Frank C. Magnotta, P.E.PC

Consulting Engineer

395 Main St., Portland, CT 06480 Phone (860)342-2191

FrankCMagnottaPE@aol.com

CIVIL, SITE, STORMWATER

SUBDIVISION, SEWAGE DISPOSAL

July 25, 2020

Town of Chaplin
Planning & Zoning Commission
Mr. Jay Gigliotti, ZEO
495 Phoenixville Road
Chaplin, CT 06235

Re: Bestway Food & Fuel, 64-66 Willimantic Road, Chaplin - Application #SP19-115 for special permit for site plan approval.

Dear Commissioners:

I am writing to respond to the Towne Engineering, Inc review memo dated June 16 , 2020 for the above referenced application.

Comment No.

#1 – This is a response to a portion of comment #1 in a June 5, 2020 letter from Towne Engineering, Inc. As indicated in this letter, the bottoms of both basins have been raised to achieve the 3ft groundwater separation. This is discussed and analyzed in detail in the revised stormwater report dated July 25, 2020.

Mr. Maynard incorrectly categorizes the proposed stormwater basins as infiltration. Because these basins use bioretention/rain garden technology, they fall under the classification of "filtering practices" as described in the CT Stormwater Quality Manual, 11-P4 pages 1-13. This chapter discusses almost exclusively, surface and deep manmade sand filters with complete underdrain systems and discharge piping. Only one paragraph and a generic diagram in this entire chapter make reference to bioretention systems. Mr. Maynard incorrectly lumps together the various design requirements for sand filters with those of rain garden type bioretention basins. He states that bioretention basins must have a forebay area equal to 25% of the water quality volume. There is no such requirement in the Stormwater Quality Manual , chapter 4.4, or the current "Low Impact Development Practices" for bioretention/rain gardens. The reason for the large forebay requirement in the manual is that it is based on the use of deep manmade sand filters with underdrains which alone do not produce a quality effluent and therefore require a large pre-treatment volume prior to the filter. It is noted that because of the nature of these sand filters, chapter 11-P4-4 of the Stormwater Quality Manual specifically states that 1) "the use of surface sand filters is not feasible in areas of high water tables" and 2) "these filtering systems are generally applicable to highly impervious sites".

However, a bioretention facility by nature, produces a higher quality effluent with the only pretreatment required being an oil/particle separator to remove floatables and sediment. For the bioretention basins, the function of the forebay is for scour protection of the pipe flow discharging in the basin and bulk sediment removal.

Again, Mr. Maynard's statement that the basins must drain within 24 hours is a requirement only of the manmade sand filters. In the situation of a bioretention basin, 1-2 days retention is considered acceptable and further enhances the quality of the effluent. The revised stormwater analysis dated, July 25,2020, provides numerical printouts for both basins showing how long it takes for the basins to dewater using 10 and 100 year rainfall events. These printouts show both basins are dewatered within 24 hours of the start of the rain event. The narrative from this stormwater analysis is attached to this letter.

#2 – No response necessary.

#3 – The 12" emergency overflow pipe from basin #2 has been rerouted to connect directly to manhole #5. The previous design with manholes #6 and 7 with a separate outlet have been deleted. The overflow from basin #2 combines with all the building roofwater drains at manhole #5 which then is piped to basin #1.

With the changes in the basin bottom elevations, the routing analysis for both basins for the regulated storms and frozen ground conditions were performed for the July 25th report, including adjustments to the existing storm drain piping for this watershed for both pipe material, slopes and invert elevations. Details of the analysis for both basins are discussed in the narrative of the stormwater report. Peak flow rates and permeability parameters have not changed from previous reports, which were reviewed and agreed upon with Mr. Maynard in the May 30th analysis.

