MWE

Consulting Civil Engineers and Land Surveyors

MetroWest Engineering, Inc.

December 11, 2024

Ms. Mary Grover
MADEP
Wetlands Division
Western Regional Office
463 Dwight Street
Springfield, MA 01103

RE MADEP File Number 246-0785
8 View Avenue, Northampton
SOC Proceedings

Dear Ms. Grover:

As I discussed during the December 4, 2024, information gathering meeting for the above-referenced project, I have been engaged by the abutter's/concerned citizen petitioners to review stormwater-related aspects of the proposed development project. I gave a brief overview of my concerns at the site meeting, and this letter provides more detail and clarification regarding the issues that I discussed. My stormwater-related concerns are as follows:

Watershed Hydrology, Hydrologic Analysis Design Point and Stormwater Standard #2

Hydrologic Analysis Requirements per Stormwater Standard 2

The Massachusetts Stormwater Handbook states (Volume 1, Chapter 1, Standard 2, Page 5) *"Measurement of peak discharge rate is calculated at a design point, typically the lowest point of discharge at the downgradient property boundary. The topography of the site may require evaluation at more than one design point, if flow leaves the property in more than one direction, An applicant may demonstrate that a feature beyond the property boundary (e.g. culvert) is more appropriate as a design point."*

Project Proponent's Selection of Design or Analysis Point

The hydrologic analysis for the project submitted by the proponent's engineer, selects an analysis not at the single low point along the downgradient property boundary, but rather along the entire upland edge of the Bordering Vegetated Wetland. This analysis selection, a linear boundary rather than a point, is removed from the low point of the property and fails to analyze the entire contributing watershed of the subject property as well as upgradient areas that contribute flow *to and through the property.*

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The area of the subject property is 5.25-acres while the watershed that contributes flow to and through the subject property has an area of approximately 60.7-acres (see attached StreamStats Report). The proponent's hydrologic analysis considered only the upland 3.56-acres of the subject property. The analysis or design point selection fails to satisfy the requirements of Standard 2, and the analysis that has been provided fails to assess potential project impacts on Bordering Land Subject to Flooding (BLSF) and Bordering Vegetated Wetlands (BVW), resource areas that are protected under 310 CMR 10.00.

Moreover, in failing to select an appropriate analysis point, the proponent's hydrologic analysis fails to demonstrate that the stormwater management system for the project will contribute to the following interests:

Flood control
Storm damage prevention

The reasons the project fails to demonstrate compliance are discussed in greater detail in the paragraphs to follow.

Appropriate Analysis or Design Point for Hydrologic Analysis, based on Stormwater Standard

On the subject property, the lowest point along the downgradient property boundary is a stone box culvert, with approximate dimensions of 5-feet wide by 2-feet tall. This culvert conveys a perennial stream under the former railroad bed which borders the northerly boundary of the property. This culvert serves as an ideal hydrologic design point, as the entire locus flows to the culvert inlet, and the culvert has definable hydraulic characteristics that allows for an accurate analysis of flood plain response to a rainfall event.

Compliance with Stormwater Management Standard 2 dictates that this point be used as the hydrologic analysis design point. More importantly, to fully evaluate the potential impacts of the proposed project on *Flood Control* and *Storm Damage Prevention*, it is critical that the culvert be used as the point of analysis.

Contributing Watershed to Box Culvert Design Point and Project Impacts

Based on a USGS StreamStats analysis, attached herewith, the watershed that contributes runoff to the intermittent stream at the box culvert has an area of approximately 60.7-acres. Much of the upgradient watershed is highly urbanized and therefore can produce high stormwater runoff volumes and respond rapidly to rainfall inputs.

To fully understand any potential flooding issues on abutting or nearby properties related to the proposed project, the hydrologic analysis must evaluate the entire contributing watershed, not only the upland portions of the subject property. Total flow to the box culvert must be analyzed in both the pre- and post-construction condition, and that analysis must evaluate water levels at the box culvert based on the culvert conveyance capacity and available flood storage volumes. This is basic hydrologic engineering, and to date this has not been performed

Stream Channel and Flood Plain Characteristics

The intermittent stream that flows to the culvert flows in a shallow channel with a bank height, along the locus, of approximately one foot or less. Beyond the bank, topography is quite flat, forming a broad flood plain up to, and possibly beyond, the limits of the Bordering Vegetated Wetland (BVW) that lies adjacent to the perennial stream.

