting of leveling the dam crest and dike crest to 1256.0 feet without raising it and installing TRMs to provide resistance to wave action, could also be considered. Our opinion of probable incremental cost associated with this alternative is $45,000. Note that TRMs have a limited service life (typically 25 to 50 years), compared to concrete structures that would generally have services lives of 50 years or longer. As such, although this alternative may be less costly initially than raising the dam crest and spillway walls, the long-term cost of this alternative may be higher.

Alternatives 3 and 4 are not likely to be cost effective compared to Alternatives 1 and 2, and would potentially result in adverse consequences; Alternative 3 would increase downstream storm flows in the South River, potentially increasing the frequency and depth of overtopping of overtopping of downstream culverts and bridges, and Alternative 4 would result in marginal decreases in the lake's depth and impoundment volume. As such, these alternatives are not currently recommended.

Table 3, below, presents a summary of the alternatives that were considered. Alternatives 3 and 4 are excluded since they are not currently considered feasible.

Table 3
Summary of Alternatives

Alternative	Summary	Freeboard Provided	Advantages	Disadvantages	OPC compared to Base Alt.
Base	Regrade dam crest to 1256.0 feet	0.8 feet	N/A	Less freeboard than generally considered acceptable.	N/A
1A	Raise dam and dike crests to 1256.4 feet (top of spillway training walls)	1.2 feet	Low cost, improved performance compared to existing conditions	Less freeboard than recommended by wave run-up analysis.	$13,000

Table 3
Summary of Alternatives

Alternative	Summary	Freeboard Provided	Advantages	Disadvantages	OPC compared to Base Alt.
1B	Raise majority of dam and dike crests to 1256.8 feet. Slope crest down to tops of training walls and cover sloped portions with TRMs.	1.6 feet, with lower areas around spillway.	Relatively low cost. Provides freeboard recommended by wave run-up analysis.	Lower design life of TRMs compared to concrete.	$27,000
1C	Raise dam and dike crests to 1256.8 feet. Raise spillway training walls to accommodate higher embankment	1.6 feet	Improved robustness and longevity.	Higher cost.	$50,000
2	Regrade dam and dike crests to 1256.0 feet, cover entire crest with TRMs	0.8 feet. TRMs to resist wave action	Lower initial cost than Alternative 1C	Shorter design life may result in higher long-term costs	$45,000

In addition, we do not recommend that the Town approach the Office of Dam Safety to reduce the hazard classification of the dike at this time, since raising the crest of the dike to match the elevation of the dam can likely be performed at relatively low cost. However, this recommendation should be revisited once the design progresses and dam repair opinions of probable cost are developed. It should also be revisited in the event that the Route 112 culvert is modified. In the interim, the dike should be considered a part of the dam system and inspected along with the dam.

We appreciate the opportunity to provide these services and look forward to continue working with the Town. Please contact Dan Buttrick at 413-572-3225 with questions, comments, or require additional information.

Very truly yours,

TIGHE & BOND, INC.

Daniel R. Buttrick, P.E.
Project Engineer

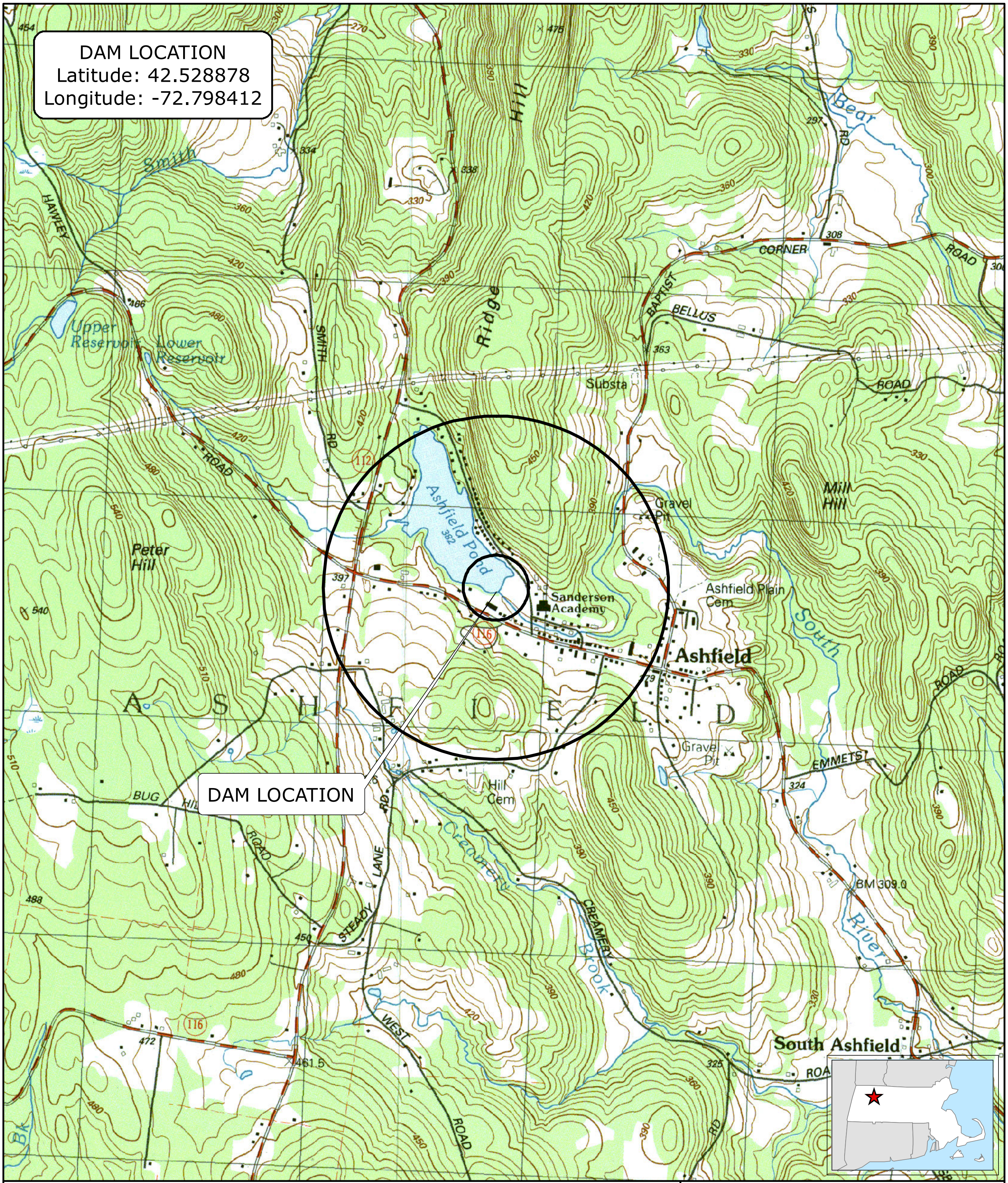
Christopher D. Haker, P.E.
Principal Engineer