A review of the DEEP Dam Safety regulations and permit requirements indicate that basin#2 does not fall under any of the requirements based on hazard level, height above existing grade, or total storage volume within the basin. Because of the small nature of the basin it is unlikely that even a simple registration under the general permit statute would be required. The one foot freeboard normally required only for the regulated storm events has been applied to the frozen ground water surface elevations determined by the routing analysis for both basins. The top of berms in both basins have been revised to provide the one foot freeboard above the 100 yr water surface elevations.

The additional stormwater discharge to the wetland has been eliminated by directing the overflow from basin #2 into basin #1, eliminating that inland wetland regulated activity. The presence of the overflow pipe from basin #2 passes within 25 ft of the reserve leaching trench which requires further review by the Eastern Highland Health District. The revised site plan will be sent to the health district for their final approval.

As stated in previous letters, frozen ground is not a regulated stormwater condition that requires an analysis for detention or flow. This condition requires a rare combination of : 1) the presence of a significant frost in the ground, 2) absolutely no snow cover in place, and 3) the occurrence of a major hurricane rainfall event, which is impossible.

#4 – No response necessary.

#5 – All other elements in the buffer are buried with only the bioretention basins visible. These visible elements will be fields of grass, extensive evergreen screening and landscaping per the zoning regulations.

The outside of the proposed concrete retaining wall for basin #2 will be backfilled to the top of the wall with rip-rap, infilled with topsoil and seeded to hide it from view. Utilizing this wall and rip-rap backfill provide an undisturbed area between the basin and the property line provides an area for buffer planting. To demonstrate this, the limits of the toe of fill for the retaining wall fill have been shown on the plan, which are 5 - 10 feet from the neighbors property line. This strip can accommodate all the proposed pine trees and mountain laurel plantings along that property line without disturbing the rip-rap fill against the wall. Note that there are no trees to be planted on the inside of the basin, only shrubs and grass as the interior basin slopes are not conducive to the planting of large trees.

Mr. Maynard's concept of utilizing the northern part of the property for added detention basins, 1) still requires the current basin #2, 2) creates significant difficulties by creating unnecessary regulated activity close to a wetland and 3) irresolvable conflicts with the sanitary radius of the proposed public water supply well. To direct stormwater to the northern side of the property would require the entire southerly side to be significantly elevated above what is currently being proposed, making the embankments and developed site more visible from the neighboring properties and street than the current plan, and grading issues with basin #2.

It is my opinion that the current site plan design and stormwater management facilities comply with all the zoning, health department and state regulations with a level of activity in the regulated upland wetland areas acceptable to the Inland Wetlands Agency.

A prudent and feasible analysis as required in the wetland regulations would show that proposed grading and other activities close to the wetland, as recommended by Mr. Maynard, would not be considered an acceptable alternative to the current site plan design. Since the revisions in the March 21st stormwater report, when actual soil permeability information was incorporated into the documents, the stormwater management design was in full compliance with the Chaplin Zoning Regulations, State Public Health Code and the CT DOT Standard Engineering Practices. The imposition of a frozen ground condition is not shown anywhere in these documents and is not required for site plan approval.

#6a– The plans have been corrected to eliminate these minor drafting issues.

#6b– The new light fixture will not have a pole, but will be attached to the canopy and it does not create any issues with lane widths or truck access to the diesel pumps.

#6c– The outlet pipe for the o/p separator is correctly labeled as a corrugated pipe to decrease the flow velocity created by a steep pipe slope discharging into the basin.

#6d– This silt fence has been adjusted. However, this location is still required because of proposed pine tree plantings parallel to the wetlands.

In response to Mr. Maynard's statement about noncompliance with zoning regulations, neither he or the zoning enforcement officer has ever indicated non-compliance with any section of the Chaplin zoning regulations regarding the design of stormwater facilities because there are none. Throughout Mr. Maynard's review he has never made reference to the standard engineering design practices in the state of CT, specifically the TR-55 Urban Hydrology for Small Watersheds, and the 2002 CT DOT Drainage Manual. The stormwater drainage facilities designed for this project use both these documents as design criteria. In fact the stormwater basin designs go well beyond the requirements of the DOT manual by designing detention basins that retain virtually all post development site runoff on site with no surface overflow. The CT statutes and the DOT Drainage Manual identifies a property owner's right to a post development surface water discharge that is equal to or less than the pre-development runoff rate.

