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March 2, 2026

Mr. Greg Wands, Chair
Conservation Commission
Town of Ashland
101 Main Street
Ashland, MA 01721

RE: 3rd Peer Review – The Residences at Ashland, 61 Waverly Street, Ashland, (Assessor's Map 15, Parcel 12), Wetland Notice of Intent (NOI).

Dear Mr. Wards:

GCG Associates, Inc. has reviewed the following information regarding The Residences at Ashland, a Chapter 40B Development, Site Plan at 61 Waverly Street in Ashland, MA. This peer review is limited to the State of Massachusetts general laws, codes, and regulations only, as directed by the Ashland Conservation Department.

Documents:

1. Response to Comments – GCG Associates letter #2, prepared by SMMA, dated February 13, 2026.
2. Buoyancy Calculations, prepared by SMMA, dated 02/11/2026.
3. Existing HydroCAD Report, prepared by SMMA, dated 02/06/2026, & Existing Hydrology Map.
4. Proposed HydroCAD Report, prepared by SMMA, dated 02/06/2026., & Proposed Hydrology Map.
5. Peak Discharge Rate Summary Table, prepared by SMMA, dated 02/11/2026.
6. Pollutant Removal Calculations, prepared by SMMA, electronic file dated 02/11/2026.
7. RipRap sizing Calculations, prepared by SMMA, dated 02/06/2026.

Plan References:

1. "The Residences at Ashland, 61 Waverly Street, Ashland, MA. Notice of Intent" (Site Plan), prepared by SMMA dated October 29, 2025, last revised (Response to Comments) 02/13/2026, consists of 17 sheets as following:

   Cover Sheet
   Existing Conditions Plan (GLM)
   C-101 Existing Conditions Plan
   C-111 Site Preparation Plan
   C-121 Layout and Materials Plan
   C-131 Grading & Drainage Plan
   C-141 Utility Plan
   C-151 Planting Plan
   C-501 Details I

C-502	Details II
C-503	Details III
C-504	Details IV
C-505	Details V
C-506	Details VI
C-507	Details VII
C-508	Detail VIII
ES100	Site Lighting Photometric Plan

This project is currently under the Zoning Board of Appeals' review for Comprehensive Permit application per M.G.L. Chapter 40B, for a multi-family residential development (The Residences at Ashland Crossing) at 61 Waverly Street (Map 15 Parcel 12). A 40B comprehensive permit would be based on a preliminary site plan in nature, further development/construction details should be provided with the Building Permit application according to the Comprehensive Permit approval conditions. Under the Comprehensive Permit process, the applicant has been requested waivers for the Ashland By-Laws associated with Wetland Protection and Stormwater Management. Hence, this peer review will be based on the Commonwealth of Massachusetts General Laws (M.G.L.), Chapter 131 Section 40, and 310 CMR 10.00 – Wetland Protections, and associated Stormwater management requirements per Massachusetts Stormwater Handbook (MSH), as required under 310 CMR 10.05(6)(k) through (q).

Based upon our review of the above information, we offer the following stormwater comments with respect to compliance with the 310 CMR 10.00 and associated Massachusetts Stormwater Handbook (MSH) Standards. The numerical section of the regulations is referenced at the beginning of each comment unless it is a general comment. Since this is a NOI submittal, the site plan has been moving forward from the 40B preliminary plan to final design. The review comments are based on plan details suitable for building permit and construction. GCG latest comments shown in "Red".

GENERAL COMMENTS:

1. The site plan set consists of 16 plan sheets. However, the cover sheet listed 15 plan sheets only. Plan C-131 should be Grading and Drainage Plan, and plan sheet C-141 - Utilities Plan was not included in the plan list. Latest plan set updated to 17 sheets, resolved.

2. The project consists of approximately 10 acres of proposed work limit, which exceeds the 1-acre limit of work threshold and requires a NPDES (National Pollutant Discharge Elimination System) CGP (Construction General Permit) and requires a SWPPP to be filed at least 14 days prior to start of construction. Statement, no response required.

3. There were wetland resource areas located on-site delineated by Goddard Consulting, (see NOI package, Appendix 4.04 - Wetland Border Report). Three sets of wetland flags were marked on the field, BVW (Bordering Vegetated Wetland) Group 1 - GCA1-GCA95; BVW Group 2 - GCC1-GCC20; and Bank of Intermittent Stream Group 3 – GCS1-GCS11. All three sets of wet flags shown on the plan were labeled GCG#, there were three flags labeled GCG#53, there were missing flags #A1, #A79, #C9, #C10, #S2, and #S10. GCG recommends relabeling the wet flag numbers to match the A, C, and S series as referenced on the wetland border report. The wetland boundary requires Conservation Commission approval. Wetland flags revised and subjects to Conservation Commission approval.