Enclosures: Figure 1. Site Location Map
Figure 2. Aerial Orthophoto
Figure 3. Drainage Area Map

Appendix A. Survey Plan
Appendix B. HydroCAD Output Report
Appendix C. Turf Reinforcement Mat Manufacturer Literature

J:\A\A0802\REPORT\Ashfield Lake Dam H&H\Ashfield Lake Dam Letter Report Format.doc

Figure 1
Site Location Map



Tighe&Bond
Consulting Engineers
Environmental Specialists

Based on USGS Topographic Map for
Ashfield, Revised in 1990
6-Meter Contour Interval
Circles indicate 500-foot and half-mile radii

1:24,000
0 1,000 2,000
Feet

N

FIGURE 1
SITE LOCUS
Ashfield Lake Dam
Town of Ashfield
Ashfield, Massachusetts
MA00523

July 2014

Figure 2
Aerial Orthophoto

FIGURE 2
ORTHOPHOTOGRAPH
Ashfield Lake Dam
Town of Ashfield
Ashfield, Massachusetts
MA00523

1:6,000
0 250 500
Feet

N

Tighe&Bond
Consulting Engineers
Environmental Specialists

Based on MassGIS Color Orthophotography (2011-2012)

July 2014

V:\Projects\A\A0802\Maps\2014\MA00523_Ortho.mxd

A-0802

Figure 3
Drainage Area Map

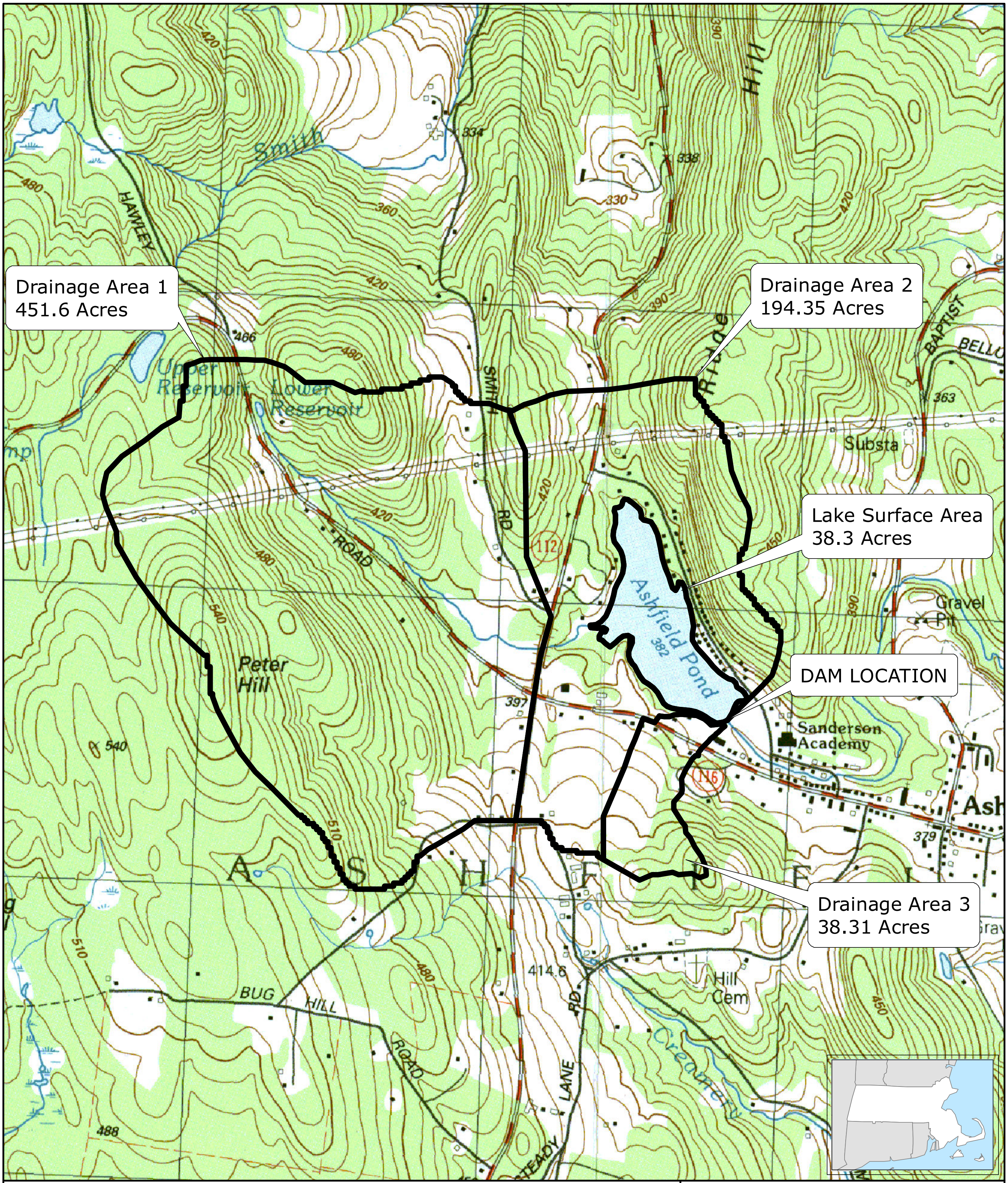


FIGURE 3
DRAINAGE AREA

Ashfield Lake Dam
Town of Ashfield
Ashfield, Massachusetts
MA00523

August 2014

Based on USGS Topographic Map for Ashfield, Revised in 1990
6-Meter Contour Interval
Circles indicate 500-foot and half-mile radii