Attached to this letter are copies of the narrative from the July 25, 2020 Stormwater Analysis and 3 pages of excerpts from the 2004 CT Stormwater Quality Manual as referenced above. Also, accompanying this letter are the July 25, 2020 stormwater analysis of both basins and related storm drainage pipes and a copy of all site development drawings.

Respectfully Submitted,

Frank C. Magnotta, PE
Principal

STORM WATER ANALYSIS

BASINS #1 & #2

FOR

BESTWAY FOOD & FUEL

PROPERTY AT

64-66 WILLIMANTIC ROAD
CHAPLIN, CONNECTICUT

PREPARED BY

FRANK C. MAGNOTTA, P.E., PC
395 MAIN STREET
PORTLAND, CT 06480
860-342-2191
FrankCMagnottaPE@Aol.com

REVISED REPORT - JULY 25, 2020

NARRATIVE

In response to comments and concerns by Towne Engineering, Inc, the following changes have been made to the design of the two proposed stormwater basins, revisions to the storm drainage pipes flowing to basin #2 and conclusions of the routing analysis for this project. Note that no changes were made to the existing condition watersheds and their resulting hydrographs. The original printouts of these hydrographs and peak flow summary sheets from the October 18, 2019 stormwater report are attached herein. For the regulated storm events using exfiltration, a numeric printout is provided for the 10 and 100 year events that list the exfiltration rates and the surface overflow rates separately for discharges from each basin. This printout also shows the time when the depth of water is a ½ inch or less which is when the basin is effectively de-watered .

Basin #1

The bottom of this basin has been raised 0.5 ft to elevation 493.5 ft providing a 3 ft groundwater separation. As part of this change, the interior westerly embankment slope has been revised to be 3:1, consistent with the other slopes within the basin. This resulted in a small volume increase for the bottom elevation and the 494 contour areas. All other design features of this basin remain unchanged from the prior routing analysis.

For the regulated storm events, the revised routing analysis shows the water surface elevations for the 2 through 50 year events to be contained within the basin with no spillway overflow. The 100 year event has a spillway overflow lasting 15 minutes with a maximum discharge of 0.16 cfs. The 10 year numeric printout shows the basin flooding to a depth of 0.5 ft and being de-watered within 16 hours of the beginning of the storm event. The 100 year numeric printout shows the basin flooding to the spillway crest (a depth of 1.35 ft) and being de-watered within 14 hours of the beginning of the storm event. Given the typical time between any re-occurring storm events, the available basin volume would be more than adequate.

For a frozen ground condition, the emergency overflow pipe from basin #2 has been re-routed and connected to manhole #5. Peak discharge rates and the pipe hydraulics are discussed under basin #2.

Because the bottom of this basin was raised 6 inches, the inlet pipes to this basin were raised to elevation 493.5'. As a result, the slopes and pipe material for connecting pipes from catch basins #3 &4, manhole #5 and the o/p separator had to be revised.

All storm events routed through this basin includedthe new overflow from basin #2. Water levels from all storms passed over the crest of the emergency overflow spillway with depths ranging from 2 to 4 inches. The outlet of this spillway sheet flows to an adjacent wetland.

Basin #2

The bottom of this basin has been raised 0.10 ft to elevation 489.60 ft providing a 3 ft groundwater separation. The resulting change in bottom area was too small to create any measurable difference and was not changed in the pond report.