The StreamStats analysis reports a bank-full depth of flow of 0.44-feet, a value consistent with my observations of the channel during the site inspection. The analysis further reports a bankfull width of flow of 5.33-feet, also consistent with my observations during the site inspection. StreamStats estimates the bankfull conveyance capacity at 3.88 CFS.

As discussed above, a thorough hydrologic evaluation of the watershed that contributes flow to the box culvert has not been performed, but it is likely that the bankfull conveyance capacity of the stream channel will be exceeded for larger storm events, and overbank flood storage will occur. This topic will be discussed further in the subsequent section.

Bordering Land Subject to Flooding (BLSF)

While the land within the subject property is not currently mapped as Flood Hazard Zone A, based on the contributing watershed area, the stream channel characteristics and the culvert hydraulics, the land adjacent to the stream, on the southerly side, likely does experience flooding for all storm events including and above the 2-year event.

The topography adjacent to the stream is quite flat, so if overbank flow does occur, it has the potential to spread out laterally a significant distance from the channel.

Although the stream does not have a mapped, 100-year zone A hazard zone mapped by FEMA (National Flood Insurance program, or NFIP), this does not preclude the presence of BLSF, based on the Definitions provided in *310 10.57 (2), Definitions, Critical Characteristics and Boundaries.*

Referring to 310 CMR 10.57 (2) (a) 3., the regulations state *"Where NFIP Profile data is unavailable, the boundary of Bordering Land Subject to Flooding shall be the maximum lateral extent of flood water which has been observed or recorded. In the event of a conflict, the issuing authority may require the applicant to determine the boundary of Bordering Land Subject to Flooding by engineering calculations which shall be:*

a. Based upon a design storm of seven inches of precipitation in 24 hours (i.e., a Type III Rainfall, as defined by the U.S. Conservation Soil Service:

b. *Based upon the standard methodologies set forth in U.S. Soil Conservation Service Technical Release No. 55, Urban Hydrology for Small Watersheds and Section 4 of the U.S. Soil Conservation Service, National Engineering Hydrology Handbook; and*
c. *Prepared by a registered professional engineer or other professional competent in such matters*

Flooding has been reported by several residents whose property abuts the stream channel, as was stated at the site meeting. Additionally, there is documented evidence of flooding at the property given in the report "*Flood and Natural Hazard Mitigation Plan, City of Northampton, Office of Planning and Development,* "as approved by the City Council on August 19 and September 4, 2004. That report, on a map provided on page 36, specifically identifies the perennial stream flowing through the parcel as having flooded during Tropical Storm Floyd in 1999.

Based on the reported flooding and the lack of analysis by the proponent's engineer, it is reasonable for the Department to require a hydrologic analysis, performed within the guidelines provided in 310 CMR 10.57 (2) (a) 3, be provided to evaluate the presence of BLSF. This is a straightforward and routine analysis for an engineer with competency in hydrologic engineering to perform. The presence of the stone box culvert provides for an ideal hydraulic control point, where a standard computer simulation model, such as HydroCad, can perform a stage-storage reservoir routing analysis to establish the maximum elevation that flood waters will reach during a 24-hour, 100-year, Type III rainfall event.

Further, as will be discussed below, the hydrologic analysis prepared by the proponent's engineer indicates that runoff volume directed to the culvert will increase, so there is a possibility that flood elevations will increase as a result of this project. Elevated flood levels are a concern of the abutters and resident group and require an evaluation which has not been provided by the proponent's engineer.

<u>Hydrograph Timing</u>

As was discussed earlier in this report, the proponent's hydrologic evaluation studies only 3.56-acres of the 60.7-acre watershed that contributes flow to the box culvert design point. Within those 3.56-acres, the project will create 31,000 square feet (SF) of new impervious area. Due to the poorly draining soils on the property, infiltration potential is severely limited, and the designer has relied upon a sub-surface detention basin to control peak runoff rates in the post-development condition.