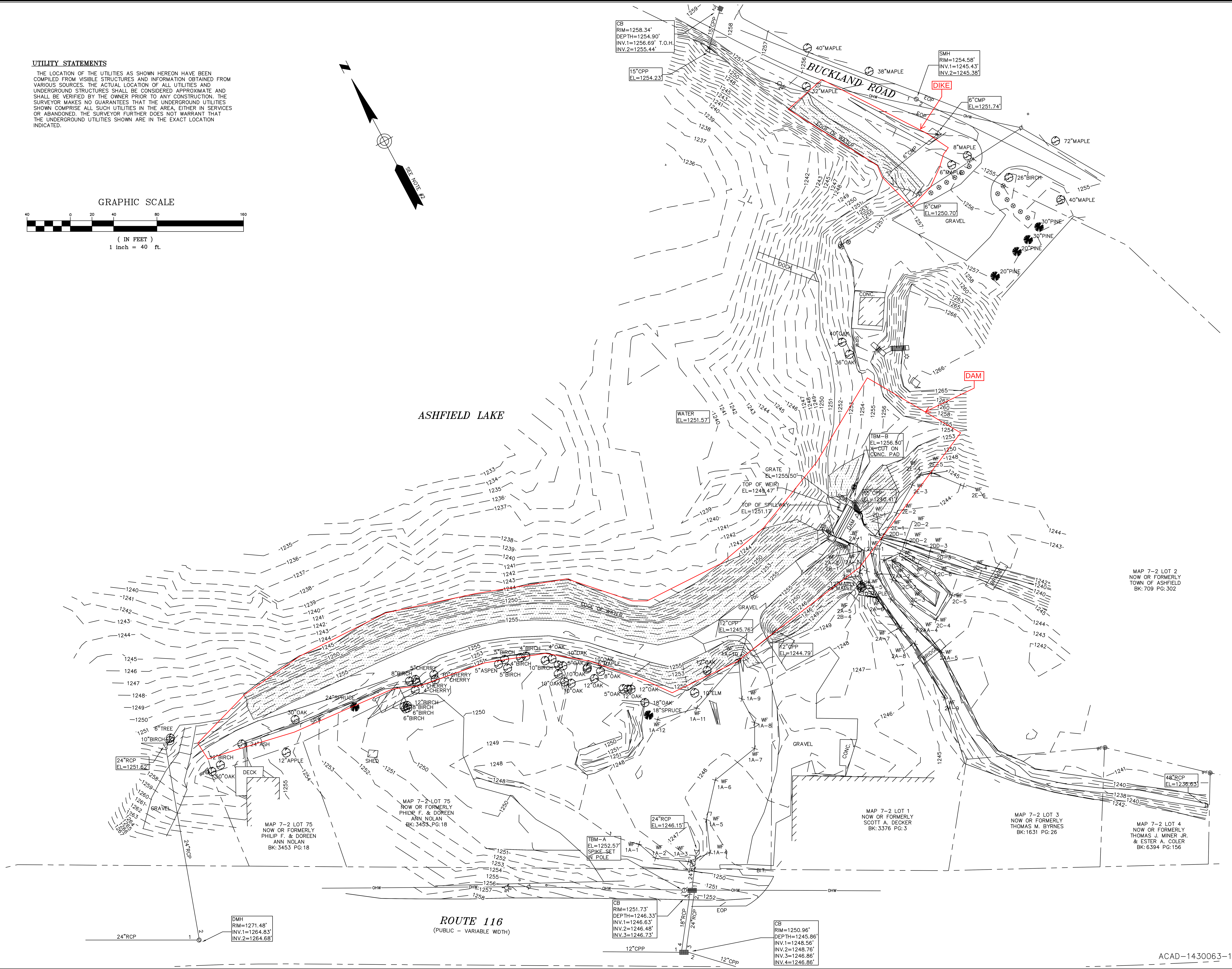
1:18,000
0 750 1,500
Feet

Tighe&Bond
Consulting Engineers
Environmental Specialists

V:\Projects\A\A0802\Maps\2014\MA00523_Drainage_New.mxd

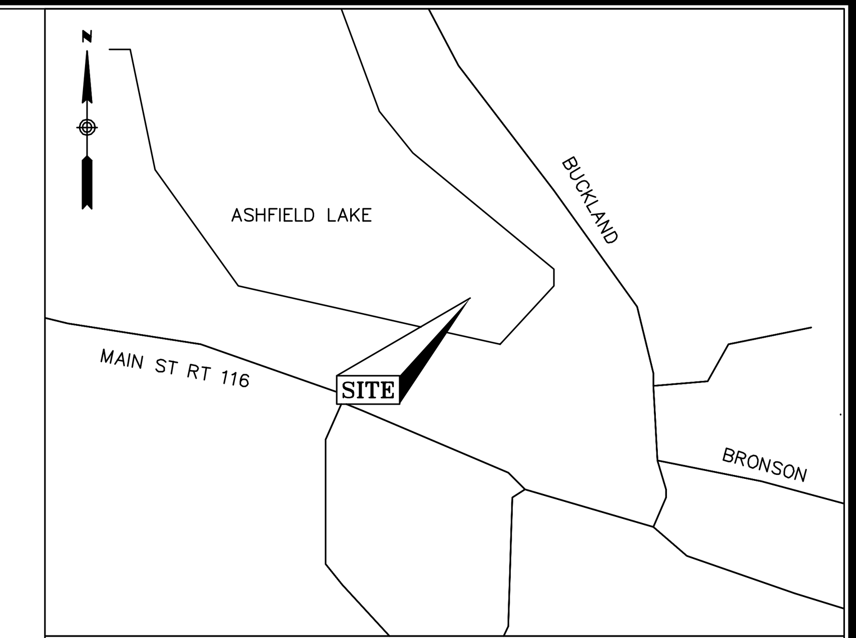
A-0802

Appendix A
Survey Plan



UTILITY STATEMENTS

THE LOCATION OF THE UTILITIES AS SHOWN HEREON HAVE BEEN COMPILED FROM VISIBLE STRUCTURES AND INFORMATION OBTAINED FROM VARIOUS SOURCES. THE ACTUAL LOCATION OF ALL UTILITIES AND UNDERGROUND STRUCTURES SHALL BE CONSIDERED APPROXIMATE AND SHALL BE VERIFIED BY THE OWNER PRIOR TO ANY CONSTRUCTION. THE SURVEYOR MAKES NO GUARANTEES THAT THE UNDERGROUND UTILITIES SHOWN COMPRISE ALL SUCH UTILITIES IN THE AREA, EITHER IN SERVICES OR ABANDONED. THE SURVEYOR FURTHER DOES NOT WARRANT THAT THE UNDERGROUND UTILITIES SHOWN ARE IN THE EXACT LOCATION INDICATED.



LOCUS MAP
(N.T.S.)

NOTES

1. THIS PLAN WAS PREPARED FROM AN ACTUAL ON THE GROUND FIELD SURVEY CONDUCTED BY WSP ON JUNE 25, 2014 – JUNE 30, 2014 AND JULY 29, 2014.

2. THE HORIZONTAL DATUM SHOWN HEREON REFERENCES MASSACHUSETTS STATE PLANE NAD83.

3. THE VERTICAL DATUM SHOWN HEREON REFERENCES NAVD88.

4. THE WETLANDS SHOWN HEREON WERE FLAGGED BY OTHERS AND FIELD LOCATED BY WSP.

5. THE PROPERTY LINES SHOWN HEREON WERE GENERATED BY DEEDS AND PLANS OF RECORD AND NOT FROM A TYPICAL NARRATIVE SURVEY.

PLAN REFERENCES

1. "PLAN OF LAND IN ASHFIELD MASSACHUSSETTS" FOR TOWN OF ASHFIELD BOARD OF SELECTMEN. PREPARED BY ALMER HUNTLEY, JR. & ASSOCIATES, INC. DATED DECEMBER 6, 1985.

2. "THE COMMONWEALTH OF MASSACHUSETTS PLAN OF ROAD" IN THE TOWN OF ASHFIELD, MASSACHUSETTS DATED APRIL 22, 1958 ON FILE AT THE MASSACHUSETTS DEPARTMENT OF TRANSPORTATION AS STATE HIGHWAY LAYOUT #4662.

