# APPENDIX G:

# Conceptual Designs for High Priority Stormwater Mitigation Sites

**Project LY-13: Extended Detention Basin Retrofit**

Site Description

Project LY-13 is located at an existing dry detention basin constructed in a forested area north of the intersection of Chase Street and High Street. The basin outlet consists of two (2) 24-inch grates seated on concrete risers. The outlet structure tie in to a complex drainage network that accumulates runoff through the Village center and enter a single outlet pipe near the North Country Federal Credit Union on Center Street. The outflow pipe empties directly in to the Passumpsic River. The basin receives runoff from three (3) stormwater pipes draining relatively steep areas of low to medium density residential development (Figure 1). The existing dry detention basin has filled with sediment and provides minimal storage volume for runoff (Figures 2 – 3). It is likely that significant volumes of sediment area carried out of the BMP during runoff events. A field assessment of the contributing drainage areas did not indicate any significant erosion areas or other sediment sources within the neighborhoods, and the Town does not apply sand within the subwatersheds. We suspect that most of the sediment is from erosion along the flow paths within the forested area, particularly from the recently re-stabilized culvert outlet draining Chase Street (Figures 4 – 5).

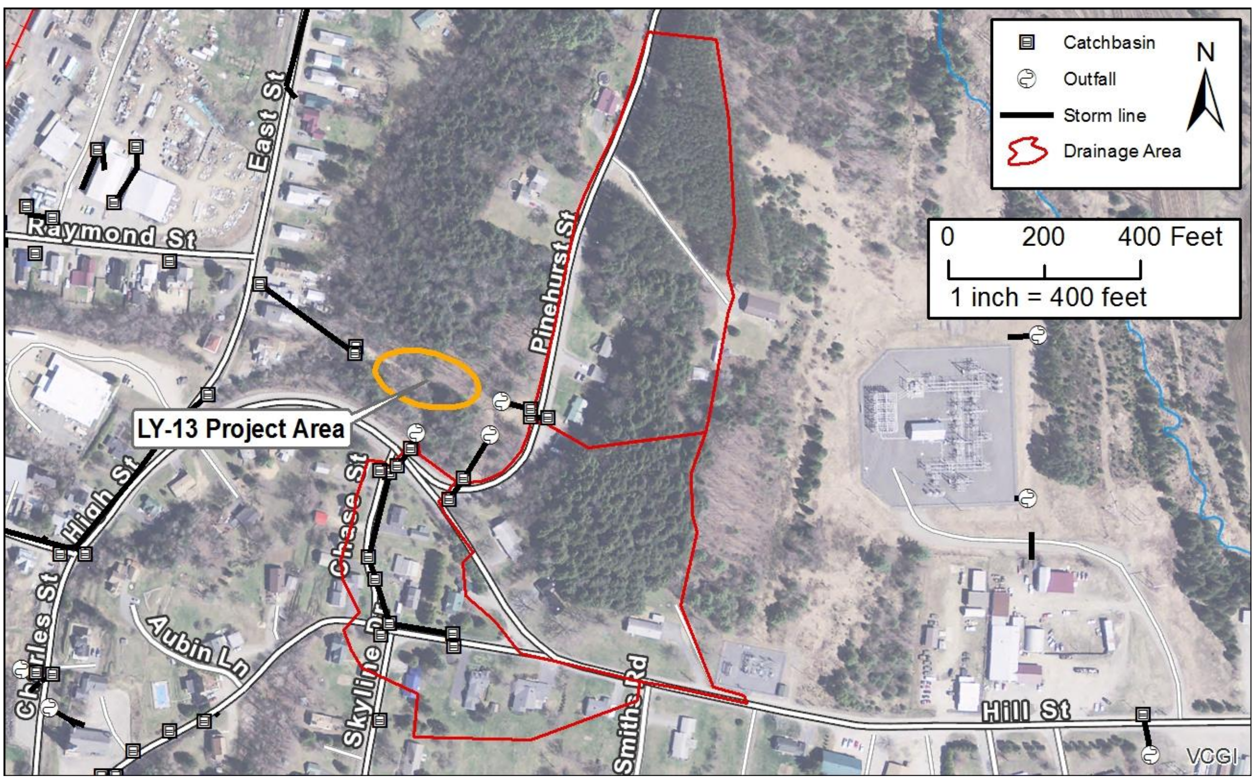



**Figure 1:** Site Location Map for Project LY-13

**Figure 2:** Existing outlet structure – two 24" square grates partially obstructed with debris

**Figure 3:** Sediment deposits filling the pond to the outlet structure elevation

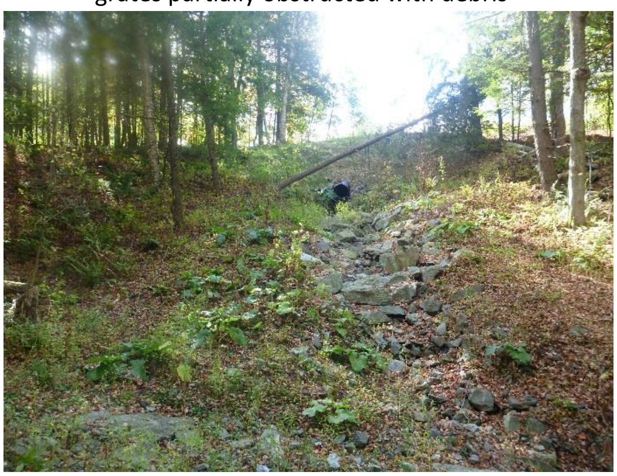


**Figure 4:** Heavily armored flow path from the Chase St. culvert outlet

**Figure 5:** Active erosion in the channel carrying runoff from the Pinehurst and High St culverts

Field Survey

Field verification of drainage areas was completed in late September 2017. VTDEC stormwater mapping drainage areas were edited based on observations of topography, road grade, and curb locations. Field survey of the detention basin was completed at the same time. FEA surveyed a long profile (Section A) including the inlet channel downstream of the confluence of the three subcatchments, the detention basin bottom, the berm west of the outlet structures, and the parking area for the house west of the berm as well as two cross sections of the detention basin and one cross section at the inlet of the detention basin. The survey elevations are relative to an elevation datum established in the field at the higher of the two outlet grates.

Drainage Area Analysis

We constructed a TR-20 hydrologic model for each of the three (3) subcatchments draining to the detention basin and the existing detention basin using HydroCAD 10.0 software. This model estimates flow rates and volume following simulated recurrence-interval rainfall events. The watershed was characterized by land cover, soils, and topography to estimate runoff volumes and peak flow rates. Land cover composition of the three subcatchments is shown in Table 1. All soils in the subcatchments were classified as hydrologic soil group A. 24-hour rainfall depths for the recurrence interval storms are shown in Table 2 and were estimated using the Extreme Precipitation in New York and New England web tool created by the Northeast Regional Climate Center and the Natural Resources Conservation Service (http://precip.eas.cornell.edu/).