This approach is problematic in that the analysis does not consider the timing of the project hydrograph, with the hydrograph that represents the flow from the upper watershed and the portion of the property that lies beyond the upper edge of the BVW. The time at which the peak flow occurs on a hydrograph is a function of the watershed characteristics, including the degree of urbanization, basin topography, and the length that water must travel from the furthest reach of the watershed to the design point. In the case of this project, the hydrograph from the larger, upper watershed will peak much later in the storm than the hydrograph from the proposed development area. However, by using detention as the primary means of rate control, the outflow hydrograph from the development will be stretched and elongated, delaying the time to peak from the project. This increases the possibility that the hydrograph peak from the project will coincide with the hydrograph peak from the larger, upper watershed. The potential risk is that the combined hydrograph in the post-development condition will exceed that of the pre-development condition, thereby increasing the peak stream flow rate in the post-development condition.

This possibility will violate Stormwater Standard Two. The proponent's hydrologic analysis has not evaluated this condition due to the improper selection of an analysis point.

Runoff Volume

The proponent's hydrologic analysis reports that total runoff volume from the developed 3.56-acres will increase by 35-percent for the 2-year storm, and 15-percent for the 100-year storm. This raises two concerns as follows.

1. *Increase in Flood Levels*

Since the property likely currently experiences some degree of flooding even for minor storm events, an increase in total runoff volume raises the likelihood that flood elevations will increase, and the duration of flooding will be extended. This may result in off-site property damage and, if so, the project will fail to protect the interests of Storm Damage Prevention and Flood Control.

2. *BVW Alteration*

The definition of "Alter" is provided in 310 CMR 10.04 as:

Alter means to change the condition of any Area Subject to Protection Under MGL c. 131, Section 40. Examples of alteration include, but are not limited to, the following:

(a) The changing of pre-existing drainage characteristics, flushing characteristics, salinity distribution, sedimentation patterns, flow patterns and flood retention areas.
(b) The lowering of the water level or water table
(c) The destruction of vegetation
(d) The changing of water temperature, biochemical oxygen demand (BOD), and other physical, biological or chemical characteristics of the receiving water

The increase in water volume delivered to the BVW will make the wetland "wetter", with water standing in the BVW for longer periods of time and at a greater depth. Additionally, water quality characteristics of the water will change. Water temperatures will increase due to heat retention within impervious surfaces and chemical properties will change due to dissolvable constituents such as sodium that cannot be removed by the stormwater system. These environmental changes will alter wetland hydrology over time. Species which tolerate wetter conditions, warmer waters and are resistant to chemicals will thrive over species that are less tolerant of such conditions. Invasive species such as Japanese Knotweed, which was observed to present in the property, are particularly adept adopting to urbanized conditions which alter water quality.

Stormwater Infiltration System #1 (SIS #1)

Stormwater Infiltration System #1 is used to satisfy the minimum groundwater recharge requirements per Stormwater Standard # 3.

The engineer/modeler takes a curious approach to stormwater modeling in the case of SIS #1; The model assumes that the system is present for the smaller design storm, the 2-year event, but assumes that it is completely absent for the larger storm events, the 10-year and 100-year event. If the design is constructed, however, the system will receive discharge not only from the smaller storms, up to the 2-year event, but also from the larger storms. The obvious question is how will the system perform when it is subjected to the higher peak flow rates and volumes associated with the large storm events? Will water break out of the down-gradient slope? Will water levels produce pressure gradients that will pop the covers off the system and allow flow through the top of the chambers. The answer to these questions are unknown, and can only be answered by running a model that accurately represents the system as proposed.

SIS #1 also fails to satisfy the minimum offset requirement of two feet between the seasonal high groundwater table and the bottom of the infiltration system. TP-(2), as shown on Sheet LC-130, reports the seasonal high-water table at 134.83-feet. The bottom of stone of SIS #1 is proposed at 136.33-feet, 1.5-feet above the reported seasonal high groundwater table.