LEGEND

TBM	TEMPORARY BENCHMARK
BOMO	BOLT OVER MAIN OUTLET
EOP	EDGE OF PAVEMENT
CMP	CORRUGATED METAL PIPE
RCP	REINFORCED CONCRETE PIPE
CPP	CORRUGATED PLASTIC PIPE
CMP	CORRUGATED METAL PIPE
SRW	STONE RETAINING WALL
INV.	INVERT
T.O.H.	TOP OF HOOD

IPF IRON PIPE FOUND
IRF IRON ROD FOUND
DECIDUOUS TREE
CONIFER TREE
FDC FIRE DEPARTMENT CONNECTION
FIRE HYDRANT
BOLLARD
SEWER MANHOLE
MONITORING WELL
DRAIN MANHOLE
CATCH BASIN
DOUBLE CATCH BASIN
CULVERT
UTILITY POLE
UTILITY POLE WITH TRANSFORMER
GUY WIRE
WETLAND FLAG
ABUTTERS LOT LINE
APPROXIMATE PROPERTY LINE
APPROXIMATE RIGHT OF WAY
DRAIN LINE
SEWER LINE
OVERHEAD WIRES
CHAIN LINK FENCE
TREE LINE
INTERMEDIATE CONTOURS
INDEX CONTOURS
WETLAND LINE
EDGE OF WATER
RIP RAP

EXISTING CONDITIONS
ASHFIELD LAKE DAM
ASHFIELD, MASSACHUSETTS
PREPARED FOR
TIGHE & BOND

WSP
Transportation & Infrastructure
155 Main Dunstable Rd. Suites 120 & 125 ▪ Nashua, NH 03060 ▪ 603.595.7900
www.wspgroup.com

Drawn By	CG,MS	Date	Job No.
Surveyed By	WP,MK	AUGUST 4, 2014	1430063
Checked By	DJH	Scale	Sheet No.
Book No.	N-268	1" = 40'	1 OF 1

Appendix B
HydroCAD Output Report

Summary for Pond 5P: Ashfield Lake Existing

```
Inflow Area =     684.211 ac,   5.96% Impervious,  Inflow Depth = 10.78"    for  1/2 PMF event
Inflow      =   1,813.86 cfs @  12.36 hrs,  Volume=        614.788 af
Outflow     =     619.33 cfs @  15.21 hrs,  Volume=        614.318 af,  Atten= 66%,  Lag= 170.6 min
Primary     =     619.33 cfs @  15.21 hrs,  Volume=        614.318 af
Secondary =         0.00 cfs @    0.00 hrs,  Volume=          0.000 af
```

Routing by Stor-Ind method, Time Span= 0.00-100.00 hrs, dt= 0.20 hrs
Peak Elev= 1,255.15' @ 15.21 hrs Surf.Area= 46.670 ac Storage= 168.782 af

Plug-Flow detention time= 238.5 min calculated for 613.092 af (100% of inflow)
Center-of-Mass det. time= 244.1 min (1,186.3 - 942.1)

Volume	Invert	Avail.Storage	Storage Description
#1	1,251.20'	308.662 af	**Custom Stage Data (Prismatic)**Listed below (Recalc)

Elevation (feet)	Surf.Area (acres)	Inc.Store (acre-feet)	Cum.Store (acre-feet)
1,251.20	38.300	0.000	0.000
1,254.00	44.730	116.242	116.242
1,258.00	51.480	192.420	308.662

Device	Routing	Invert	Outlet Devices
#1	Primary	1,251.20'	**30.0' long x 20.0' breadth Spillway** Head (feet) 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60 Coef. (English) 2.68 2.70 2.70 2.64 2.63 2.64 2.64 2.63
#2	Secondary	1,255.70'	**775.0' long x 14.0' breadth Broad-Crested Rectangular Weir** Head (feet) 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60 Coef. (English) 2.64 2.67 2.70 2.65 2.64 2.65 2.65 2.63

Primary OutFlow Max=619.31 cfs @ 15.21 hrs HW=1,255.15' (Free Discharge)
└─**1=Spillway** (Weir Controls 619.31 cfs @ 5.23 fps)

Secondary OutFlow Max=0.00 cfs @ 0.00 hrs HW=1,251.20' (Free Discharge)
└─**2=Broad-Crested Rectangular Weir** (Controls 0.00 cfs)

Summary for Pond 11P: Ashfield Lake Existing No Route 112

```
Inflow Area =     684.211 ac,   5.96% Impervious,  Inflow Depth = 10.78"    for  1/2 PMF event
Inflow      =   3,822.08 cfs @  12.48 hrs,  Volume=        614.788 af
Outflow     =   2,444.09 cfs @  12.91 hrs,  Volume=        614.369 af,  Atten= 36%,  Lag= 25.9 min
Primary     =     963.91 cfs @  12.91 hrs,  Volume=        523.285 af
Secondary =   1,480.19 cfs @  12.91 hrs,  Volume=         91.084 af
```

Routing by Stor-Ind method, Time Span= 0.00-100.00 hrs, dt= 0.20 hrs
Peak Elev= 1,256.50' @ 12.91 hrs Surf.Area= 48.956 ac Storage= 233.563 af

Plug-Flow detention time= 205.5 min calculated for 613.143 af (100% of inflow)
Center-of-Mass det. time= 211.5 min (1,039.1 - 827.6)

Volume	Invert	Avail.Storage	Storage Description
#1	1,251.20'	308.662 af	**Custom Stage Data (Prismatic)** Listed below (Recalc)

Elevation (feet)	Surf.Area (acres)	Inc.Store (acre-feet)	Cum.Store (acre-feet)
1,251.20	38.300	0.000	0.000
1,254.00	44.730	116.242	116.242
1,258.00	51.480	192.420	308.662

Device	Routing	Invert	Outlet Devices
#1	Primary	1,251.20'	**30.0' long x 20.0' breadth Spillway** Head (feet) 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60 Coef. (English) 2.68 2.70 2.70 2.64 2.63 2.64 2.64 2.63
#2	Secondary	1,255.70'	**775.0' long x 14.0' breadth Broad-Crested Rectangular Weir** Head (feet) 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60 Coef. (English) 2.64 2.67 2.70 2.65 2.64 2.65 2.65 2.63

Primary OutFlow Max=958.22 cfs @ 12.91 hrs HW=1,256.48' (Free Discharge)
└**1=Spillway** (Weir Controls 958.22 cfs @ 6.05 fps)

Secondary OutFlow Max=1,426.70 cfs @ 12.91 hrs HW=1,256.48' (Free Discharge)
└**2=Broad-Crested Rectangular Weir** (Weir Controls 1,426.70 cfs @ 2.35 fps)

Summary for Pond 27P: Ashfield Lake Proposed No Route 112

Inflow Area = 684.211 ac, 5.96% Impervious, Inflow Depth = 10.78" for 1/2 PMF event
Inflow = 3,822.08 cfs @ 12.48 hrs, Volume= 614.788 af
Outflow = 1,221.37 cfs @ 13.32 hrs, Volume= 614.367 af, Atten= 68%, Lag= 50.1 min
Primary = 1,221.37 cfs @ 13.32 hrs, Volume= 614.367 af