The previous pipe section from manhole #6 to the outlet near the wetlands has been eliminated. The emergency overflow pipe has been re-routed and connected between the basin and manhole #5. This new pipe is 145 LF of PVC pipe with a roughness coefficient of 0.010 . This reduced coefficient allowed the inlet invert to be lowered to 494.0 ft resulting in a pipe slope of 0.40%.

In order for this PVC pipe to pass under the 4" sewer line , the sewer invert at the first D-box had to be raised to 495.5'. Both pipe crossing details have been revised to show the resulting pipe clearances.

For the regulated storm events, the revised routing analysis shows the water surface elevations for the 2 through 100 year events to be contained within the basin with no spillway overflow. The 10 year numeric printout shows the basin flooding to a depth of 0.65 ft and being de-watered within 13.5 hours of the beginning of the storm event. The 100 year numeric printout shows the basin flooding to a depth of 1.6 ft (4.4 ft is the depth to the overflow pipe) and being de-watered within 16 hours of the beginning of the storm event. With this rate of dewatering, the full basin volume would be available for any re-occurring storm events.

For a frozen ground condition, the water surface elevation for the 2 year storm event does not reach the emergency overflow pipe. The water surface elevations for the 10 thru 100 year storm events reach the invert of the emergency overflow pipe with water depths from 1 ½ inch to 6 inches respectively for the full range of storm events. The pipe hydraulics for the overflow pipe resulted in a flow velocity of 3.29 feet per second using the 100 year event discharge rate of 0.91 cfs.

innovative and emerging technologies as secondary treatment practices. These technologies are designed to remove a variety of stormwater pollutants, but have not been evaluated in sufficient detail to demonstrate the capability to meet established performance standards. Sizing and selection criteria for stormwater treatment practices are addressed in Chapter Seven and Chapter Eight, respectively.

New Development Versus Retrofits

Stormwater treatment practices can be implemented for new development projects as well as existing, developed sites. Retrofitting existing developments can improve water quality mitigation functions of older, poorly designed, or poorly maintained stormwater management systems. Incorporating stormwater retrofits into developed sites is typically more difficult than implementing treatment practices for new development due to the numerous site constraints associated with developed areas such as subsurface utilities, buildings, conflicting land uses, and maintenance access. Chapter Ten describes common stormwater retrofit options for existing development and redevelopment projects, including:

- *Stormwater collection system retrofits*
- *Stormwater management facility retrofits*
- *New stormwater controls at storm drain outfalls*
- *In-stream practices in existing drainage channels*
- *Parking lot stormwater retrofits*
- *Wetland creation and restoration*

3.7 Stormwater Quantity Control

Stormwater quantity controls include drainage and flood control. As shown in **Figure 3-1,** stormwater quantity and quality controls are related and complementary elements of an effective stormwater management strategy. Stormwater drainage systems can be designed to reduce the potential erosive velocity of stormwater runoff and maintain pre-development hydrology through infiltration and the use of vegetated conveyances, thereby preserving the water quality mitigation functions of a site. Similarly, stormwater treatment practices such as stormwater ponds and wetlands can provide dual flood control and water quality treatment benefits.

This Manual addresses the topics of drainage design and flood control as they relate to stormwater quality management. The Manual identifies stormwater treatment practices that also provide peak runoff attenuation and channel protection functions. However, this document is not intended to serve as a drainage or flood control design manual. Other recommended guidance documents and manuals on these topics include:

- *2000 Connecticut Department of Transportation Drainage Manual, October 2000*
- *Connecticut Department of Environmental Protection, Model Hydraulic Analysis, revised February 13, 2002*
- *Urban Hydrology for Small Watersheds, TR-55, Natural Resource Conservation Service (formerly Soil Conservation Service), June 1986*

In addition, municipal ordinances, as well as some DEP regulatory programs, contain specific stormwater quantity control requirements for land development projects, as described in Chapter One.