**Table 1:** Land cover (acres) for subcatchments draining to the detention basin.

| Land Cover | Chase Street Catchment | High Street Catchment | Pinehurst Street Catchment | Combined Subcatchments |
|---|---|---|---|---|
| Forest | 0.2 | 2.65 | 4.25 | 7.1 |
| Paved Parking & Roadway | 0.99 | 0.86 | 0.78 | 2.63 |
| Rooftops | 0.45 | 0.32 | 0.08 | 0.85 |
| Developed (>75% Grass Cover) | 2.3 | 1.2 | 0.65 | 4.15 |

**Table 2:** Rainfall depth (inches), inflow discharge estimates (cubic feet per second), and inflow volume estimates (acre-feet) for select recurrence interval storms at the detention basin.

| Return Interval | Rainfall Depth (in) | Inflow Discharge to Detention Basin (cfs) | Inflow Volume to Detention Basin (af) |
|---|---|---|---|
| WQv* | 1 | 4.74 | 0.229 |
| 1 | 1.98 | 10.07 | 0.509 |
| 2 | 2.31 | 11.85 | 0.604 |
| 10 | 3.26 | 16.95 | 0.878 |
| 100 | 5.35 | 28.16 | 1.590 |

* Water quality volume (WQv) is equivalent to the 1" rainstorm.

Dry Detention Basin Retrofit Design

The proposed retrofit design includes cleaning out the sediment deposits within the basin, retrofitting the outlet structure, and installation of a sediment forebay (see Sheets 1 & 2). These improvements will significantly reduce maintenance requirements and will provide enough storage volume to store and likely infiltrate the WQv 1-inch rainfall event. We recommend 3-4 days of construction oversight to assist with BMP layout, grading, and outlet structure configuration.

**Basin Cleanout**

Based on conversations with the Lyndon Town Highway Supervisor (Joe Dauphin) the basin has accumulated approximately 2 feet of sediment since the last cleanout in 2015. The current sediment elevation is equal to the lower outlet grate, providing minimal ponding volume and sediment storage. Removal of the sediment deposits will increase basin storage by approximately 3,000 cubic-feet.

**Outlet Structure**

Retrofitting the existing outlet structures will further increase storage and will reduce the potential for outlet blockage from debris. We recommend replacing each of the 24" grate with 18" diameter riser pipes that will be set to an elevation 1-foot higher than the upper grate (see Sheet 3 outlet detail). This will further increase basin storage volume by an additional 2,000 cubic feet. Two low flow orifices are included in the design to reduce the duration of ponding following storm events. These orifices are 2" holes cored in to the existing concrete riser, and will require a trash/debris screen to prevent clogging. **The low flow orifices may not be necessary depending on soil infiltration rates.**

**Sediment Forebay**

A sediment forebay constructed upslope (east) of the existing basin will facilitate sediment maintenance and improve long-term basin performance. This feature will require excavation into the existing banks and the construction of a large berm between the forebay and the existing basin. The forebay is designed to store 1,800 cubic feet, approximately 15% of the WQv. The sediment forebay will be located adjacent to the existing access road to facilitate maintenance. **The sediment forebay should be inspected annually and cleaned as needed.** A fixed vertical sediment depth marker will be installed in the forebay to inform cleanout requirements.

**Inflow Channel Stabilization**

The existing channels carrying runoff from the culvert outlets to the basin are unstable and have several areas with active erosion. The channels should be excavated and lined with 12” minus stone. Construction of the sediment forebay will require reconfiguration of approximately 40’ of the inflow channel. As shown in Sheet 2, we recommend constructed steps to stabilize the steepened channel.

**WQv Treatment**

The BMP retrofit results in a reduction in peak flow during the WQv storm from 4.33 cfs to 0.33 cfs (Fig. 6). The center of mass detention time increases from 5 minutes to 176 minutes. The water surface elevation of the detention basin is raised from 819.7’ to 822.22’. We assumed a conservative infiltration rate of 2”/hour. If the actual infiltration rates are found to be higher, the low flow orifices can be reduced or removed. The pond storage volume below the primary outlet standpipes is greater than the WQv, potentially allowing for complete infiltration of the 1-inch storm.

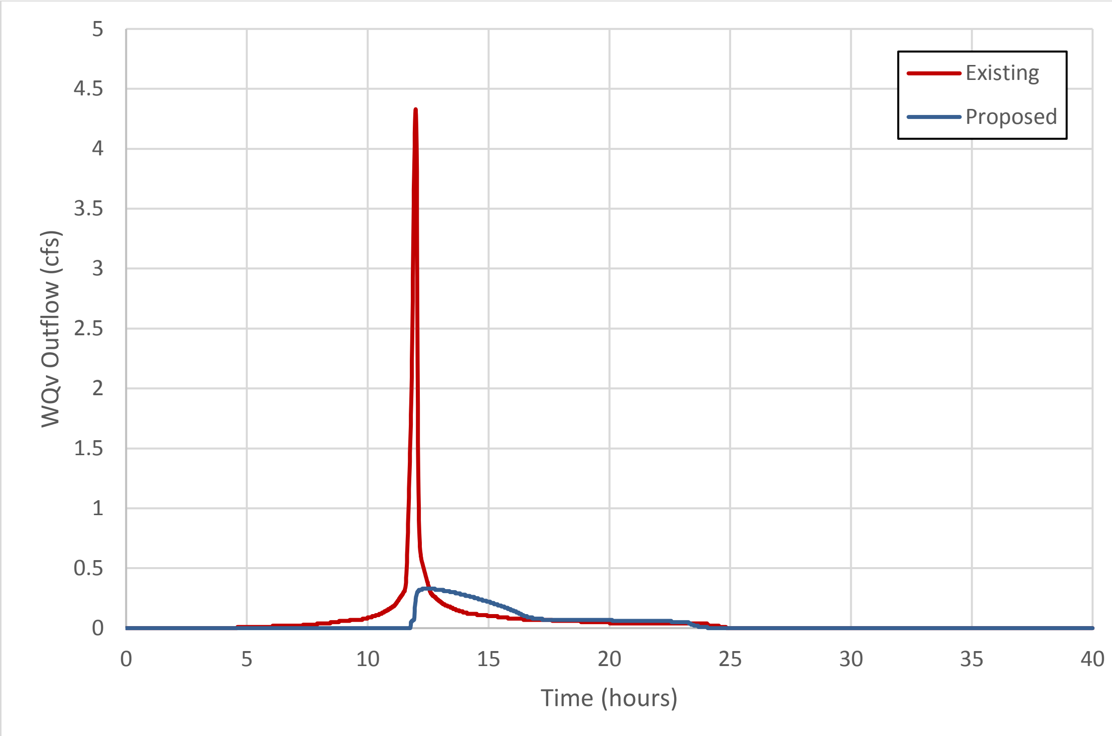



**Figure 6:** Flow hydrographs for the WQv storm at the outflow of the detention basin.

**100-year Storm Passage**

The BMP retrofit does not result in a peakflow reduction during the 100-year storm (Fig. 7). The water surface elevation of the detention basin in the retrofit scenario is raised from 821.97' to 822.64'. The low point (elevation 825.96') of the berm is 3.32' higher than this flood elevation. The water surface elevation in the 100-year storm is only slightly higher (0.16') than the berm in the sediment forebay.