Routing by Stor-Ind method, Time Span= 0.00-100.00 hrs, dt= 0.20 hrs
Peak Elev= 1,257.41' @ 13.32 hrs Surf.Area= 50.487 ac Storage= 278.647 af

Plug-Flow detention time= 233.4 min calculated for 614.367 af (100% of inflow)
Center-of-Mass det. time= 232.6 min (1,060.2 - 827.6)

Volume	Invert	Avail.Storage	Storage Description
#1	1,251.20'	308.662 af	**Custom Stage Data (Prismatic)** Listed below (Recalc)

Elevation (feet)	Surf.Area (acres)	Inc.Store (acre-feet)	Cum.Store (acre-feet)
1,251.20	38.300	0.000	0.000
1,254.00	44.730	116.242	116.242
1,258.00	51.480	192.420	308.662

Device	Routing	Invert	Outlet Devices
#1	Primary	1,251.20'	**30.0' long x 20.0' breadth Spillway** Head (feet) 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60 Coef. (English) 2.68 2.70 2.70 2.64 2.63 2.64 2.64 2.63

Primary OutFlow Max=1,218.20 cfs @ 13.32 hrs HW=1,257.40' (Free Discharge)
└─**1=Spillway** (Weir Controls 1,218.20 cfs @ 6.55 fps)

Summary for Pond 5P: Ashfield Lake Existing

Inflow Area = 684.211 ac, 5.96% Impervious, Inflow Depth = 3.64" for 100-yr event
Inflow = 640.85 cfs @ 12.42 hrs, Volume= 207.710 af
Outflow = 239.93 cfs @ 15.97 hrs, Volume= 207.322 af, Atten= 63%, Lag= 212.8 min
Primary = 239.93 cfs @ 15.97 hrs, Volume= 207.322 af
Secondary = 0.00 cfs @ 0.00 hrs, Volume= 0.000 af

Routing by Stor-Ind method, Time Span= 0.00-100.00 hrs, dt= 0.20 hrs
Peak Elev= 1,253.30' @ 15.97 hrs Surf.Area= 43.120 ac Storage= 85.447 af

Plug-Flow detention time= 344.3 min calculated for 207.322 af (100% of inflow)
Center-of-Mass det. time= 342.6 min (1,252.5 - 910.0)

Volume	Invert	Avail.Storage	Storage Description
#1	1,251.20'	308.662 af	**Custom Stage Data (Prismatic)** Listed below (Recalc)

Elevation (feet)	Surf.Area (acres)	Inc.Store (acre-feet)	Cum.Store (acre-feet)
1,251.20	38.300	0.000	0.000
1,254.00	44.730	116.242	116.242
1,258.00	51.480	192.420	308.662

Device	Routing	Invert	Outlet Devices
#1	Primary	1,251.20'	**30.0' long x 20.0' breadth Spillway** Head (feet) 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60 Coef. (English) 2.68 2.70 2.70 2.64 2.63 2.64 2.64 2.63
#2	Secondary	1,255.70'	**775.0' long x 14.0' breadth Broad-Crested Rectangular Weir** Head (feet) 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60 Coef. (English) 2.64 2.67 2.70 2.65 2.64 2.65 2.65 2.63

Primary OutFlow Max=239.86 cfs @ 15.97 hrs HW=1,253.30' (Free Discharge)
└─**1=Spillway** (Weir Controls 239.86 cfs @ 3.81 fps)

Secondary OutFlow Max=0.00 cfs @ 0.00 hrs HW=1,251.20' (Free Discharge)
└─**2=Broad-Crested Rectangular Weir** (Controls 0.00 cfs)

Summary for Pond 11P: Ashfield Lake Existing No Route 112

Inflow Area = 684.211 ac, 5.96% Impervious, Inflow Depth = 3.64" for 100-yr event
Inflow = 1,237.83 cfs @ 12.52 hrs, Volume= 207.710 af
Outflow = 297.12 cfs @ 13.72 hrs, Volume= 207.343 af, Atten= 76%, Lag= 72.0 min
Primary = 297.12 cfs @ 13.72 hrs, Volume= 207.343 af
Secondary = 0.00 cfs @ 0.00 hrs, Volume= 0.000 af

Routing by Stor-Ind method, Time Span= 0.00-100.00 hrs, dt= 0.20 hrs
Peak Elev= 1,253.62' @ 13.72 hrs Surf.Area= 43.858 ac Storage= 99.426 af

Plug-Flow detention time= 330.1 min calculated for 206.929 af (100% of inflow)
Center-of-Mass det. time= 336.4 min (1,190.2 - 853.8)

Volume	Invert	Avail.Storage	Storage Description
#1	1,251.20'	308.662 af	**Custom Stage Data (Prismatic)** Listed below (Recalc)

Elevation (feet)	Surf.Area (acres)	Inc.Store (acre-feet)	Cum.Store (acre-feet)
1,251.20	38.300	0.000	0.000
1,254.00	44.730	116.242	116.242
1,258.00	51.480	192.420	308.662

Device	Routing	Invert	Outlet Devices
#1	Primary	1,251.20'	**30.0' long x 20.0' breadth Spillway** Head (feet) 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60 Coef. (English) 2.68 2.70 2.70 2.64 2.63 2.64 2.64 2.63
#2	Secondary	1,255.70'	**775.0' long x 14.0' breadth Broad-Crested Rectangular Weir** Head (feet) 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60 Coef. (English) 2.64 2.67 2.70 2.65 2.64 2.65 2.65 2.63

Primary OutFlow Max=296.78 cfs @ 13.72 hrs HW=1,253.62' (Free Discharge)
└—**1=Spillway** (Weir Controls 296.78 cfs @ 4.09 fps)

Secondary OutFlow Max=0.00 cfs @ 0.00 hrs HW=1,251.20' (Free Discharge)
└—**2=Broad-Crested Rectangular Weir** (Controls 0.00 cfs)

Summary for Pond 27P: Ashfield Lake Proposed No Route 112

Inflow Area = 684.211 ac, 5.96% Impervious, Inflow Depth = 3.64" for 100-yr event
Inflow = 1,237.83 cfs @ 12.52 hrs, Volume= 207.710 af
Outflow = 297.12 cfs @ 13.72 hrs, Volume= 207.343 af, Atten= 76%, Lag= 72.0 min
Primary = 297.12 cfs @ 13.72 hrs, Volume= 207.343 af

Routing by Stor-Ind method, Time Span= 0.00-100.00 hrs, dt= 0.20 hrs
Peak Elev= 1,253.62' @ 13.72 hrs Surf.Area= 43.858 ac Storage= 99.426 af

Plug-Flow detention time= 330.1 min calculated for 206.929 af (100% of inflow)
Center-of-Mass det. time= 336.4 min (1,190.2 - 853.8)