Proposed works (drainage outlet connection) within Waverly Street:

4. The proposed bioretention area and the surface infiltration basin are equipped with outlet pipes, which connect and discharge to an existing catch basin and drainpipe system within Waverly Street. The drain carries the outflow from the site to the downstream stone box culvert located at the northern side of 73 Waverly Street crossing beneath the roadway. The Ashland DPW has identified the box culvert consists of a 12” RCP inlet pipe (east side) and transitioned to a 3-feet wide field stone, open bottom culvert with varies height (approximately 3’ to 4’). The ceiling of stone culvert is equipped with steel beams support (which are rusty/rotting conditions). The proposed drainage connection requires DPW Road Opening Permit approval. Based on the DPW provided site photos, the stone box culvert is in poor condition. The condition of the current conditions of the existing 12” RCP pipe on Waverly from the existing catch basin (proposed to be converted to drainage manhole - DMH 1-1 for connection) should be evaluated which should include a Closed-Circuit Television (CCTV) inspection of the pipe at minimum. Visual inspection of existing photos from inside the culvert by the DPW shows the pipe to appear to be disjointed and not at a uniform grade. This drain line is approximately 6” to 12” below the catch basin rim, as shown on the DPW photo, the RCP does not have the recommended minimum pipe cover (12” minimum). This pipe is too shallow for constant vehicle live load with lacking pipe cover protection. The applicant should provide calculations and documentation that the existing RCP’s condition and capacity are suitable to handle the overflows from this development. MSH Standard 2, requires that post-development peak discharge rates do not exceed pre-development peak rates for the 2-year and 10-year 24-hours storms events and evaluate the impact of off-site flooding due to the increase of peak discharge rates from the 100-year 24-hour storm. The project Peak discharge Rate Summary currently shown slight rates increase during the 10-year storm events to DP-4 and DP-5, and substantial discharge rates increase to DP-1, Dp-2, DP-3 during the 110-year storm events. The rates increase would impact the downstream properties during the 100-year storm event and should be addressed. Typically, development would control the post-development 100-year peak discharge rates to below the pre-development rates at the property boundary to avoid analysis downstream impacts. MSH does have De Minimis Stormwater Discharges for purposes of Standard 4. (MSH - Vol.3, Ch.1, Pg.35); However, the De Minimis does not apply to Standard (peak rate attenuation), which must be achieved on a site wide basis. (MSH - Vol.3, Ch.1, Pg.36). The report indicated post-development peak rates increase to DP-1 from 0.42 cfs to 1.8 cfs.; peak rates increase to DP-2 from 0.48 cfs to 1.36 cfs.; and peak rates increase to DP-3 from 0.12 cfs to 0.25 cfs which should be considered substantial increases. GCG recognizes there are limitations with Hydrology software and roundoff errors with calculations fluctuations, but the rates increase as shown are beyond those limits. (See additional Stormwater Report comments below).

The applicant has proposed to replace the existing 12” RCP from CB_2578 to the existing stone box culvert and requested the work should be authorized under this Order of Conditions. However, the applicant should address the post-development peak discharge rates increase from 0.42 cfs to 1.80 cfs during the 100-year storm event as shown on the summary table mentioned above. The 12” diameter drainpipe is approximately 170’ long with 1.3’ to 1.6’ cover over drainpipe and should be replaced with ductile iron pipe to support H20 loading. The connection to the existing stone box culvert (in poor conditions) details should be provided. The work is within Waverly Street right-of-way, Ashland DPW approval is required. Based on the current 90 Waverly Street Notic of Intent filing, (provided by Conservation Commission), there is wetland resource area delineated at the stone box culvert outlet. The proposed stone box culvert connection works would be within the 100-feet buffer area. GCG would recommend the replacement of the 12” culvert be designed now as part of the submittal. The latest HydroCAD report showed a net increase of post-development (DP-1) peak flow rate of 1.15cfs (cubic foot per second) to the existing 12” RCP (proposed to be replaced by the applicant), which flows northeastward to the existing stone box culvert in front of 90 Waverly Street during the 100-year storm event. Hence, off-site downstream flooding evaluation is required. Furthermore, the post-development Wetland (DP-2) also flows to the stone box culvert. Based on the Runoff Volumes Summary table, the combined post-development (DP-1 and DP-2) has a net increase of 1.061 ac-ft (46,217 c.f. or 345, 727 gal.) runoff volume discharge through the box culvert and onto 90 Waverly Street during the 100-year storm event and without a benefit of an easement. The increased peak flow rate and runoff volume are substantial and should be addressed. Since there is a similar peak flow rate and runoff volume decreased toward DP-4 (Waverly Street southward), it appeared the proposed drainage layout has altered the drainage flow pattern northward to the stone box culvert. Altering the flow pattern should be avoided.