Drainage Design and Flood Control Principles for Water Quality

The traditional approach to drainage design has been to collect and remove runoff from the site as quickly as possible through the use of curbs, gutters, catch basins, and storm sewers, often resulting in the discharge of polluted runoff directly to receiving waters. While this approach effectively removes runoff from a site, it does not address water quality or downstream flooding and erosion issues. Similarly, the traditional approach to flood control has been to attenuate peak runoff to pre-development levels through the use of detention and retention ponds. While stormwater detention or retention facilities can effectively reduce peak discharge rates, they also typically prolong the duration of elevated flows and do not reduce runoff volumes unless infiltration is incorporated into their design. Historically, these facilities have not adequately addressed problems associated with water quality, runoff volume, and downstream channel erosion.

Drainage and flood control facilities should be designed according to the following principles to address water quality objectives:

- *Identify and assess existing stormwater runoff rates and volumes at the site, as well as downstream flooding and erosion concerns.*
- *Preserve pre-development hydrologic conditions, including peak discharge, runoff volume, groundwater recharge, and natural drainage paths.*
- *Reduce the potential for increases in runoff quantity by minimizing impervious surfaces and maximizing infiltration of stormwater runoff. Eliminate curbs where possible and encourage sheet flow from paved areas. If*

outlet of a site. Runoff flow paths, ground surface slope and roughness, and channel characteristics affect the time of concentration. Site design techniques that can modify or increase the runoff travel time and time of concentration include:

- *Maximizing overland sheet flow*
- *Increasing and lengthening drainage flow paths*
- *Lengthening and flattening site and lot slopes (although may conflict with goal of minimizing grading and disturbance)*
- *Maximizing use of vegetated swales*

(Prince George's County, Maryland, 1999).

4.4 Low Impact Development Management Practices

Low Impact Development (LID), a relatively new concept in stormwater management pioneered by Prince George's County, Maryland and several other areas of the country, is a site design strategy that employs many of the concepts and practices already described in this chapter. The goal of LID is to maintain or replicate predevelopment hydrology through the use of small-scale controls integrated throughout the site (U.S. EPA, 2000). Site design techniques such as those described above are one component of the LID approach. The other major component of the LID approach is the use of micro-scale integrated management practices to manage runoff as close to its source as possible. This involves strategic placement of lot-level controls to reduce runoff volume and pollutant loads through infiltration, evapotranspiration, and reuse of stormwater runoff.

The appropriateness of LID practices is highly dependent on site conditions. Soil permeability, slope, and depth to water table and bedrock are physical constraints that may limit the use of LID practices at a site. Community perception and local development rules may also present obstacles to the implementation of LID practices, as described previously in this chapter. Although alternative site design and LID practices may not replace the need for conventional stormwater controls, the economical and environmental benefits of LID practices are well documented (U.S. EPA, 2000). LID practices described in the following sections include:

- *Vegetated Swales, Buffers, and Filter Strips*
- *Bioretention/Rain Gardens*
- *Dry Wells/Leaching Trenches*
- *Rainwater Harvesting*
- *Vegetated Roof Covers (Green Roofs)*

The main feature that distinguishes these practices from conventional structural stormwater controls is scale. These small systems are typically designed as off-line systems that accept runoff from a single residential lot or portions of a lot, as opposed to large multiple-lot or end-of-pipe controls. The following sections contain summary descriptions of these small-scale LID practices. The design sections of this Manual contain more detailed guidance for similar, larger-scale stormwater treatment practices such as bioretention, infiltration, and filtration systems.

4.4.1 Vegetated Swales, Buffers, and Filter Strips

Vegetated swales, buffers, and filter strips are vegetative practices that can be incorporated into a site to maintain predevelopment hydrology. These practices are adaptable to a variety of site conditions, are flexible in design and layout, and are relatively inexpensive (U.S. EPA, 2000). Vegetated swales can provide both water quantity and quality control by facilitating stormwater infiltration, filtration, and adsorption. Vegetated buffers are strips of vegetation (natural or planted) around sensitive areas such as wetlands, watercourses, or highly erodible soils (Prince George's County, Maryland, 1999). Similarly, filter strips are typically grass or close-growing vegetation planted between pollutant source areas and downstream receiving waters or wetlands. Filter strips are commonly located downgradient of stormwater outfalls and level spreaders to reduce flow velocities and promote infiltration/filtration. Chapter Eleven provides additional design guidance on these vegetative practices.