Volume	Invert	Avail.Storage	Storage Description
#1	1,251.20'	308.662 af	**Custom Stage Data (Prismatic)** Listed below (Recalc)

Elevation (feet)	Surf.Area (acres)	Inc.Store (acre-feet)	Cum.Store (acre-feet)
1,251.20	38.300	0.000	0.000
1,254.00	44.730	116.242	116.242
1,258.00	51.480	192.420	308.662

Device	Routing	Invert	Outlet Devices
#1	Primary	1,251.20'	**30.0' long x 20.0' breadth Spillway** Head (feet) 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60 Coef. (English) 2.68 2.70 2.70 2.64 2.63 2.64 2.64 2.63

Primary OutFlow Max=296.78 cfs @ 13.72 hrs HW=1,253.62' (Free Discharge)
└—**1=Spillway** (Weir Controls 296.78 cfs @ 4.09 fps)

Summary for Pond 5P: Ashfield Lake Existing

Inflow Area = 684.211 ac, 5.96% Impervious, Inflow Depth = 6.48" for 500-yr event
Inflow = 1,133.57 cfs @ 12.38 hrs, Volume= 369.220 af
Outflow = 408.32 cfs @ 15.98 hrs, Volume= 368.795 af, Atten= 64%, Lag= 215.8 min
Primary = 408.32 cfs @ 15.98 hrs, Volume= 368.795 af
Secondary = 0.00 cfs @ 0.00 hrs, Volume= 0.000 af

Routing by Stor-Ind method, Time Span= 0.00-100.00 hrs, dt= 0.20 hrs
Peak Elev= 1,254.19' @ 15.98 hrs Surf.Area= 45.054 ac Storage= 124.858 af

Plug-Flow detention time= 284.3 min calculated for 368.795 af (100% of inflow)
Center-of-Mass det. time= 283.2 min (1,202.6 - 919.5)

Volume	Invert	Avail.Storage	Storage Description
#1	1,251.20'	308.662 af	**Custom Stage Data (Prismatic)**Listed below (Recalc)

Elevation (feet)	Surf.Area (acres)	Inc.Store (acre-feet)	Cum.Store (acre-feet)
1,251.20	38.300	0.000	0.000
1,254.00	44.730	116.242	116.242
1,258.00	51.480	192.420	308.662

Device	Routing	Invert	Outlet Devices
#1	Primary	1,251.20'	**30.0' long x 20.0' breadth Spillway** Head (feet) 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60 Coef. (English) 2.68 2.70 2.70 2.64 2.63 2.64 2.64 2.63
#2	Secondary	1,255.70'	**775.0' long x 14.0' breadth Broad-Crested Rectangular Weir** Head (feet) 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60 Coef. (English) 2.64 2.67 2.70 2.65 2.64 2.65 2.65 2.63

Primary OutFlow Max=408.29 cfs @ 15.98 hrs HW=1,254.19' (Free Discharge)
└─**1=Spillway** (Weir Controls 408.29 cfs @ 4.55 fps)

Secondary OutFlow Max=0.00 cfs @ 0.00 hrs HW=1,251.20' (Free Discharge)
└─**2=Broad-Crested Rectangular Weir** (Controls 0.00 cfs)

Summary for Pond 11P: Ashfield Lake Existing No Route 112

Inflow Area = 684.211 ac, 5.96% Impervious, Inflow Depth = 6.48" for 500-yr event
Inflow = 2,285.15 cfs @ 12.49 hrs, Volume= 369.220 af
Outflow = 639.78 cfs @ 13.47 hrs, Volume= 368.824 af, Atten= 72%, Lag= 58.2 min
Primary = 639.78 cfs @ 13.47 hrs, Volume= 368.824 af
Secondary = 0.00 cfs @ 0.00 hrs, Volume= 0.000 af

Routing by Stor-Ind method, Time Span= 0.00-100.00 hrs, dt= 0.20 hrs
Peak Elev= 1,255.24' @ 13.47 hrs Surf.Area= 46.816 ac Storage= 172.824 af

Plug-Flow detention time= 275.9 min calculated for 368.824 af (100% of inflow)
Center-of-Mass det. time= 275.0 min (1,115.2 - 840.2)

Volume	Invert	Avail.Storage	Storage Description
#1	1,251.20'	308.662 af	**Custom Stage Data (Prismatic)** Listed below (Recalc)

Elevation (feet)	Surf.Area (acres)	Inc.Store (acre-feet)	Cum.Store (acre-feet)
1,251.20	38.300	0.000	0.000
1,254.00	44.730	116.242	116.242
1,258.00	51.480	192.420	308.662

Device	Routing	Invert	Outlet Devices
#1	Primary	1,251.20'	**30.0' long x 20.0' breadth Spillway** Head (feet) 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60 Coef. (English) 2.68 2.70 2.70 2.64 2.63 2.64 2.64 2.63
#2	Secondary	1,255.70'	**775.0' long x 14.0' breadth Broad-Crested Rectangular Weir** Head (feet) 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60 Coef. (English) 2.64 2.67 2.70 2.65 2.64 2.65 2.65 2.63

Primary OutFlow Max=638.13 cfs @ 13.47 hrs HW=1,255.23' (Free Discharge)
└─**1=Spillway** (Weir Controls 638.13 cfs @ 5.28 fps)

Secondary OutFlow Max=0.00 cfs @ 0.00 hrs HW=1,251.20' (Free Discharge)
└─**2=Broad-Crested Rectangular Weir** (Controls 0.00 cfs)

Summary for Pond 27P: Ashfield Lake Proposed No Route 112

Inflow Area = 684.211 ac, 5.96% Impervious, Inflow Depth = 6.48" for 500-yr event
Inflow = 2,285.15 cfs @ 12.49 hrs, Volume= 369.220 af
Outflow = 639.78 cfs @ 13.47 hrs, Volume= 368.824 af, Atten= 72%, Lag= 58.2 min
Primary = 639.78 cfs @ 13.47 hrs, Volume= 368.824 af

Routing by Stor-Ind method, Time Span= 0.00-100.00 hrs, dt= 0.20 hrs
Peak Elev= 1,255.24' @ 13.47 hrs Surf.Area= 46.816 ac Storage= 172.824 af

Plug-Flow detention time= 275.9 min calculated for 368.824 af (100% of inflow)
Center-of-Mass det. time= 275.0 min (1,115.2 - 840.2)

Volume	Invert	Avail.Storage	Storage Description
#1	1,251.20'	308.662 af	**Custom Stage Data (Prismatic)** Listed below (Recalc)

Elevation (feet)	Surf.Area (acres)	Inc.Store (acre-feet)	Cum.Store (acre-feet)
1,251.20	38.300	0.000	0.000
1,254.00	44.730	116.242	116.242
1,258.00	51.480	192.420	308.662

Device	Routing	Invert	Outlet Devices
#1	Primary	1,251.20'	**30.0' long x 20.0' breadth Spillway** Head (feet) 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60 Coef. (English) 2.68 2.70 2.70 2.64 2.63 2.64 2.64 2.63