5. GCG recommends having the applicant to assess if there is any intermittent stream associated with the existing Stone Box Culvert, (upstream and downstream). The MassMapper/MassGIS contours layer indicated the intermittent stream (Wet flags series GCC1 to GCC20) slopes toward the downstream 12” RCP stone box culvert inlet. Typically, the 12” RCP creates a contraction at the entrance and form an intermittent stream/drainage swale in front of the RCP. Which would require Conservation Commission review as stated in 310 CMR 10.02(2)(d) and 310 CMR 10.05(6)(b). 10.02(2)(d) stated that activities outside the areas subject to protection under M.G.L. c. 131, § 40., could altered an area subject to protection and 10.05(6)(b) stated that the Order shall impose conditions setting limits on the quantity and quality of discharge from a point source (both closed and open channel), when said limits are necessary to protect the interests identified in M.G.L. c. 131, § 40. Therefore, the Conservation Commission may also have jurisdiction over the resource area located at the stone culvert area. As stated in comment #4 above, the peak rates increase during the 100-year storm event do not qualify for di minimis conditions. The proposed discharge rates at 1.80 cfs during the 100-year storm event to the DP-1 (12” RCP in Waverly Street) is 428% increases of the pre-development flow at 0.42 cfs. The proposed replacing the 12” pipe to the existing stone box culvert is within the 100-feet buffer of the wetland resource area at 90 Waverly Street. Based on the 90 Waverly Street’s NOI application documents, the proposed replacement of existing 12” RCP on Waverly Street is partially in the 100 feet buffer of the BVW resource area located on 90 Waverly Street and under the Conservation Commission’s jurisdiction.

6. The proposed DMH 1-1 and CB 1-1 rims grade are approximately 7” higher than the existing catch basin rim which would not work with the existing roadway grade and should be addressed. (Lowering the proposed drain manhole and catch basin rims to match existing catch basin rim would create an insufficient pipe cover issue. GCG recommends using ductile iron pipe for locations where it has less than 2 feet of cover over pipe. Structures CB 1-1 and DMH 1-1 rim elevations have been lowered to match existing grade, the applicant should address comments 4 and 5 above.

7. The existing stone culvert under the road appears to be connected to the wetland located on site which the development discharges into. An evaluation and understanding of the connection should be performed to have a full understanding of the impact of the development on the stone culvert which is in very poor shape. Additional wetlands may need to be flagged. Adding flow or impacting the flow to this culvert should not be permitted as it may impact the stability of the culvert. The visual inspections reveal the steel beams on top of the culvert have corroded as mentioned above which support the slab above it. The Ashland DPW is aware of the

deteriorated stone culvert conditions as shown on their inspection report and photos, since the entire culvert is within the wetland 100-feet buffer (per 90 Waverly Street NOI documents) any increased peak runoff rate entering the culvert would potentially exacerbate the culvert's structure integrity. The post-development increased peak flow rate and runoff volume during the 100-yr storm event as mentioned in comment #4 above are substantial, which raises concerns with the structural integrity of stone box culvert. Replacement of the 12" RCP requires Ashland DPW approval. GCG concurs that the existing 12" drainpipe and the stone box culvert have the capacity to handle the extra post-development flow. However, the analysis should be based on the pre-development flow versus the post-development flow. Any increases in post-development peak flow rate and volume would impact on the downstream properties.

8. Proposed Porous Pavement at the site driveway entrance does not meet the 10 feet minimum setback to property line requirement, (MSH Vol. 2, Ch. 2, page 120). The 2 feet vertical separation above ESHGW and bedrock should be verified. Based on the existing 12" RCP's depth and exposed ledge along the site frontage, ledge is expected to be within couple feet below surface. The proposed 1,450 s.f. of porous pavement area is extremely small in comparison with the total proposed impervious area on site. However, the porous pavement surface requires post signs identifying porous pavement areas; cleaning the surface using vacuum sweeping machines monthly, and no winter sanding allowed, as part of the maintenance requirement, (MSH, Vol. 2, Ch.2, Pg. 122.).

9. The post-development impervious surface runoff from sub-catchments PR 1.1, PR 1.2, PR 1.3, and 4.1, do not discharge to the infiltration system. Calculations to show compliance with the 65% rule (MSH, Vol. 3, Ch.1 Pg. 27) should be provided. GCG verified that the proposed infiltration system volume has the capacity to meet the required additional recharge volume. However, the calculations should be provided by the applicant for the record.

10. The proposed Subsurface Detention System -1 is partially submerged below the ESHGW. The system was specified to be wrapped in impervious geotextile fabric, the applicant should provide additional details/procedures to assure the liners are watertight to prevent any groundwater seepage.