The project narrative, on page 4, states that sub-watershed catchment Area P3 drains into Subsurface Detention System #1 (SDS #1), and then into SIS #1. In fact, the area is first discharged into a flow diversion manhole (DIV #1), which first diverts flow into SIS #1, based on the detail of DIV #1 provided on Sheet LC-501. Flow to SDS #1 only occurs after the water level in SIS #1 reaches the top of the manhole weir, at elevation 137.70-feet. The post-development hydrologic model should be revised to reflect this design element accurately.

Stormwater Detention System #1 (SDS #1)

As I have previously noted, the current engineering design relies on detention, rather than groundwater recharge, to attenuate peak flood flows. This is accomplished by means of SDS #1, located under the pavement in the northeast corner of the site.

SDS #1 consists of 224 plastic chambers that are assembled in the field in a Lego-like manner. The chambers are set on a bed of stone and covered with a bed of stone.

To provide for flood water storage and detention, these chambers, including the stone above and along the sides of the chambers, must be empty and not holding water before a storm event begins. Thus, the system of chambers and stones must be impermeable to groundwater that may be present under or along the sides of the system.

To prevent groundwater intrusion into the chambers and surrounding stone, the engineer has called for a 30-mil thick poly-vinyl barrier to be wrapped on all four sides and the bottom and top of the system, to create an impermeable barrier to prevent groundwater intrusion.

Creating a water-tight wrap where the top, bottom and all sides of the system are water-tight using a membrane wrap is a formidable construction challenge, one that I have never seen achieved in my 40-plus years of practice. The membrane itself is difficult to work with as it is not very flexible, and establishing watertight seams can be exceedingly difficult. Moreover, it is virtually impossible to field test the system for water-tightness before backfilling, so even if meticulously constructed, the system as-built may not be water-tight.

Complicating this design is a water table that will vary in depth across the system. Based on the soil tests provided, I estimate that the water table at the northwest end of the system will be at elevation 135.6-feet, and roughly 0.7-feet below the stone bedding supporting the system. However, at the southeast end of the system the water table will be at 137.1-feet, about 0.8-feet above the bottom of the stone, and above the bottom of the chambers. The system will therefore be subject to a non-uniform hydrostatic pressure from below, pushing the system up. This force is analogous to the hydrostatic force that sometimes can push an in-ground swimming pool out of the ground in the winter). If ground water does intrude into the system at the southeast end, it will continue to flow into the chambers until the water level within the chambers fills to, approximately, elevation 137.1-feet. This will displace approximately 50 percent of the flood storage capacity of the system.

The system will be subject to both non-uniform hydrostatic pressure from below, and vehicle loading pressure from above. These forces will stress the system integrity and over time may result in a breach of the watertightness, as the modules will shift, and lateral strain will be exerted on the impervious liner. Any breach of the liner can fully compromise the detention capacity of the system by allowing groundwater intrusion, which will result in a failure of the detention component. This in turn leads to a failure to comply with Stormwater Standard # 2.

The design as presented is poor as it is nearly impossible to build and even if built well, will eventually fail due to internal stresses. In my opinion, more suitable design choices are available, and I will discuss alternative approaches at the end of this report.

Subsurface Infiltration System #2 (SIS #2)

Subsurface Infiltration System #2 is also used to satisfy the groundwater recharge requirements of Stormwater Standard #3. There are a number of issues with the design of this system, as discussed below:

Soil Conditions and Testing

No soil evaluations were conducted within the footprint of SIS #2, and these are required as per the MADEP Stormwater Handbook. A soil evaluation of soil texture and groundwater levels should be provided within the footprint of the proposed infiltration system.

Groundwater Offset to System

SIS #2 is designed with a bottom of stone elevation at 135.7-feet. The design engineer selected this elevation based on the reported groundwater level of 132.67-feet observed in TP-(1). This test pit is, however, located approximately 15-feet away from and vertically down-gradient from SIS #2. Since the water table here has been shown to reflect the surface topography, the actual water table is likely higher at the location of SIS #2, because the existing elevation in this location is higher than at the location of Tp-(1). This can be seen by reviewing the water table reported in TP-3, which is located at the same surface elevation as SIS #2, and 18-feet to the southwest. The reported water table in this test hole was 134.6-feet, and within the two-foot required minimum offset between the bottom the system and the high-water table.