Primary OutFlow Max=638.13 cfs @ 13.47 hrs HW=1,255.23' (Free Discharge)
└─**1=Spillway** (Weir Controls 638.13 cfs @ 5.28 fps)

Appendix C
Turf Reinforcement Mat
Manufacturer Literature

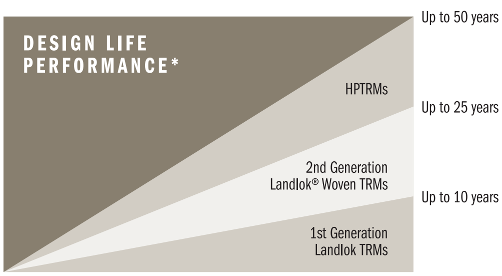
LANDLOK® TURF REINFORCEMENT MATS

Our Landlok® Turf Reinforcement Mats (TRMs) are the industry's most advanced solutions for applications requiring immediate, long-term erosion protection, vegetative reinforcement and water quality enhancement capabilities. Our first generation TRMs are constructed of a dense web of 100% polypropylene fibers positioned between two biaxially oriented nets. When vegetated, they provide twice the erosion protection of vegetation alone.

Now we've taken the same woven technology in our High Performance Turf Reinforcement Mats (HPTRMs) and used it to design the next generation of TRMs. These netless, composite-free three-dimensional second generation TRMs feature a rugged material construction that combines superior tensile strength, flexibility and UV stability. This allows them to deliver better, long-term performance over traditional methods like rock riprap and concrete paving and increased design life over first generation netted, fused, glued or stitch-bonded TRMs. All Landlok TRMs feature our patented X3® fiber technology, which provides 40% greater surface area for trapping and protecting seed and soil.

1ST GENERATION LANDLOK® TRMs FEATURES & BENEFITS

- Provides permanent turf reinforcement to enhance vegetation's natural ability to filter soil particles and prevent soil loss during storm events
- 100% synthetic and UV-stabilized components
- Utilizes X3 fiber technology for up to 40% greater surface area to protect emerging seedlings and sediment retention
- Promotes infiltration which leads to groundwater recharge
- More aesthetically pleasing than conventional methods (i.e. rock riprap and concrete paving)
- Superior product testing and performance
- Easier installation than conventional solutions (no heavy equipment required)



*Design life performance may vary depending upon field conditions and applications.

2ND GENERATION LANDLOK® WOVEN TRMs FEATURES & BENEFITS

All the features and benefits of first generation Landlok TRMs, plus:

- A unique, patented matrix of pyramids formed with X3 fibers that gridlocks soil in place under high-flow conditions
- 3-D woven material with superior tensile strength for loading and/or survivability requirements
- Greater flexibility to maintain intimate contact with subgrade, resulting in rapid seedling emergence and minimal soil loss
- Completely interconnected yarns that provide superior UV resistance throughout the TRM
- A combination of superior characteristics for long-term performance and a longer design life than first generation Landlok TRMs
- Meets requirement of 5 mm^2 or less mesh size to prevent wildlife entanglement in any sensitive habitats

Outperforms and is more cost-effective than conventional erosion control methods, including:

- **Rock riprap**
- **Concrete paving**
- **Erosion Control Blankets (ECBs)**

LANDLOK® TURF REINFORCEMENT MATS PRODUCT FAMILY TABLE

PRODUCT	DESCRIPTION	FUNCTIONAL LONGEVITY	COLOR	FIBER TYPE	# OF NETS	FHWA FP-03, SECTION 713 COMPLIANCE
LANDLOK® 450	1ST GENERATION TRM	PERMANENT	TAN OR GREEN	POLYPROPYLENE X3® FIBER TECHNOLOGY	2	TYPE 5A, 5B, 5C
LANDLOK 1051	1ST GENERATION TRM	PERMANENT	TAN	POLYPROPYLENE X3 FIBER TECHNOLOGY (GEOTEXTILE BACKING)	1	TYPE 5A, 5B, 5C
LANDLOK 300	2ND GENERATION TRM	PERMANENT	TAN OR GREEN	POLYPROPYLENE X3 FIBER TECHNOLOGY	0 (WOVEN)	TYPE 5A, 5B, 5C

PROPEX® | THE ADVANTAGE CREATORS.™
GEOSYNTHETICS

LANDLOK® TURF REINFORCEMENT MATS

APPLICATION SUGGESTIONS FOR LANDLOK® TURF REINFORCEMENT MATS

	APPLICATION	FUNCTIONAL LONGEVITY	PRODUCT STYLE	INSTALLED COST[1]	ANCHOR SUGGESTIONS[5]
SLOPES[2]	UP TO 1H:1V	PERMANENT	LANDLOK® 300	\$10.00 - 15.00/yd^2 \$11.96 - 17.94/m^2	2.5 ANCHORS/yd^2 3 ANCHORS/m^2
SLOPES[2]	UP TO 1.5H:1V UP TO 2H:1V	PERMANENT	LANDLOK 450	\$9.00 - 14.00/yd^2 \$10.77 - 16.75/m^2	2 ANCHORS/yd^2 2.5 ANCHORS/m^2
CHANNELS[3]	SHEAR STRESS UP TO 10 lb/ft^2 (479 N/m^2) VELOCITY UP TO 18 ft/sec (5.5 m/sec)	PERMANENT	LANDLOK 450	\$9.00 - 14.00/yd^2 \$10.77 - 16.75/m^2	2.5 ANCHORS/yd^2 3 ANCHORS/m^2
CHANNELS[3]	SHEAR STRESS UP TO 12 lb/ft^2 (576 N/m^2) VELOCITY UP TO 20 ft/sec (6.1 m/sec)	PERMANENT	LANDLOK 300	\$10.00 - 15.00/yd^2 \$11.96 - 17.94/m^2	2.5 ANCHORS/yd^2 3 ANCHORS/m^2
BANKS[4]	WAVE ACTION < 1 ft (30 cm)	PERMANENT	LANDLOK 1051	\$10.00 - 15.00/yd^2 \$11.96 - 17.94/m^2	2.5 ANCHORS/yd^2 3 ANCHORS/m^2

NOTES: 1. Installed cost estimates range from large to small projects according to material quantity. The estimates include material, seed, labor and equipment. Note that costs vary greatly in different regions of the country. 2. For slopes steeper than 1H:1V, please see our Pyramat® HPTRM product brochure. 3. Values shown are short-term fully vegetated maximums. For channels with a shear stress greater than 12 lb/ft^2 (576 N/m^2) and velocity greater than 20 ft/sec (6.1 m/sec), please see our Pyramat HPTRM product brochure. 4. For wave action greater than 1 ft (30 cm), please see our Pyramat HPTRM product brochure. 5. For anchor size and style, please see our TRM Installation Guidelines.