SITE PLAN SET

C-101 - Existing Conditions Plan

1. Soil test pits SH-TP-105 and SH-TP-106 were both terminated by refusal (on boulders) at elevation 230.5 and 234.5, respectively. Also, there is an exposed surface of ledge shown approximately 10 feet west of TP-106 on this plan. There appeared to be a series of boulders or probably ledge beneath the proposed subsurface infiltration basin B-2B. Soil logs indicated gravelly loamy sand found at both pits, which is considered well drained soil. If these two pits were refusal on boulders and not ledge, should the (estimated seasonal high groundwater) ESHGW be aligned with the nearby wetland surface elevation 228+. The proposed infiltration pipe/stone bed system's bottom of stone is at elevation 229 and with the required minimum 2' separation to bedrock, the bottom of the excavation would be approximately 15 feet below the surface ledge and 3' below the nearby wetland. GCG recommends performing additional soil test pits (with heavy equipment) at the proposed infiltration system B-2B location to clarify any ledge and ESHGW concerns. GCG recommends providing surface spot grades at wetland flags

GCA-26 to GCA-29, which should provide reasonable indication of the actual ESHGW elevation near the area. (The ESHGW elevations found in TP-102 and TP-103 were relatively close to the nearby wetland surface elevation). The proposed SIS-2 bottom of stone at 229.00 is approximately 15 feet below surface (southerly SIS-2 system corner, contour 244), and the minimum bottom of bedrock or ESHWG required to be at least 2' below the bottom of stone bed at elevation 227.0. Where the Sanborn Head showed estimated bedrock elevation contour at 230. Therefore, the southern portion of the SIS-2 would be below the bedrock. Since all subsurface conditions are based on interpolations from available widely spaced explorations, as stated on the Sanborn Head map note #5, 'actual conditions may vary from those shown'. GCG recommends conditioning the applicant to perform additional deep test pits at the southern SIS-2 corner prior to the start of construction and witness by the Town to verify the ledge profile assumption. The applicant has agreed to perform additional deep test pits at the southern SIS-2 corner prior to start of construction which should be witnessed by the Conservation Agent or assigned agent. GCG recommends requiring the addition soil testing as part of the approval conditions.

2. GCG does not recommend installing large infiltration systems in ledge or bedrock. Removal of ledge to allow for infiltration may open up fissures/cracks in bedrock which can redirect flow paths of the groundwater possibly drying up wetland or opening up other avenues for the water to flow. Since this site is directly upgradient of Waverly Street and has a ledge face along the street, the stormwater design and building construction need to prevent any possibility of groundwater breakout through this ledge face onto Waverly Street. Blasting or hammering of ledge could very easily redirect the groundwater flow. See comment #1 above.

<u>C-131 Grading and Drainage Plan.</u>

3. The Legend's CPE (Corrugated Polyethylene Pipe) shall be specified with smooth interior. Plan note #2 added, resolved.
4. The proposed southeastern subsurface infiltration basin label should be B-2B, (B-2A labeled), the northern subsurface infiltration basin label should be B-2A, (B-1A labeled), to match Details A5, plan sheet C-505 and the HydroCAD models. Renamed, resolved.
5. Both subsurface infiltration basin B-1A and B-2B consist of 48" diameter CMPs (corrugated metal pipe) embedded in stone bed, which are identified as Shallow UIC Class V Injection Wells. MassDEP requires a minimum 10 feet setback to water supply line. Both proposed systems are within 10 feet of the proposed water supply line, 10' minimum setback required. The proposed CMP is not a durable material, considering buried in a high moisture environment and 7 to 8 feet below surface. GCG recommends replacing it with more durable pipe material. The water supply line has been relocated; GCG had abundant experience with CMP (including bituminous coated CMP) in drainage system with bottom section rotted away, poorly reacted with the salt and sand treated winter runoff. This is a private development, the applicant has their right to choose the pipe material within the site, CMP is considered durable material. However, HPDE pipe should be considered to eliminate corrosion concerns. GCG concurs that the applicant does have their right to choose any acceptable pipe materials that meets the loading requirements in a private development.
6. Roof drain pipe size (diameter) along the building perimeter should be specified on the plan. Roof drainpipe size shown, resolved.
7. The two subsurface infiltration basins outlet pipe connect to OCSs 2-1 and 3-1, should have the pipe diameter specified on the plan. Resolved.

8. Architecture building roof plan should be provided to show roof runoff collection system directing the portion of roof runoff to the assigned roof drain according to the watershed map. Proposed parking lot garages' roof plan should be provided to ensure the roof runoff drains to the assigned watershed. GCG recommends specifying the architectural roof runoff collection system to be designed according to the watershed sub-catchment divides as part of the approval conditions.
9. The proposed 12" roof drain pipe at the northern side of building connects to WQU 2-1 has a pipe slope of 8%, which would have a flow full velocity of 12.8 feet per second, not accounting the roof height's hydraulic head unless there is a substantially small flow from the roof runoff, the flow velocity is expected to exceeded the 12 feet per second. CBs 2-2, 2-5, 2-12, 3-1 3-2, 3-3 and 3-4 outlet pipes were set at 5.0%, with a flow full velocity of 10.2 feet per second, since the inflow watersheds are relatively small, the flow velocity could be below 10 feet per second. The eastern 12" roof drain connects to DMH 1-4 pipe slope should be 0.015 ft/ft. The plan has utilized 12" diameter drain pipe for the entire site, GCG recommends providing pipe sizing, capacity, and velocity analysis for review. Resolved.
10. All drain manholes should have a minimum of 0.1 feet internal drop to compensate the hydraulic drop within the structure. Resolved.
11. OCS 1-2, an inlet pipe is needed to connect the basin to the OCS to allow outflow at 231.50. Resolved.
12. Bioretention basin surface area did not match the HydroCAD calculations. Bioretention basin eliminated, resolved.
13. WQU 1-1 is a 3' diameter structure with proposed three 12" pipe connection. GCG recommends increasing the structure to 4' diameter. DMH 2-2 and DMH 2-7 should be considered with 5' diameter structure. Detail notes #2 added, resolved,
14. The proposed DMH 1-1 and CB 1-1 rim elevations are 7" +/- above the existing Waverly Street ground grade and should be addressed. Lowered, resolved.
15. The plan stated no ESHGW find at elevation 225' (TP-8). Is the foundation drain at 234.30 necessary? Resolved.
16. The proposed retaining wall at the northern side of CMP infiltration basin B-2B should be designed to support the hydraulic pressure from the nearby CMP infiltration basin and with impervious function prevent breakout though the wall surface. Impervious liner added, resolved.
17. CB2-3 would require removal of six feet of ledge to install which is directly upgradient of the rock slope along Waverly. Address impact or how this would be constructed without potentially opening of a flow path for groundwater directly onto Waverly. CB2-3 removed, resolved.