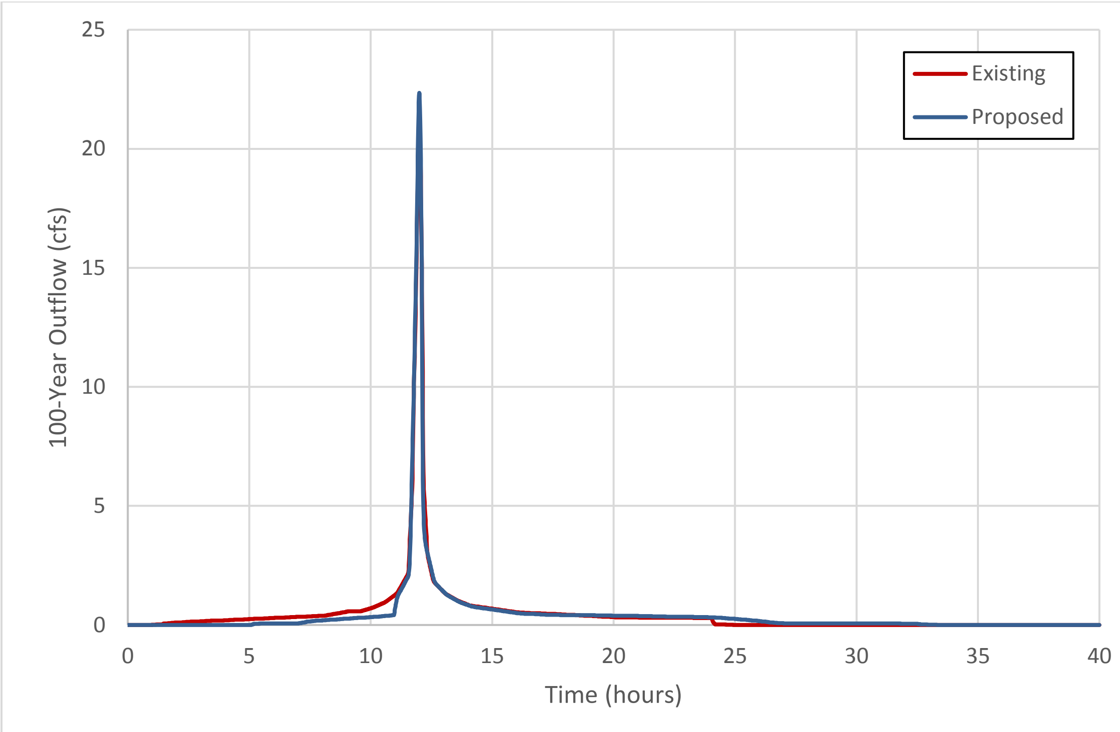



**Figure 7:** Flow hydrographs for the 100-year storm at the outflow of the detention basin.

Next Steps

- The proposed BMP retrofit will result in increased water surface elevations and total volume of water stored in the detention basin during storms. Therefore, structural assessment of berm is needed in later design phases to assess the risk to the residential area to the west.
- Determining the infiltration capacity of underlying soils is a critical design element required for finalizing BMP plans. Infiltration rates should be tested at the proposed pond bottom elevation in multiple locations within the project area.
- The location and size of utilities need to be determined for later design phases to parameterize plans and elevations.
- Further work is needed to characterize the type and amount of material needed to stabilize the inflow channels.

Preliminary Cost Opinion

**Table 3:** Preliminary cost estimate for the LY-13 project.

| Item/Task | Quantity | Unit | Cost/Unit | Total Cost |
|---|---|---|---|---|
| Site/Berm Survey | 1 | L.S. | N/A | $ 2,000 |
| Infiltration Testing | 1 | L.S. | N/A | $ 1,000 |
| Final Design | 1 | L.S. | N/A | $ 5,000 |
| Equipment Mobilization | 1 | L.S. | N/A | $ 1,000 |
| Construction Oversight | 1 | L.S. | N/A | $ 2,500 |
| Common Excavation | 275 | CY | $ 15 | $ 4,125 |
| Berm Fill Installation | 50 | CY | $ 20 | $ 1,000 |
| Stone Armor Installation | 130 | CY | $ 50 | $ 6,500 |
| Trucking | 15 | Hours | $ 100 | $ 1,500 |
| Outlet Structure Retrofit | 2 | Each | $ 2,500 | $ 5,000 |
| | | | **Total Cost** | **$ 29,625** |

