Review of Drainage and water-related issues at the proposed 10 Lee Road Multi-family housing Proposal

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Documents reviewed for this assessment

- Drainage Analysis, Ambit Engineering, 29 October 2021.
- Ambit Engineering Cover letter 11 Dec 2020
- Plot Plan 2020-12-11
- Ambit Engineering Cover letter 10 May 2021
- Plot Plan 2021-05-10
- Ambit Engineering Cover letter 1 Nov 2021
- Maintenance Plan (undated)
- Staff Review Memo (21 November 2021)
- Response to Staff Review (30 November 2021)
- Cover Letter 2022-01-19 from Ambit Engineering
- Plot Plan 2022-01-19
- Durfee Email 2022-02-01

Overview

A site visit was conducted on 12 November 2022. A representative from Ambit Engineering was present at that visit to aid with describing existing and proposed conditions. The site resides at Madbury Assessor's Tax Map 8 as Lot 9. The total lot size 36.4258 acres. Newly constructed impervious area increases sitewide impervious cover from 0.704 acres (1.95%) in the pre-development condition to 1.238 acres (3.42%) in the developed condition. These fractions are compared to the total site area.

Drainage Analysis

The Drainage Analysis Introduction indicates that the drainage analysis is solely for the proposed development and not existing infrastructure. The objective of the Drainage Analysis and stormwater infrastructure is peak flow control and not volume management or water quality improvement. It is represented that runoff will increase to downgradient abutters.

There is very little existing drainage infrastructure. What exists today is basically to drain stormwater away from structures so that it is not a nuisance. The proposed drainage network includes catch basins, storm drains, a detention pond, and armored outlets.

The Drainage Analysis compares runoff from the existing site to the site after the proposed development is constructed in order to assess changes to hydrology. As this reviewer has consistently represented to the Town, the site should also be modeled as though it were completely undeveloped. In doing so, it becomes apparent how the proposed stormwater management designs bring the site to be (or not) hydrologically transparent to abutters and the environment. That is, the stormwater management objective of the proposed site development should not be to recreate the present day site hydrology (where there is no real stormwater management other than to drain the site), but rather to return the site runoff as closely as possible to the hydrology and water quality of an undeveloped site. This is most notable in Table 3 on page 6 of the drainage report, where the pre- and post-development peak flows are dominated by the almost 24 acres of undeveloped land to the south.

HydroCAD Stormwater Modeling

The limits and extent of HydroCAD subwatershed E5 or P5 are not clear. These subwatersheds are presumed to be the undeveloped southern portion of the site, but without seeing it drawn on a topographic map, it is not clear if it was modeled accurately (including runon from neighboring properties).

Design precipitation was selected from NRCC. The curve number method was selected to compute runoff hydrographs. Both are considered the standards of practice, although some NH state agencies are using NOAA Atlas 14 extreme rainfall estimates. Storms with return periods of 2-year, 10-year, 25-year, and 50-year were modeled. No account for future extreme rainfall increases was employed as recommended by the New Hampshire Coastal Flood Risk Summary Part 1: Science (2019).

Some fundamental input/site data to the HydroCAD models was missing, for example how time of concentration was calculated and the flowpaths used for the calculations. Instead of calculating time of concentration for the smallest subwatershed, a value of 5-minutes was used. The 5-minute time of concentration is a time-honored selection from the last century and older computer codes, techniques, and data. However, the latest version of HydroCAD (used by the applicant) allows times of concentration as small as 1 minute. After impervious surfaces, time of concentration is the most significant variable determining runoff peak for and volume, and therefore care is recommended in its selection, and such selection should strive to be as representative as possible.

The weighted curve number (weighted-CN) approach was employed for modeling runoff. The weighted excess precipitation (weighted-Q) approach is preferred (NEH Part 630, Chapter 10) and considered more accurate, especially on sites where CN may have a wide range. The curve number for existing subwatershed E2 was assumed to be that for ¼-acre subdivision with 38% impervious cover. This subwatershed should be split to separate the uphill developed area from the downhill undeveloped area, then runoff generated for each portion using the weighted-Q method. In general, the curve numbers should account for the runoff generated from the impervious areas and the pervious areas (weighted-Q approach) and not lump them together.

Existing drainage subcatchments (E1, E2, E3, E4, E5) were drawn without recognition of topography. That is, their boundaries are not true subwatershed divides. In Table 1, add a column for runoff volumes for each design storm.

The proposed development increases impervious cover from 0.704 to 1.238 acres. This of course increases runoff. No infiltration methods are proposed for stormwater management, nor was intentional stormwater infiltration modeled.

There are features in the obtained HydroCAD model that are undefined in the Drainage Analysis reports (3S, 8P, 9R). It is not clear why they are in the HydroCAD model and not described in the Drainage Analysis.

Maintenance Plan

The Maintenance Plan is missing some important details, specifically who will; be doing the inspections and the maintenance. This was addressed in a subsequent letter (Cover Letter 2022-01-19). In addition, some metrics should be defined rather than left as subjective, for example, "…*erosion*…" and "…*excessive accumulation of sediments*…"

The Maintenance Plan focusses on stormwater generically and should be more specific about practices to be performed during construction versus post-construction. In addition, post-construction maintenance/inspection should be clearly subdivided into short term versus long-term. The plan makes little reference to winter maintenance practices other than snow storage. Low to no-salt strategies are recommended for winter maintenance of traffic areas as the entire site is uphill from the water supply well and its zone of influence.

Existing snow storage was not obvious on the plan set, it is marked on the proposed site plan.