17a. The OCS 2-1 I.OUT elevation should be 228.95. The labeled 229.05 is higher than the proposed 2"x2" orifice invert at 229.00

C-505 – Details V now C-504 – Details IV

18. Detail C1 – OCS 1-2 "E" elevation should be 223.55. OCS 2-1 – should have only one 1" orifice at elevation 229.15' to match HydroCAD report. Resolved.
19. Detail C1 – OCS 3-1 "A" elevation 231.42' did not match the plan C-131 (232.42 shown). Resolved.
20. Detail C1 – OCS 3-1 "B" elevation 231.46' did not match the HydroCAD (232.46 used). Resolved.
21. Detail C1 – OCS 3-1 "E" elevation 231.42' did not match the plan C-131 (FES = 232.20 shown). Resolved.
22. Detail C1 – OSC inlet pipe size should be specified, where applicable. Resolved.

23. All OCS structures have a standard width of 4' and equipped with 2.0' long sharp-crested rectangular weir (at invert "D"). The details should call out the 2" weir width on the top view and specify the top of the weir wall, where it widened to 4'. Replaced with baffle, resolved.

C-507 – Details VII

24. Detail A5 Bioretention Area Section – the detail shows the bottom of bioretention soil at elevation 216.67, which does not match the HydroCAD storage bottom elevation (215.67), the base of the bioretention basin area should be specified on the plan. Based on the HydroCAD report, the bottom of the bioretention soil (Engineered Soil Mix) area should align with the surface area at elevation/contour 202. This detail calls for 1'-4" depth of bioretention soil (min.), but the HydroCAD calculations were based on 2.33', MassDEP MSH requires the depth of the soil media between 2 and 4 feet, and a minimum of 30 inches for nitrogen removal credit. See additional Bioretention comments under the "Hydrologic Modeling and Supporting Information" below. Bioretention basin replaced by subsurface detention basin, resolved.

25. The proposed surface infiltration basin is constructed in fill; the proposed earth berm is approximately 7.5+/- feet over the existing grade. An earth berm construction detail should be provided, the berm should be constructed with an impervious (low permeability) material core embedded into the existing earth base, pipe should be bedded in the low permeability core material. Surface Detention Basin used, resolved.

Hydrologic Modeling and Supporting Information (NOI Package Appendix 4.02)

HydroCAD report - Existing Hydro:

1. Majority of the existing site surface coverage was modeled as Woods/grass combination, which means 50% woods and 50% grass per Technical Release 55, (TR55), Table 2-2c. Footnote #5. GCG reviewed the historic aerial image from 1995 to current street view image and found that the site consists of predominantly dense woods coverage for the past 30 plus years. GCG recommends replacing the Woods/grass combination surface to Woods, in good condition. The portion of >75% Grass cover surface used in sub-catchments EX 1.2 and Ex 1.3 were deemed reasonable, where relatively matching the open area at the northerly portion of the site as shown on current aerial image. Sub-catchment EX 2.1 Tc should start from the high point, (GCG scaled approximately 200'+/- of total flow length). Minimum Tc should be 6 minutes (0.1 hour) per TR55. However, the Tc in this case should have relatively minor impact on calculations. Resolved.
2. Existing Hydro analysis points: GCG recommends setting the design points (DP) based on the existing runoff flow direction. The existing sub-catchments EX-1.1 and EX-1.2 surface runoff drains toward Waverly Street and flows southwestward to DP-1. However, Sub-catchments EX 1.3 and 2.1, both drains north and northwestward to the existing wetland and intermittent stream (wet flag series GC S1 – S10) and onto the abutting property at 63 Waverly Street. A new design point should be added to the northern property boundary at the intermittent stream. Sub-catchment EX 3.1 drains southwestward onto the East Union Street drainage system (DP-3); Sub-catchment EX2.2 drains southeastward onto East Union Street drainage system (DP-2). There is an existing high point in front of utility pole #20 on Waverly Street. Therefore, majority of the existing project site runoff drains southwestward along Waverly Road with small portion of the site runoff drains northward to the existing catch basin (proposed DMH 1-1). Based on the MassMapper/MassGIS's 1-foot contour layer, there appeared to be some additional watershed areas (0.4+/- acre, since there are grass area and paved surface, which may affect the existing flows toward the existing catch basin) from EX 4.1 and EX 2.1 drains toward EX 1.1; Based on