Landowner Support

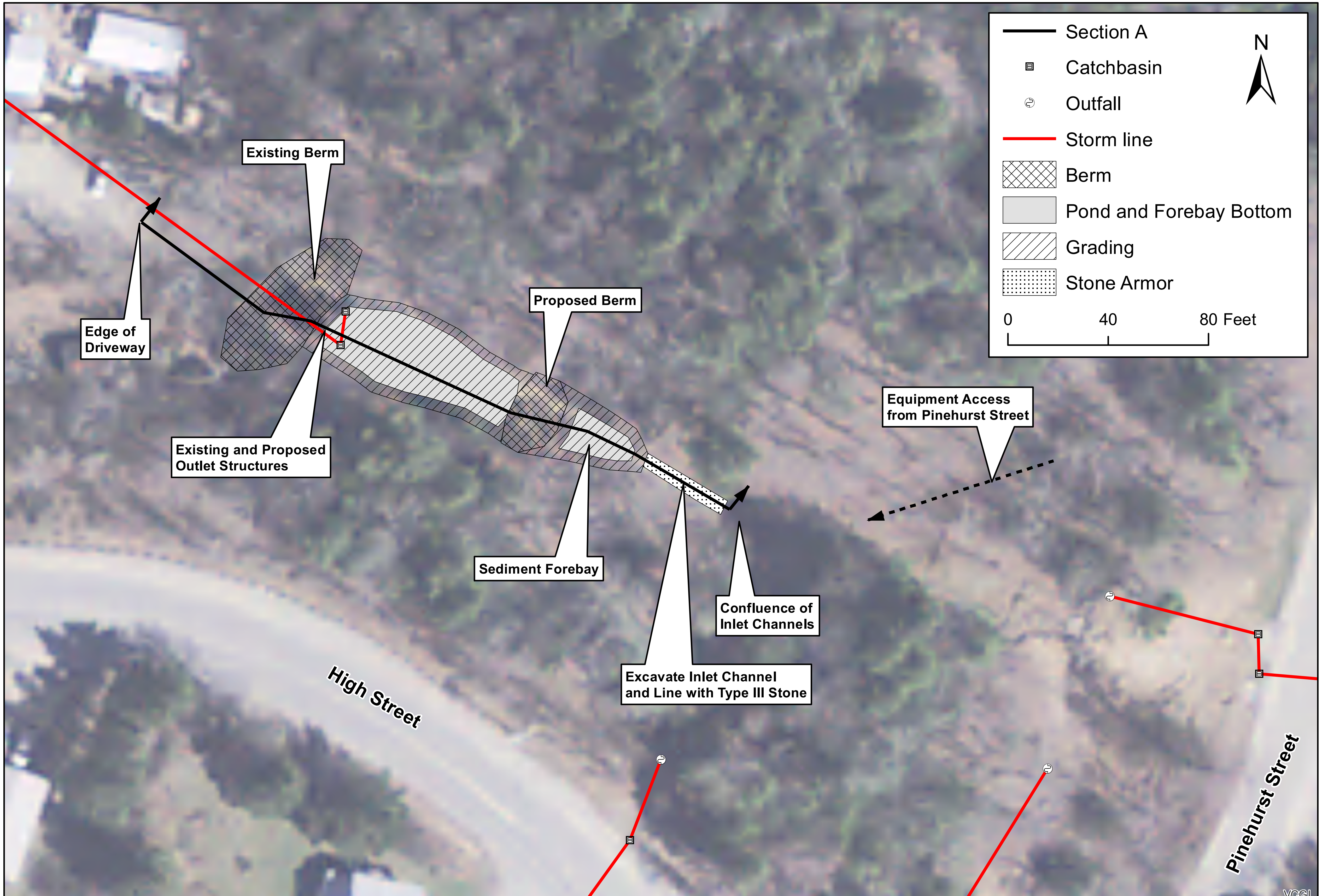
The project has been discussed with the Village, which has expressed support for the project and for the required ongoing maintenance. CCNRCD will continue outreach with adjacent landowners to help advance the project. The public works supervisor, Joe Dauphin, toured the site with Joe Bartlett and Evelyn Boardman of FEA during the concept design survey. Joe Dauphin provided valuable input on drainage basin characteristics, history of the treatment feature, and the location of existing project constraints.

Project Implementation and Funding Partners

VTANR ERP is a potential funding partner for this project site. The CCNRCD would likely be available to assist with grant applications and coordination of funding.

Permitting Requirements

We do not anticipate any permitting requirements for this project.




Notes:
Imagery from VCGI (2014)

Map By: EHB / JHB
Date: November 16, 2017
Scale: 1 inch = 40 feet

**Lyndon Stormwater Master Plan**
**Project LY-13**
**Lyndon, Vermont**

**SHEET 1 - 30% Concept Design Plan**

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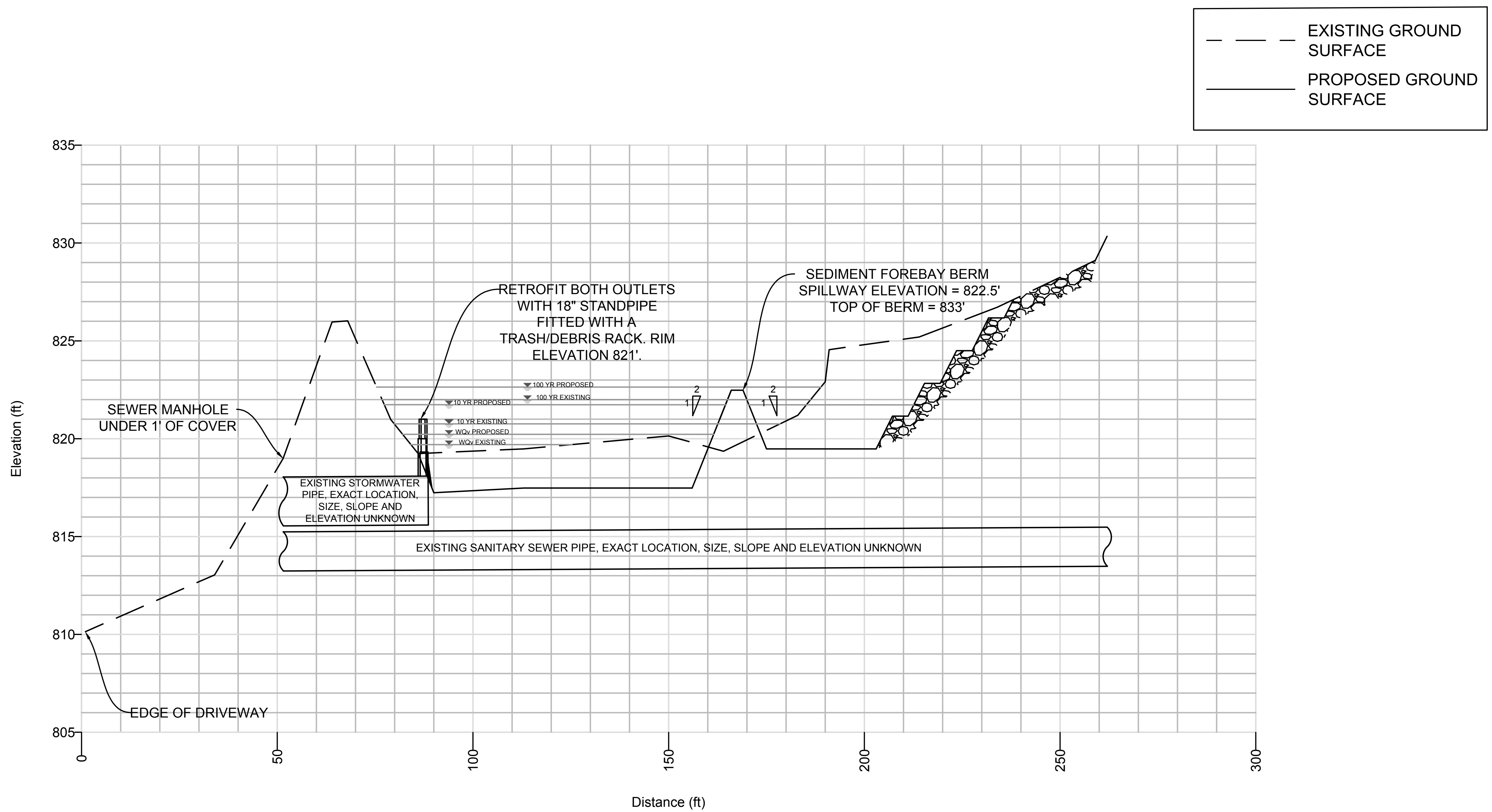