KEY PHYSICAL PROPERTIES OF LANDLOK® TURF REINFORCEMENT MATS

- Tensile Strength: High-strength and low-strain minimizes seed, root damage and material under heavy loads.
- Flexibility: Greater flexibility allows our TRMs to conform and maintain intimate contact with the prepared grade, increasing the ease of successful installation.
- Seedling Emergence: Landlok TRMs, now with X3® fiber technology, offer 40% more fiber surface area to capture the critical sediment and moisture needed to increase seed germination within the first 21 days.
- UV Resistance: All Landlok TRM components are constructed with the top-tested UV stabilizers, such as carbon black and hindered amine light stabilizers (HALS).

SEVEN STEPS FOR SUCCESSFUL TRM SELECTIONS*

1	2	3	4	5	6	7
SELECT APPLICATIONS	DETERMINE FUNCTIONAL LONGEVITY	ANTICIPATE CLIMATE (ARID, SEMI-ARID OR TEMPERATE)	UNDERSTAND TRADITIONAL SOLUTION	PREDICT NON-HYDRAULIC STRESSES (MAINTENANCE STRESSES)	KNOW VEGETATION TYPE	CALCULATE HYDRAULIC STRESSES

*See Propex Engineering Bulletin or EC-DESIGN® software for more information.

LANDLOK® TURF REINFORCEMENT MAT PROPERTY TABLE[1] ENGLISH & METRIC UNITS

	PROPERTY	TEST METHOD	VALUE[2]	LANDLOK® 450	LANDLOK® 1051	LANDLOK® 300
PHYSICAL	MASS PER UNIT AREA	ASTM D-6566	MARV	10.0 oz/yd^2 340 g/m^2	14 oz/yd^2 475 g/m^2	8.3 oz/yd^2 281 g/m^2
PHYSICAL	THICKNESS	ASTM D-6525	MARV	0.4 in 10.1 mm	0.4 in 10.1 mm	0.3 in 7.6 mm
PHYSICAL	LIGHT PENETRATION	ASTM D-6567	TYPICAL	20%	5%	50%
PHYSICAL	COLOR	VISUAL	–	GREEN, TAN	TAN	GREEN, TAN
MECHANICAL	TENSILE STRENGTH	ASTM D-6818	MARV	400 x 300 lb/ft 5.8 x 4.3 kN/m	300 x 225 lb/ft 4.3 x 3.2 kN/m	2400 x 2000 lb/ft 35.0 x 29.2 kN/m
MECHANICAL	TENSILE ELONGATION	ASTM D-6818	MAXIMUM	50%	85%	50%
MECHANICAL	RESILIENCY	ASTM D-6524	MARV	90%	80%	75%
MECHANICAL	FLEXIBILITY	ASTM D-6575	TYPICAL	0.026 in-lbs 30000 mg-cm	0.022 in-lbs 25000 mg-cm	0.195 in-lbs 225000 mg-cm
ENDURANCE	FUNCTIONAL LONGEVITY	OBSERVED	TYPICAL	PERMANENT	PERMANENT	PERMANENT
DURABILITY	UV RESISTANCE	ASTM D-4355	MINIMUM	80% @ 1000 HOURS	80% @ 1000 HOURS	90% @ 3000 HOURS
PERFORMANCE	SEEDLING EMERGENCE[3]	ECTC DRAFT METHOD #4	TYPICAL	409%	220%	296%
PACKAGING	ROLL WIDTH	MEASURED	TYPICAL	6.5 ft 2.0 m	6.5 ft 2.0 m	8.5 ft 2.6 m
PACKAGING	ROLL LENGTH	MEASURED	TYPICAL	138.5 ft 42.2 m	138.5 ft 42.2 m	106 ft 32.3 m
PACKAGING	ROLL WEIGHT	CALCULATED	TYPICAL	75 lb 34 kg	101 lb 46 kg	51 lb 23 kg
PACKAGING	ROLL AREA	MEASURED	TYPICAL	100 yd^2 84 m^2	100 yd^2 84 m^2	100 yd^2 84 m^2

NOTES: 1. The listed property values are effective 06/2009 and are subject to change without notice. 2. MARV indicates Minimum Average Roll Value calculated as the typical minus two standard deviations. Statistically, it yields a 97.7% degree of confidence that any sample taken during quality assurance testing will exceed the reported value. 3. Calculated as percent increase in average plant biomass with tall fescue grass seed in sand 14 days after seeding versus traditional monofilament TRMs and HPTRMs.

LANDLOK® TURF REINFORCEMENT MAT PERFORMANCE VALUES ENGLISH & METRIC UNITS

MATERIAL	FUNCTIONAL LONGEVITY	SHORT-TERM MAXIMUM SHEAR STRESS AND VELOCITY						MANNING'S "n"		
		VEGETATED[4, 7]		PARTIALLY[5]		UNVEGETATED[6]		0"-6"	6"-12"	12"-24"
LANDLOK® 450	PERMANENT	10 lb/ft^2 479 N/m^2	18 ft/sec 5.5 m/sec	8 lb/ft^2 383 N/m^2	15 ft/sec 4.6 m/sec	5 lb/ft^2 239 N/m^2	12 ft/sec 3.7 m/sec	0.035	0.025	0.021
LANDLOK 1051	PERMANENT	10 lb/ft^2 479 N/m^2	18 ft/sec 5.5 m/sec	n/a	n/a	5 lb/ft^2 239 N/m^2	12 ft/sec 3.7 m/sec	0.036	0.026	0.020
LANDLOK 300	PERMANENT	12 lb/ft^2 576 N/m^2	20 ft/sec 6.1 m/sec	–	–	–	–	0.030	0.028	0.018

NOTES: 4. Maximum permissible shear stress has been obtained through fully vegetated (70% to 100% density) testing programs featuring specific soil types, vegetation classes, flow conditions and failure criteria. These conditions may not be relevant to every project nor are they replicated by other manufacturers. Please contact Propex for further information. 5. Maximum permissible shear stress has been obtained through partially vegetated (30% to 70% density) testing programs featuring specific soil types, vegetation classes, flow conditions and failure criteria. These conditions may not be relevant to every project nor are they replicated by other manufacturers. Please contact Propex for further information. 6. Maximum permissible shear stress has been obtained through unvegetated (0% to 30% density) testing programs featuring specific soil types, vegetation classes, flow conditions and failure criteria. These conditions may not be relevant to every project nor are they replicated by other manufacturers. Please contact Propex for further information. 7. Maximum permissible shear stress achieved after only 14 weeks of vegetative establishment versus the industry standard of two full growing seasons.

For downloadable documents like construction specifications, installation guidelines, case studies and other technical information, please visit our web site at **geotextile.com.** These documents are available in easy-to-use Microsoft® Word format.

PROPEX® | THE ADVANTAGE CREATORS.™

GEOSYNTHETICS

Propex Operating Company,LLC
6025 Lee Highway, Suite 425
PO Box 22788
Chattanooga, TN 37422

PH: 423 899 0444
PH: 800 621 1273
FAX: 423 899 7619
www.geotextile.com