the existing contour 230 ended near GCA #17 and another contour 230 ended next to GCA #85. (these two contour 230 should be connected at the northwestern side of contour 232 ridge) which indicates that Sub-Catchment X3.1 should flow northwestern to DP-2(EX); Therefore, only EX3.2 flows to the Nikki Street culvert - DP-3(EX); Since EX 5.2 and EX 5.3 pitching toward Waverly Street, should these two sub-catchments be discharged to the Framingham Reservoir DP-4(EX). And only sub-catchment EX 5.1 drains to Union Street Drain System. GCG concurs with this latest HydroCAD modeling; Based on the 100-year storm peak flow rate and runoff volume, which shows the main pre-development runoff discharges to DP-4 (Sheet flow onto Waverly Street, southwestward). (The pre-development site consists of HSG 'A' soil and mostly wooded surface, existing surface runoff during the 2-year and 10-year storm events would be minimal. However, once the site soil saturated during the 100-year storm event, excessive surface runoff flows southwestward as shown on the peak discharge rate summary table.) The 1.15 cfs increase to the peak discharge rate for the DP-1 100-year, 24-hour storm in post-development conditions is significant. (1.15 cfs is equivalent to 8.6 gals of water discharged onto the downstream property every second, with a calculated volume increase of 276,954+/- gallons during the 100-year storm event.)

HydroCAD report - Proposed Hydro

3. As recommended in comment #1 above, all existing Woods/grass combination surface cover to remain should be modeled as Woods, good cover. Resolved.
4. Sub-catchment PR 1.2, this watershed consists of the proposed bioretention basin surface area, since the bottom of the bioretention soil (engineered soil mix media) at 215.67 (HydroCAD), or at 216.67 (shown on Detail A5, C507), discrepancy to be resolved, is below the ESHGW at 217.5 (SB-TP-101). The bioretention area's base will be lined with impervious liner, which eliminated all exfiltration function of the system (become Bio-detention system). GCG is not against modeling the bioretention/biodetention area surface with grass CN value. However, the HydroCAD Pond BIO 1: Bioretention model also takes 35% void credit within the engineered soil mix layer for stormwater storage, then the bioretention area surface (in sub-catchment PR 1.2) should be modeled as water surface CN=98. GCG recommends keeping the bioretention area surface with grass surface CN value and eliminate the 35% void credit used in the pond model BIO 1. Bioretention Basin has been replaced with surface detention system (SDS1, with 42 - SC-310 chamber units). Based on the SH-TP-101, the ESHGW is at elevation 217.5', approximately 78" below surface. The proposed SDS1 bottom of stone at 215.76 is approximately 1.74' below the ESHGW. Buoyancy calculations were provided. There are 6" perforated pipes proposed 10' apart wrapped with filter fabric at the bottom of stone bed. Should the entire system, chambers and stone bed be wrapped in waterproof membrane? The applicant should clarify how the storage volume be protected against ground water seepage (groundwater filled up the stone voids and chambers storage volume during high ground water season.) Buoyancy calculations for the subsurface detention system -1 (Stormtech SC-800 chambers) and surface detention basin were accepted. The applicant should provide details for the impervious liner to ensure watertightness against groundwater seepage.
5. Tc used in TR-55 should be 0.1 hour or 6 minutes minimum. Resolved.
6. Sub-catchment PR 1.3 – The proposed Tc slope should be verified; there appeared to be 3H:1V side slope along the basin earth berm. The Tc with 29' flow length and 2% slope used on the calculation should be verified. The proposed surface detention basin (SDB1) is within sub-catchment PR 1.3. Since the entire detention basin is lined with impermeable liner, not just the bottom area, the Water Surface CN98's area should match the entire lined surface area. Buoyancy calculations should be provided for SDB1, the liner should be designed at the depth that sufficient topsoil weight were provided to counter the buoyancy force during high groundwater season. Resolved.