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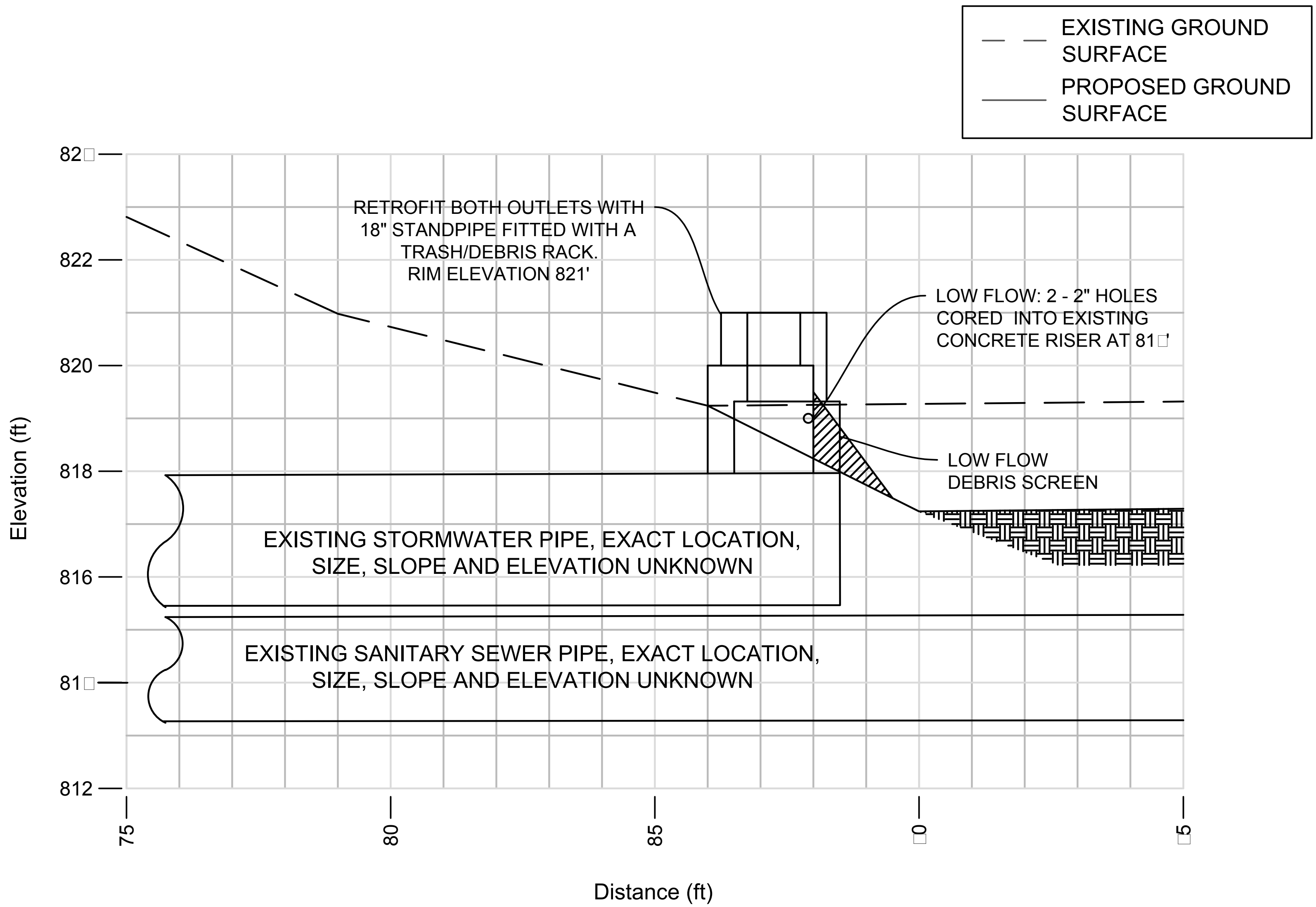
Lyndon Stormwater Master Plan
Project LY-13
30% Concept Design Plan
Lyndon, Vermont

Vertical Scale: 1" = 6'
Horizontal Scale: 1" = 30'
November 13, 2017
Drawn: KS & EHB

Notes:
- Survey elevation datum established at the higher of the two outlet grates and assumed to be 820 feet.
- Location and size of utilities will be identified in later design phases.

SHEET 2
Section A
Long Profile

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# Lyndon Stormwater Master Plan
# Project LY-13
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# Lyndon, Vermont
# SHEET 3: Section A Outlet Detail

Scale: 1" = 3'

November 13, 2017

Drawn: KS & EHB

Notes:
- Survey elevation datum established at the higher of the two outlet grates and assumed to be 820 feet.
- Location and size of utilities will be identified in later design phases.

**Project LSC-7: Gravel Wetland**

<u>Site Description</u>

Project LSC-7 is a proposed stormwater treatment system located in the grassed area south of the Student Services Building (The Brown House). The underlying soils for the subwatershed are primarily fill taken from the hillslope to the west. Descriptions of the area following rain events provided by LSC staff suggest that the fill soils are heavy and poorly drained. The treatment system will be located downslope of the 18" CMP culvert under Vail Drive. The culvert collects runoff from portions of Vail Drive, approximately half of the staff parking lot, and several small subwatersheds draining portions of the baseball fields and the adjacent grassed areas (Figures 8-10). The culvert outlet drains to a wide grassed swale with no defined flow path, eventually reaching the wetland area and a small stream.

- Subwatershed 1 empties in to a catchbasin along College Road and is then piped under Subwatershed 2 to an outlet on the slope down to the parking lot.
- Subwatershed 2 drains to a small rain garden installed in 2017 (Project LSC-6) which has an overflow onto the slope.
- Subwatershed 3 sheetflows down the slope and is collected by a small ditch along the edge of the parking lot.
- Subwatershed 4 drains to the parking lot and flows across the lot to Vail Drive.
- Subwatershed 5 flows across the parking lot and through a narrow grassed strip to the ditch along Vail Drive.
- Subwatershed 6 flows to Vail Drive and is collected by the roadside ditch.