To best evaluate the water table elevation at SIS #2, in the absence of an actual soil evaluation, I interpolated the groundwater elevation using the results reported in TP-(1), TP-3, TP-4 and TP-2, which surround the location of SIS #2. Based on that interpolation, the probable elevation of the water table at SIS #2 is at elevation 133.9-feet.

Based on this interpolation, the bottom of SIS #2 is offset only 1.8-feet from the water table and fails to meet even the minimum groundwater offset requirement of 2.0-feet.

Groundwater Mounding Analysis Approach is Invalid

While a groundwater mounding analysis for SIS #2 has been provided, the methodology employed is flawed.

Model Applicability

The modeler has used the Hantush Method to model potential groundwater heights. The Hantush method, developed by the USGS and available in a simple to use Excel Spreadsheet, solves the differential equations used to define groundwater flow by making several simplifying assumptions. Most importantly, the Hantoush approach assumes that the aquifer in which the infiltration system resides is uniform and infinite in the horizontal plane, and that there are no limiting boundary conditions present that could alter the shape of a groundwater mound that develops as a result of an infiltration input. Examples of boundary conditions that can invalidate the results of a Hantush analysis are streams, wells, and barriers, such as walls. In practice, the Hantush results can be assumed to be representative as long as any boundary conditions ae located more than 120-feet from the outside edge of the infiltration input, as the model truncates analysis at 120-feet.

In the case of SIS #2, the designer has called for an impervious liner to be installed 15-feet away from the north end of the system, well within the 120-foot zone of influence for the Hantush model. The purpose of the barrier is to prevent groundwater breakout from the infiltration system along a steep slope that defines the limit of grading along the wetland boundary.

This impervious boundary violates the basic assumption underlying the development of the Hantush model. In practical terms, the groundwater mound that develops will exceed that predicted by the Hantush model. Since the mound cannot extend laterally to the length predicted by the model, the height of the mound will be more extreme than predicted.

I have included the USGS report that documents the development of the Hantush approach as an attachment to this report.

The solution to this issue is to employ a model that account for a non-isotropic aquifer and boundary conditions. A groundwater model such as Modflow has the capability to accurately model the proposed conditions and predict a mound height that represents the actual proposed design conditions.

Model Input

Even if the Hantush model was appropriate, which it is not, the data input used in the model is questionable.

The Hantush method requires the following input data:

(a) Recharge Rate (volume of water to be recharged in a specific time period in feet per day)
(b) Specific Yield (a dimensionless value representing available storage in the aquifer)
(c) Horizontal Saturated Hydraulic Conductivity of the aquifer soil
(d) Basin dimensions (1/2 length and ½ width)
(e) Recharge duration period.
(f) Initial thickness of saturated zone

The modeler used the following data for these values:

(a) RR: 0.54 feet per day
(b) SY: 020
(c) K: 5.4 feet per day
(d) One-half basin dimensions: 13.86 ft by 9.9 ft
(e) Recharge duration; 1.7 days
(f) Initial thickness of saturated zone; 5.67-feet

In my opinion, the data input for items (a), (d) and (e) are incorrect for the following reasons.

The Recharge Rate, RR should be based on the total infiltration volume of 823 CF that occurs during the 100-year storm, applied over the footprint of the infiltration area which is 480 SF. This results in a recharge rate of 1.71 feet per day, not the 0.54 feet per day used in the report.

The basin dimensions, even though SIS #2 has an irregular shape, should equate to the actual surface area of the infiltration system, which is 480 SF. The dimensions input in the analysis submitted, 13.86-feet by 9.9-feet equate to an area of 549 SF, overestimating the infiltration footprint by 22 percent. The dimensions should be adjusted to approximately 9.0-feet by 13.3-feet.

Finally, the infiltration duration period should be set to one day, as the model is simulating the response to a 24-hour storm.

Using the parameters discussed above, the Hantush model predicts a groundwater mound at the center of the infiltration system of 3.3 feet. This mound will therefore extend into the infiltration system.