7. Sub-catchments PR 1.4, the surface infiltration area (100-year event ponding surface) should be modeled as water surface with 98 CN value. The exfiltration rate has been credited to the basin surface area during the pond modeling. See comment 6 above.
8. Pond B-2A: CMP Infiltration – the proposed 2' long weir in a 4' wide concrete structure should have the top of weir wall elevation specified in the design. Now SIS1 consists of one 1" orifice at elevation 229.15, but (Detail C1 – C504) Outlet Control Structure shown two 1" orifice at elevation 229.15. Orifices this small should have some sort of protection around them to prevent clogging or plugging. The latest OCS 1-1 is equipped with a 0.5" diameter orifice and should be equipped with some sort of protection around them to prevent clogging or plugging. OCS 1-2 is equipped with three (3) – 1" diameter orifice, this structure has a 24" diameter inlet flared end section (FES) specified with a trash rack from the surface detention basin. Typical 24" FES trach rack's opening does not screen any small debris, GCG expects vegetation debris collected from the open basin, screen should be provided to protect the 1" diameter orifices.
9. Pond B-2B: CMP Infiltration – the proposed bottom of system (229.00) is 13' below the exposed surface ledge (242 contour) shown on the existing conditions plan. The two nearby test pits (TPs 105 & 106) were terminated due to refusal on boulders (the extent of the boulders should be determined, by additional test pits performed with heavy excavator equipment capable of removing boulders). This is relatively large subsurface infiltration system, buried in ledge should be avoided. The proposed outlet 2" orifice invert at elevation 232.46 is a foot higher than the invert" B" elevation shown on Detail C1, sheet C-505. The proposed 2' long weir in a 4' wide concrete structure should have the top of weir wall elevation specified in the design. Based on the Sanborn Head Bedrock Contour Map, the southwesterly bottom corner of SIS2 could be in the ledge, the applicant should provide additional test pit at the start of construction, or relocate the system out of the bedrock profile. GCG has recommended additional test pit be performed as part of the approval conditions, see C-101, comment #1.
10. Pond B-2C: Surface Infiltration – the proposed 12" outlet pipe length and inverts do not match with plan sheet C131. The proposed 2' long weir in a 4' wide concrete structure should have the top of weir wall elevation specified in the design. Should the proposed 1" orifice and two (2) – 4" orifice be vertical? (horizontal modeled). The proposed outlet orifices appeared to be buried in the earth berm; a connection pipe from the basin to the OCS is needed. The Board-Crested Rectangular Weir dimensions should be specified on the plan with erosion control armor. Emergency spillway sizing calculation should be provided, based on brimful conditions, (no pond storage, no outlet and capable to allow 100-year inflow passing through without overtopping the earth berm), MSH Vol. 2, Ch. 2, Pg. 91. Resolved.
11. Pond BIO 1: Bioretention – The bioretention soil layer thickness (215.67 -218.00 = 2.33') does not match with the Detail A5, C-507, which shown 1' - 4". GCG does not recommend using the 35% voids in the bioretention soil layer for stormwater storage volume, since the bioretention area surface was modeled as grass cover surface in sub-catchment PR 1.4. The calculated bioretention surface area appeared to be larger than the contours surface area shown on the plan. The proposed 2' long weir in a 4' wide concrete structure should have the top of weir wall elevation specified in the design. The proposed bioretention soil/engineered soil mix depth should be between 2' to 4', and 30" minimum depth to qualify for Nitrogen removal credit. No longer apply, resolved.
12. Design Points – the pre-development site runoff mainly flows toward Waverly Street (southwest direction, due to the high point in front of utility pole #20, sub-catchments EX 1.1 and EX 1.2). and majority of the remaining site runoff drains northeastward to the on-site wetland and discharges to the house 65 Waverly Street intermittent stream. There are minor runoff drains southward to East Union Street (east and west directions). Therefore, the CMP infiltration basins B-2A and B-2B overflow to the on-site wetland should be compared with the pre-development flow toward 65 Waverly Street (recommended a new Existing Hydro design point).

The proposed surface infiltration and bioretention area outflows are designed to discharge to the existing catch basin on Waverly Street, which discharge northeastward to the stone box culvert. This discharges to the opposite direction of the pre-development flow pattern, drainage pattern should not be altered especially the downstream outfall discharge to a private property. Moreover, the proposed DMH 1-1 and CB 1-1 rim elevations are approximately 7" higher than the road grade, based on Google street view image, which is infeasible. See Existing Hydro comment #2 above. The proposed PR 5.1 should be compared with EX 5.1, which discharge to the Union Street drainage system DP-5 (PR); and PR 3.3 should be compared with EX 3.2, which drains to the Nikkie St. culvert – DP-3 (PR). If the PR 3.1's 3,605 s.f. 'Grass Cover' to be revised to Meadow finish (specify on plan, no mowing more than twice per year). The CN value should be dropped to 30. And these two design points DP-5 and DP-3 combined peak runoff should have the post-development flow equal to pre-development flow during the 100-year storm event. The rest of the Design Points (DP-1, DP-4, and DP-2) should be able to control to below the pre-development flow by adjusting the numbers of chambers. PR 5.2 and PR 5.3 should be discharge to DP-4, Pr-3.1 should be discharge to DP-2. GCG recommends controlling the post-development peak discharge to DP-1 to below the pre-development rate during the 100-year storm event. The applicant should consider re-directing the excessive runoff back to Waverly Street in the southwest direction. GCG is aware that there is existing deficiency of drainage system downstream at Waverly Street and East Union Street intersection. However, that is the existence and intended to flow through the culvert underneath the railroad tracks.