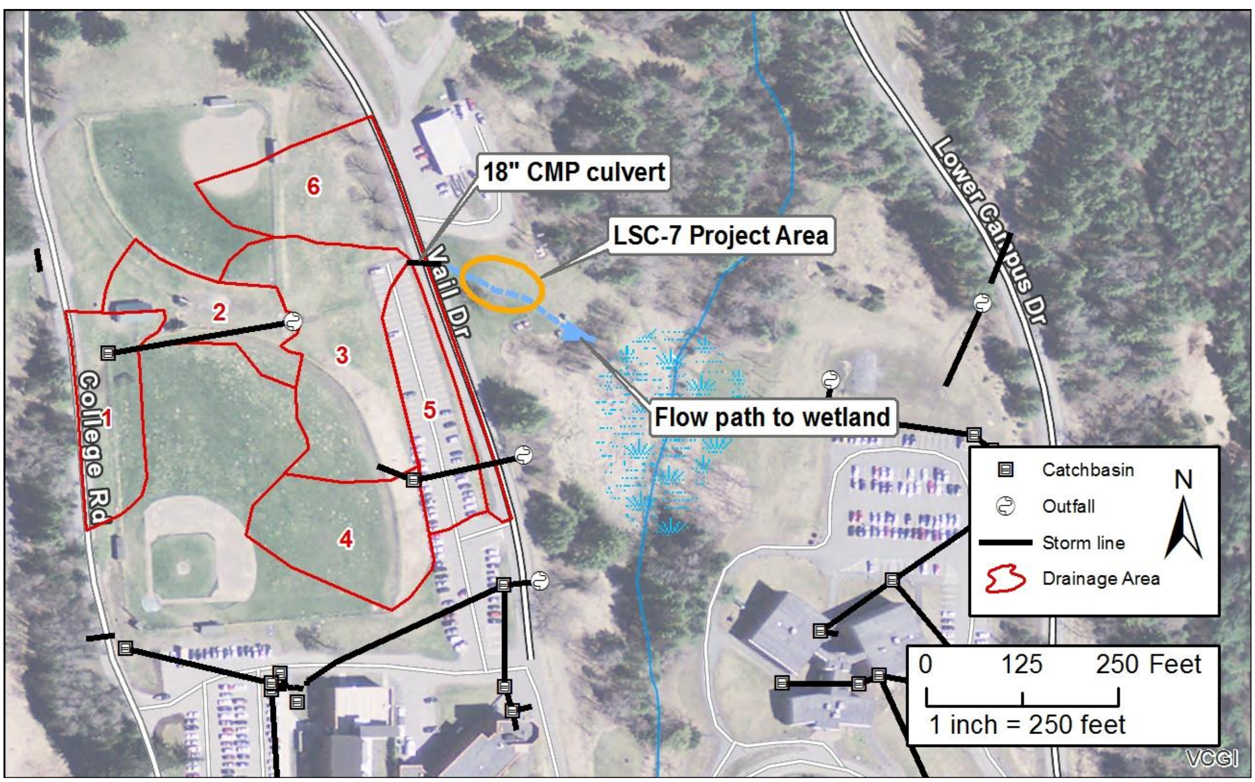



**Figure 8:** Site Location Map for Project LSC-7

**Figure 9:** Ditches along Vail Drive join the small ditch along the staff parking lot at the culvert inlet

**Figure 10:** Erosion at the culvert inlet

Field Survey

Field verification of drainage areas was completed in late September 2017. VTDEC stormwater mapping of drainage areas were edited based on observations of topography and existing ditches. FEA surveyed the parking lot slope and the profile of the existing culvert. The survey elevations are relative to an elevation datum established in the field on a concrete transformer pad near the proposed treatment BMP.

Drainage Area Analysis

We constructed a TR-20 hydrologic model for each of the six (6) subcatchments draining to the culvert under Vail Drive using HydroCAD 10.0 software. This model estimates flow rates and volume following simulated recurrence-interval rainfall events. The watershed was characterized by land cover, soils, and topography to estimate runoff volumes and peak flow rates. The variability in land cover and slope produced a wide range for time of concentration (ranging from 70 minutes for Basin 4 to 3 minutes for Basin 5). All soils in the subcatchments were listed as fill soils and did not have a specified hydrogroup. We assumed hydrogroup D for the areas that are maintained (baseball fields) and hydrogroup C for the vegetated slope. 24-hour rainfall depths for the recurrence interval storms are shown in Table 4 and were estimated using the Extreme Precipitation in New York and New England web tool created by the Northeast Regional Climate Center and the Natural Resources Conservation Service (http://precip.eas.cornell.edu/).

**Table 4:** Rainfall depth (inches), inflow discharge estimates (cubic feet per second), and inflow volume estimates (acre-feet) for select recurrence interval storms at the BMP.

| Return Interval | Rainfall Depth (in) | Peak Discharge to Culvert (cfs) | Inflow Volume to Culvert (af) |
|---|---|---|---|
| WQv* | 1 | 0.75 | 0.07 |
| 1 | 1.98 | 1.72 | 0.25 |
| 2 | 2.31 | 2.14 | 0.32 |
| 10 | 3.25 | 3.49 | 0.56 |
| 100 | 5.34 | 6.94 | 1.17 |

* Water quality volume (WQv) is equivalent to the 1” rainstorm.

Site Drainage Improvements

Grass-lined swales will be installed to improve conveyance of runoff to the proposed BMP. The first swale is located along the toe of the steep slope from the baseball field. Currently runoff sheetflows down this slope and then flows across the parking lot to the entrance and onto Vail Drive (subwatershed 4). This concentrated flow is causing erosion and damage to the pavement at the parking lot entrance. Extending the existing swale along the western edge of the parking lot will provide a stabilized flow conveyance to the BMP. Approximately 0.5 acres of the parking lot currently drain to Vail Drive via sheetflow through the narrow grassed strip along the eastern edge of the parking lot (subwatershed 5). This runoff is increasing erosion along the road edge and likely increases road icing issues during the winter months. A grass-lined swale installed along the edge of the parking lot will provide a stable conveyance to the culvert. The existing 18" culvert under Vail Drive is scheduled for replacement in 2018. We recommend upsizing to a 24" culvert.

Gravel Wetland WQv Treatment Design

The proposed BMP design will utilize a pair of connected gravel wetlands to facilitate nutrient removal for the WQv storm (See attached Sheets 1 – 3). Gravel wetlands were selected due to limited infiltration capacity of the underlying soils and the moderate slope in the project area. Gravel wetlands provide an excellent medium for biological uptake of excess nutrients and will be sized to detain 100% of the runoff from the WQv event.

**Sediment Forebay**

Water enters the treatment area through a sediment forebay downstream of the Vail Drive culvert (see Sheets 1 & 2). The proposed forebay is approximately 30' long and 15' wide and lined with stone. The forebay has two outlets. The first outlet is a 6" vertical riser pipe designed to convey the WQv storm into the gravel treatment wetlands. The second outlet is an 18" vertical riser pipe designed to bypass the gravel wetlands and convey larger storms to the stone-stabilized outlet downstream of the wetlands. The permanent pool elevation of the forebay will be set at the WQv inlet riser (2 feet deep). The outlets will be housed in a 48" diameter manhole with multiple side inlet openings fitted with trash/debris screens. The exact size, slope, and location for the overflow pipe will be determined in later design phases.