Recommendations

For the hydrologic analysis, it is recommended to focus on only the portion of the site already developed and proposed to be developed: from the new well and northwards, and not include the large portion of the property south of the new well. As none of the southerly property is to be changed in any way, including it in hydrologic calculations simply masks site hydrologic consequences.

The New Hampshire Stormwater Manual is clear that modern stormwater management should include groundwater recharge, peak flow reduction, and runoff volume reduction. The drainage analysis that was submitted only focused on peak flow reduction.

The detention pond is one of the weakest stormwater management strategies to improve runoff water quality. At the end of this document is a table from the UNH Stormwater Center 2012 Annual Report that identifies the pollutant removal capabilities of various stormwater structural measures. It is possible that the detention pond offers infiltration, but such infiltration does not appear in the design or calculations. Instead of the detention pond, practices that do better to improve water quality and increase infiltration are strongly recommended. Options other than the detention plan should be considered, including distributing stormwater management throughout the site rather than an end-of-pipe solution. This is called out in the Town Site Plan Review Regulations (Low Impact Development).

The effluent from the proposed detention pond will overflow from a pipe to a rip rap splillway and flow southerly to wetlands. This flowpath should be inspected annually to assess if gullying becomes a problem.

Instead of one single pipe discharge from the detention pond, consideration should be made for multiple pond discharge points to diffuse the detention pond effluent.

UNHSC Measured Median Pollutant Removal Efficiencies

	TSS Total Suspended Solids (mg/l)			TPH-D Total Petroleum Hydrocarbons in the Diesel Range (ug/l)			NO3-N (DIN) Dissolved Inorganic Nitrogen (mg/l)			TZn Total Zinc (mg/l)			TP Total Phosphorus (mg/l)			Average Annual Peak Flow Reduction	Average Annual Lag Time
Treatment Unit Description	Influent	Effluent	% Removal	Influent	Effluent	% Removal	Influent	Effluent	% Removal	Influent	Effluent	% Removal	Influent	Effluent	% Removal	% Reduction	Minutes
Conventional Treatment Technologies																	
Retention Pond	55	30	68%	710	100	82%	0.3	0.2	33%	0.05	0.01	68%	0.09	0.11	NT	86	455
Detention Pond	77	16	79%	490	165	74%	0.3	0.2	25%	0.03	0.02	50%	0.05	0.05	NT	93	639
Stone (rip-rap) Swale	30	15	50%	580	380	33%	0.4	0.7	NT	0.07	0.02	64%	-	-	-	6	7
Vegetated Swale	48	16	56%	710	207	82%	0.3	0.3	NT	0.04	0.02	40%	0.08	0.10	NT	52	38
Berm Swale	51	23	50%	637	61	81%	0.2	0.3	NT	0.03	0.02	50%	0.07	0.09	NT	16	58
Deep Sump Catch Basin	48	34	9%	510	440	14%	0.2	0.3	NT	0.04	0.04	NT	0.08	0.07	NT	NT	NT
Manufactured Treatment Devices																	
ADS Infiltration Unit	49	BDL	99%	766	BDL	99%	0.3	0.9	NT	0.05	BDL	99%	0.12	0.02	81%	87	228
StormTech	87	13	83%	750	45	91%	0.3	0.5	NT	0.03	0.01	67%	0.07	0.03	52%	78	235
Aquifilter	28	11	62%	573	156	66%	0.3	0.3	NT	0.04	0.02	43%	0.07	0.05	24%	NT	NT
Online Hydrodynamic Separators	41	29	29%	774	442	42%	0.4	0.4	NT	0.05	0.04	26%	0.09	0.11	NT	NT	NT
Offline Hydrodynamic Separators (HDS)	120	21	75%	570	180	64%	0.2	0.3	NT	0.03	0.02	21%	0.05	0.05	NT	NT	NT
Low Impact Development (LID)																	
Surface Sand Filter	45	19	51%	788	17	98%	0.3	0.4	NT	0.06	0.01	77%	0.12	0.06	33%	69	187
Bio I - 48" depth (42" filter depth)	37	1	97%	798	BDL	99%	0.4	0.1	44%	0.07	BDL	99%	-	-	-	75	266
Bio II - 30" depth (24" filter depth)	48	6	87%	750	BDL	99%	0.2	0.2	NT	0.04	0.02	73%	0.08	0.05	34%	79	309
Bio III - 30" depth (24" filter depth)	120	8	91%	450	163	64%	0.4	0.3	44%	0.03	0.01	75%	0.03	0.05	NT	84	216
Bio IV - 37" depth (24" filter depth)	80	11	83%	495	165	65%	0.3	0.2	42%	0.03	0.01	67%	0.07	0.06	NT	95	61
Subsurface Gravel Wetlands	61	4	96%	644	BDL	99%	0.3	0.1	75%	0.04	0.01	84%	0.06	0.02	58%	92	391
Porous Asphalt	32	BDL	99%	631	BDL	99%	0.2	0.5	NT	0.04	0.01	75%	0.08	0.04	57%	82	1,275
Pervious Concrete	101	11	85%	310	BDL	99%	0.3	0.5	NT	0.03	0.01	75%	0.06	0.65	NT	93	1,011
Permeable Interlocking Concrete Pavement	51	BDL	99%	610	BDL	99%	0.4	BDL	99%	0.05	BDL	99%	0.13	BDL	99%	99	see pg 16
Tree Filter	31	2	91%	631	BDL	99%	0.2	0.2	1%	0.04	0.01	75%	0.07	0.06	NT	31	204

*BDL indicates a value that is Below Detection Limit of the test method.
NT indicates no treatment.