13. Operation and Maintenance Plan, the catch basin should be inspected, and inlet grate be cleaned at a minimum of 4 times per year. There is no indication of any roof drain collection system shown on the NOI package. The building (main structure and garages) roof collection system, gutter and leader or roof drain inlet should be part of the roof runoff inlet system and should be inspected and cleaned at a minimum of twice per year. Resolved. There is no operation and maintenance for the proposed Porous Pavement, see porous pavement comment above.
14. The proposed development generated more than 1,000 vehicle trips per day as stated in the project narrative and is a Land Uses with Higher Potential Pollutant Loads (LUHPPL), the proposed BMPs appeared to meet the treatment requirements for LUHPPL project. However, the bioretention area should be modified to meet the minimum engineered soil mix layer depth. Does this project generated more than 1,000 traffic trips per day as a LUHPPL? This development will generate approximately 1,032 daily trips and is considered LUHPPL. GCG concurs that the two subsurface infiltration systems meet the 44% pretreatment and 80% TSS removal requirements for LUHPPL uses. However, the proposed subsurface detention system-1 and surface detention basin -1, both systems have achieved the 44% pretreatment credits, (25% deep sump hooded catch basin with 50% WQU credits, combined 65% TSS removal pretreatment credit.) and should be designed as extended dry detention basin BMP, which requires two stages detention with lower stage to retain the 2-year storm for at least 24 hours to remove pollutants from the runoff. (MSH Volume 2, Ch. 2, Pg. 50, Special Features).
15. The bottom of the bioretention area is proposed below the ESHGW, buoyancy calculations should be provided, to ensure there will be sufficient soil weight to counter the buoyancy force. Buoyancy calculations for the subsurface should be provided to determine the depth of the impermeable liner where sufficient topsoil weight to counteract the buoyancy force. Buoyancy calculations accepted, resolved.
16. TSS Removal Calculation Worksheet – this worksheet was not intended to address 90% TSS removal requirements. The Bioretention Area requires pretreatment to achieve a total of 90% TSS removal, (Bioretention/Biodetention soil media thickness should be addressed). The proposed surface infiltration basin and CMP infiltration basin/ (Subsurface Structure). Both require pretreatments to qualify for the 80% TSS removal credit. This project is a LUHPPL and

with rapid soil, the proposed deep sump hooded catch basin (25% TSS removal credit) combined with the WQU (50% TSS removal credit) do meet the 44% TSS removal pretreatment requirements. GCG recommends utilizing the US EPA (Region 1), BMP Performance Analysis (Curve or Extrapolation Tool) to address the 90% TSS removal rate and nutrient removal requirements, as recommended by MassDEP. MSH's TSS removal work sheet is not suitable to demonstrate the 90%TSS removal requirements, as recommended by MassDEP (draft copy of MSH), the applicant should utilize the US EPA -Region 1, BMP performance curve to address the 90% TSS and nutrient removal requirements. Regardless the manufacturer's TSS removal calculations for the WQU rating, the NJDEP assigned only 50% TSS credit for all the common WQU devices. Since this project is classified as LUHPPL the extended dry detention basin BMP, should be equipped with two stages detention with lower stage to retain the 2-year storm for at least 24 hours to remove pollutants from the runoff. See comment 14 above.

Summary

The applicant should verify the ESHGW and boulders/ledge extent at the CMP infiltration system (B-2B) location. The overflow discharges to Waverly Street should be re-evaluated, the existing site runoff flows southwestward to the Waverly Street drainage system, but the post-development overflow drains northeastward to the stone box culvert and altering the pre-development drainage pattern. Any runoff discharge or overflow connection to a municipal drainage system should require the Ashland DPW approval. GCG recommends providing documentation that the existing drainage system conditions and sufficient capacity to handle the development flows. Runoff peak and volume should be controlled when discharges to existing drainage system to avoid downstream flooding. Provided evidence that the blasting or hammering of ledge will not impact the groundwater flow on site which could adversely impact the wetlands and Waverly Street.

As directed by the Conservation Commission, no increase of post-development peak runoff is allowed in comparison with the pre-development flow. If there is increase in the 100-year storm events, downstream flooding analysis should be provided. GCG recommends controlling the post-development peak flow rates to below pre-development flow within the property boundary. As presented, there were substantial peak flow rates increased during the 100-year storm event, which indicated the proposed drainage mitigation is undersized. These latest drainage calculations still show substantial net increase of 1.15 cfs in peak flow rate at the existing stone box culvert in front of 90 Waverly Street during the 100-year storm event and potentially causing flooding (per increased runoff volume). Which does not comply with MSH standard #2. However, there is similar peak rate reduction toward the Waverly Street southwest direction. The applicant should consider maintaining the pre-development runoff flow patterns and control the post-development peak flow to below the pre-development rate.

If you have any questions regarding this matter, please contact our office.

Respectfully submitted,
GCG ASSOCIATES, INC.

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