**Gravel Wetland Cells**

Two sequential gravel wetland cells are filled with 36" of gravel overlain with 3" of aggregate material overlain with 9" of topsoil planted with wetland vegetation (Sheet 3 Gravel Wetland Detail). The proposed gravel wetland cells are approximately 30' x 30' with an impermeable liner. Water from the sediment forebay enters the first gravel wetland cell near the bottom of the gravel layer through a perforated T-pipe, allowing the water to spread through the gravel and pass through the interstitial spaces. The outlet is a perforated T located near the gravel/aggregate interface. An overflow pipe with a trash/debris screen is located above the topsoil and will tie in with the lower outlet pipe. The overflow from the first wetland will enter the second wetland through the same inlet structure as above. The combined capacity of the wetlands will allow for full storage of the WQV storm with a maximum ponding depth of 1-foot above the soil layer.

Next Steps

- Site survey to verify drainage areas and proposed BMP location
- Further site assessment is needed to determine the location and depth of the overflow pipe and swale
- All utilities must be located and incorporated into final designs
- Project must be discussed with LSC prior to the planned 2018 parking lot grading and paving
- Delineate the Class II wetland boundary to the east of the project as necessary to confirm no project impacts on state jurisdictional wetlands.

Preliminary Cost Opinion

**Table 5:** Preliminary cost estimate for the LSC-7 project.

| Item/Task | Quantity | Unit | Cost/Unit | Total Cost |
|---|---|---|---|---|
| Site Survey | 1 | L.S. | N/A | $ 2,000 |
| Final Design | 1 | L.S. | N/A | $ 5,000 |
| Equipment Mobilization | 1 | L.S. | N/A | $ 1,000 |
| Construction Oversight | 1 | L.S. | N/A | $ 2,500 |
| Grass Swale Installation | 500 | LF | $ 10 | $ 5,000 |
| Excavator Operation | 12 | Days | $ 1,000 | $ 12,000 |
| Trucking | 15 | Hours | $ 100 | $ 1,500 |
| Laborer | 18 | Days | $ 320 | $ 5,760 |
| Ammended Soil | 50 | CY | $ 40 | $ 2,000 |
| Aggregate | 20 | CY | $ 25 | $ 500 |
| Gravel | 200 | CY | $ 20 | $ 4,000 |
| Wetland Shrubs (3-gal) | 30 | Each | $ 25 | $ 750 |
| Wetland Plants (plugs) | 120 | Each | $ 5 | $ 600 |
| Stone 12" minus | 40 | CY | $ 50 | $ 2,000 |
| Rubber Liner | 2800 | sqft | $ 1 | $ 3,640 |
| Inlet/Outlet Structures | 1 | L.S. | $ 7,500 | $ 7,500 |
| Overflow Pipe | 1 | L.S. | $ 3,000 | $ 3,000 |
| | | | **Total Cost** | **$ 58,750** |

Landowner Support

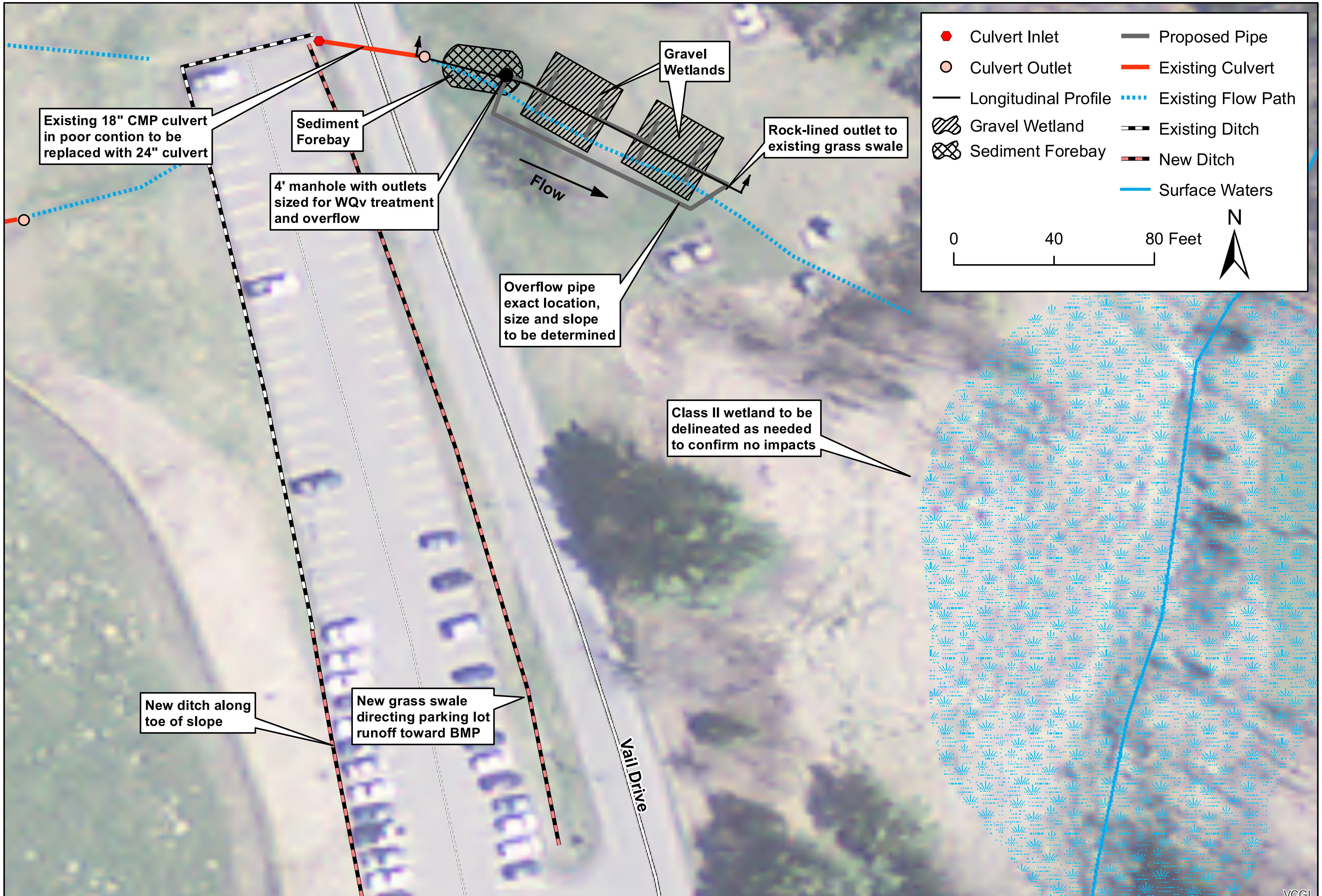
The project has been discussed with Tom Archer, the Lyndon State College Director of Physical Plant. Joe Bartlett and Tom toured the site and discussed the project goals and design. Tom is supportive of the project and is interested in incorporating the drainage improvements into their planned project to regrade and pave the parking lot, scheduled for 2018.