4.4.2 Bioretention/Rain Gardens

Bioretention is a practice to manage and treat stormwater runoff by using a specially designed planting soil bed and planting materials to filter runoff stored in a shallow depression (Prince George's County, Maryland, 1999). Bioretention areas are composed of a mix of functional elements, each designed to perform different functions in the removal of pollutants and attenuation of stormwater runoff. Bioretention removes stormwater pollutants through physical and biological processes, including adsorption, filtration, plant uptake, microbial activity, decomposition, sedimentation, and volatilization (U.S. EPA, 2000). The major components of a bioretention system include:

same general configuration, with specialized filter media targeted at removal of various particulate and soluble pollutants. Most of these pre-manufactured systems consist of a sedimentation chamber and a filtration chamber that holds a series of canisters with replaceable/recyclable media cartridges. These systems currently are not considered primary treatment practices due to limited peer-reviewed data on their performance under field conditions. Proprietary filtering designs are discussed further as secondary treatment practices later in this chapter.

Advantages

- *Applicable to small drainage areas.*
- *Can be applied to most sites due to relatively few constraints and many design variations (i.e., highly versatile).*
- *May require less space than other treatment practices. Underground filters can be used where space limitations preclude surface filters.*
- *Ideal for stormwater retrofits and highly developed sites.*
- *High solids, metals, and bacteria removal efficiency.*
- *High longevity for sand filters.*
- *Bioretention can provide groundwater recharge.*

Limitations

- *Pretreatment required to prevent filter media from clogging.*
- *Limited to smaller drainage areas.*
- *Frequent maintenance required.*
- *Relatively expensive to construct.*
- *Typically require a minimum head difference of approximately 5 feet between the inlet and outlet of the filter.*
- *Surface sand filters not feasible in areas of high water tables.*
- *Should not be used in areas of heavy sediment loads (i.e., unstabilized construction sites).*
- *Provide little or no quantity control.*
- *Surface and perimeter filters may be susceptible to freezing.*
- *Surface filters can be unattractive without grass or vegetative cover. Bioretention may be a more aesthetically pleasing alternative due to incorporation of plants.*
- *May have odor and mosquito-breeding problems if not designed properly.*

Siting Considerations

Drainage Area: The maximum contributing drainage area for most surface and underground filtering practices is between 5 and 10 acres. Filtering practices can be used to treat runoff from larger drainage areas if properly designed, although the potential for clogging increases for drainage areas larger than 10 acres. Bioretention should be restricted to drainage areas of 5 acres or less.

Slopes and Head Requirements: Filtering systems can be used on sites with slopes of approximately 6 percent or less. Most stormwater filter designs require between 5 and 7 feet of head difference between the filter inlet and outlet to allow sufficient gravity flow through the system. Perimeter sand filters and bioretention areas require as little as 2 feet of head.

Soils: Stormwater filtering systems that return filtered runoff to the conveyance system and do not infiltrate into the ground can be used in almost any soil type. Bioretention designs that rely on infiltration can be used only when the soil infiltration characteristics are appropriate (see the Infiltration Practices section of this chapter).

Land Use: Filtering systems are generally applicable to highly impervious sites.

Water Table: At least 3 feet of separation is recommended between the bottom of the filter and the seasonally high groundwater table to maintain adequate drainage, prevent structural damage to the filter, and minimize the potential for interaction with groundwater.

Design Criteria

The design criteria presented in this section are applicable to surface sand filters, bioretention systems, and underground filters. Considerations for specific design variations are also included.