Roof Drainage Conveyance System

Capacity

The stormwater submittal includes a Rational Method Analysis to substantiate the diameter and slope of pipes used to convey roof water from downspouts to the infiltration systems. Unfortunately, no map or diagram was provided to identify which pipes, serving which buildings, were analyzed. As such, there is no way to review or confirm the analysis. A diagrammatic sketch should be provided along with the analysis so that the pipe runs, diameters and slopes can be reviewed.

Cover

The cover over the pipes in the rear of the buildings needs to be assessed to confirm that a minimum one one-foot of ground cover over the top of the pipes is provided. Depending on the required pipe diameters and slopes needed to achieve conveyance capacity, several of the conveyance pipes may have insufficient cover. Of particular concern is the 6-inch diameter pipe that runs behind Building #8. Based on the minimum required slopes, the invert at the north end of this pipe will be approximately 138.8-feet and the crown of the pipe will at elevation 139.3-feet. The proposed finish grade at this location is elevation139.5.feet, or only 2.4-inches above the top of the pipe. While this may seem like a minor point, it is indicative of a design plan that has been rushed and does not consider constructability issues.

Maintenance

The submitted Stormwater Operation and Maintenance Plan makes no mention of the roof gutter and piping conveyance network, even though the entire hydrologic analysis is predicated on the collection and conveyance of roof stormwater to the disposal systems. If the roof water is not collected and conveyed, for all design storm events, then the analysis as presented will not represent the actual site conditions and the downstream flooding will absolutely occur. Maintenance of the gutters, downspouts and sub-surface conveyance conduits is critical to the performance of the stormwater management system.

As it is presently designed, the collection and conveyance system cannot be inspected, cleaned or maintained. The sub-surface piping network calls for numerous tee-connections, bends, and sharp angles. No cleanouts or inspection ports brought to finish grade are called for on the plans.

As anyone who has ever cleaned a gutter knows, roof shingles gradually degrade and discharge aggregate into the gutters. Gutters are also a magnet for leaves, pine needles, acorns and children's toys. All these deleterious materials will eventually find its way into the conveyance pipes. Clogging is inevitable, especially at bends, junctions and angle points where changes in velocity can result in material deposition. The current plans provide for no mechanism to inspect and clean the pipes, and the O & M does not even require inspection or maintenance of the system. This is a gross oversight that must be corrected.

Low Impact Development Components

The Stormwater Management System as submitted offers no Low Impact Development (LID) components. Contrary to the engineer's assertion in the Stormwater Report, infiltration systems, which now have been largely eliminated in favor of sub-surface detention, are not considered LID components.

Conclusions and Recommendations

The Stormwater Management System for this project has been designed on a knife's edge, pushing hard against the regulatory limits for a site with such poorly draining soils and a water table close to the ground surface. If any of the design assumptions prove to be incorrect the system will fail to protect the interests of the Wetlands Protection Act and the downstream residents. If minor irregularities occur during construction, which is the norm, system failure is the likely outcome. If the ground settles over time causing even a minor rupture of the Subsurface detention system, failure will result. Given the physical limitations of this site, there will be no available remedy to correct deficiencies once construction is complete and nearby properties will incur damage.

In my opinion better stormwater management alternatives are available that would allow the project to move forward at a similar density. I specifically recommend that the following alternatives be explored:

(a) A stormwater basin, open to the air and above-grade, should be considered in the northwesterly portion of the property. The system could employ rain gardens, an open infiltration basin, or a constructed wetland to manage the site's stormwater runoff. Such a system offers significant benefits in terms of sustainability, inspection and maintenance, and greatly reduce the possibility of a system failure. This approach would likely require an adjustment in the building design and placement, perhaps combining single family units into attached units to free up real estate for stormwater management purposes.

(b) Porous or pervious pavement should also be considered for the driveways, the main access road, or both. Since the site is being filled in the driveway and road locations, porous pavement can be an effective means to minimize stormwater runoff and reduce the required size of other stormwater management systems.

Thank you for your consideration. I am available at your convenience to discuss any aspect of this report or my concerns.

Sincerely yours,

Robert A. Gemma, PE, PLS
President