Project Implementation and Funding Partners

VTANR ERP is a potential funding partner for this project site. LSC could be a potential funding and labor partner for the project. The CCNRCD would likely be available to assist with grant applications and coordination of funding.

Permitting Requirements

The grass-swale carrying the overflow may require some stabilization within the 50’ wetland buffer.




**Notes:**
Imagery from VCGI (2014)

**Map By:** EHB / JHB
**Date:** November 16, 2017
**Scale:** 1 inch = 40 feet

**Lyndon Stormwater Master Plan**
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**SHEET 1 - 30% Concept Design Plan**


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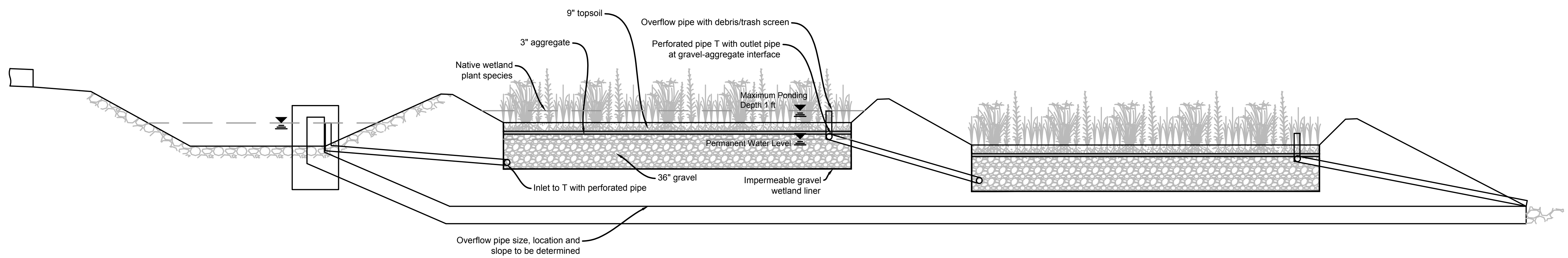


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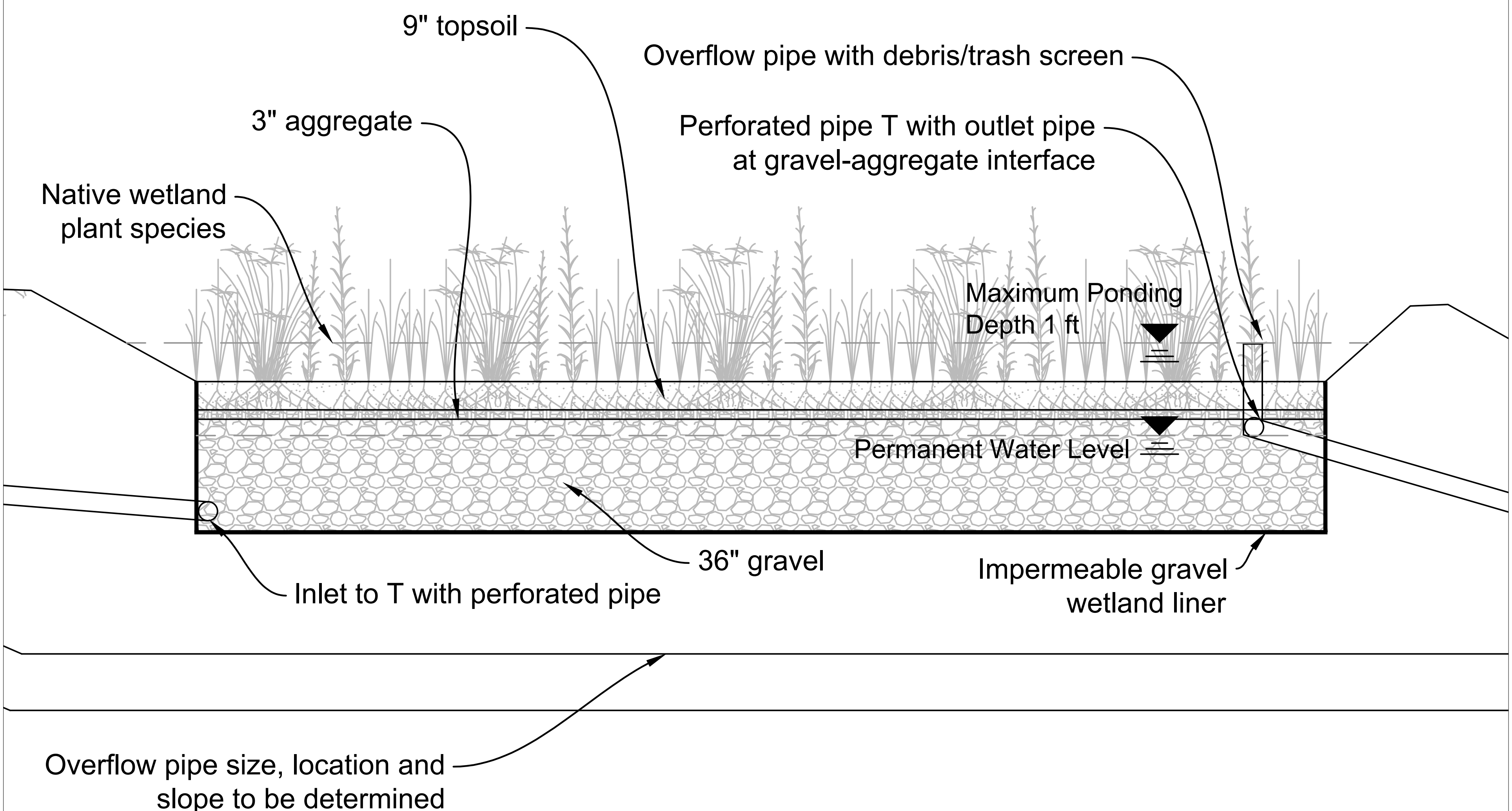
Lyndon Stormwater Master Plan
Project LSC-7
30% Concept Design Plan
Lyndon, VT

Scale: 1" = 10'
November 17, 2017
Drawn: KS & EHB
Approved: JHB & EPF

Notes:
30% conceptual design for planning purposes only. Future survey and design work is required to locate utility constraints, gather topographic data, and size stormwater structures.

SHEET 2
Longitudinal Profile

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# Lyndon Stormwater Master Plan
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# Lyndon, Vermont
# SHEET 3: Gravel Wetland Detail

Scale: 1" = 4'

November 17, 2017

Drawn: KS & EHB

Approved: JHB & EPF

Notes:
30% conceptual design for planning purposes only. Future survey and design work is required to locate utility constraints, gather topographic data, and size stormwater structures.