

02/13/2026

Ashland Conservation Commission
Greg Wands - Chair
101 Main Street
Ashland, MA 01721

61 Waverly Street Comprehensive Permit / 24142

RE: Comment Letter Responses

Dear Mr. Wands and members of the Commission:

SMMA in collaboration with Sanborn Head & Associates, and Goddard Consulting is pleased to provide this memorandum on behalf of the Applicant, The Gutierrez Company, to respond to the comments prepared by GCG Associates in their letter dated February 2, 2026. This letter is GCG Associates second review comment letter. The majority of the original comments from their initial review letter received on December 17, 2025 have been closed – **32 of the 49 Applicant responses stated “Resolved,” or similar comment requiring no further action by the Applicant.** In order to help facilitate review, the following letter will address only the 17 open items.

Comment Responses

General Comments

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-foot 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.

Response: The Applicant maintains, and the record demonstrates, that the proposed project complies with the Massachusetts Stormwater Management Standards and does not result in a material increase in discharge or a material alteration of any protected

resource area, with only a de minimis modeled difference occurring during the 100-year storm event of 1.60 CFS, across all design points. The 12" RCP pipe within Waverly Street can support 2.5 CFS, based on conservative assumptions, more than sufficient capacity to support the Design Point-1 post-development peak discharge rate of 1.80 CFS. (See calculations attached). Notwithstanding this, and without conceding that any mitigation is required under the Wetlands Protection Act, the Applicant is willing, as a voluntary and conditional goodwill measure, to replace the existing 12-inch reinforced concrete pipe within the roadway, subject to issuance of a road opening permit by the Town and provided that the remaining terms and conditions of the Order of Conditions are otherwise acceptable to the Applicant. This replacement is proposed to address an apparent pre-existing infrastructure condition and provide a public benefit. The work shall be authorized under this Order of Conditions and shall not require the filing of a separate Notice of Intent or additional wetland delineation, provided it is conducted in accordance with the approved plans. This voluntary improvement shall not be construed as an acknowledgment of project-related impact or a regulatory requirement.

Prior to construction, the Applicant shall submit to the Ashland Department of Public Works, for administrative review and approval, a complete set of drawings consistent with the approved Notice of Intent plans, detailing the proposed work. All work shall be performed in accordance with applicable Town of Ashland rules, regulations, and building codes. The Town shall reasonably cooperate with the Applicant in the issuance of any required local permits. Upon completion of the work, the Applicant shall provide record ('as-built') drawings to the Department of Public Works. Ownership, operation, maintenance, and long-term responsibility for the pipe shall remain with the Town, and the Applicant shall have no ongoing maintenance obligations or liability related thereto.

GCG Associates Comment #2: 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 H2O 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-foot buffer area. GCG would recommend the replacement of the 12" culvert be designed now as part of the submittal.

Response #2: The Applicant has revised the stormwater design. DEP MSH Standard 2 is met for all design points with the post-development peak discharge rates not exceeding pre-development peak rates for the 2-year and 10-year 24-hour storm events.

The post-development peak discharge rate does not exceed the pre-development peak rate for the 100-year 24-hour storm event for the entire project site (0.07 cfs site-wide reduction). Individual design points DP-2, DP-3, and DP-4 are also all reduced in the post-development model. The post-development peak discharge rate for all on-site and off-site contributing drainage areas to DP-1 (12" RCP within Waverly Street (that the Applicant has offered to replace)) is 1.57 cfs, which is less than full flow capacity of 2.52 cfs. The pipe will be 62.3% capacity during the 100-year 24-hour storm event. The post-development peak discharge rate for the 100-year 24-hour storm event is also well under the stone box culvert capacity of >70 cfs. Although, the DP-5 100-year 24-hour storm peak discharge rate increases by 0.01 cfs in post-development conditions, there is a 0.001 ac-ft decrease in the volume of runoff that discharges in the DP-5 100-year 24-hour storm post-development condition.

A new Detail A1/C-505 is provided to show the proposed 12" RCP pipe replacement. The sketch shows the 100-ft buffer zones associated with the open channel upgradient from the 12" RCP headwall and downgradient from the stone box culvert. The detail includes notes regarding the replacement construction and the coordination and approval process with the Ashland Department of Public Works.

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.

Response: The Applicant acknowledges the peer reviewer's comment regarding the potential presence of an intermittent stream associated with the open drainage swale system upstream of the stone box culvert. As previously discussed, the Applicant does not object to characterizing the open, daylighted swale conveying flow from the on-site wetland toward the culvert as an intermittent stream for purposes of this review.

However, under the Wetlands Protection Act and its implementing regulations, the identification of a potential intermittent stream is a threshold consideration and does not, in and of itself, establish that the proposed project will result in a material alteration of a protected downstream resource area. The applicable regulatory standard under 310 CMR 10.02(2)(d) and 10.05(6)(b) requires a showing that a project activity is reasonably likely to alter a protected interest, and that any limits on discharge quantity or quality are necessary to protect those interests.

In this case, the project's stormwater system has been designed in accordance with and evaluated for compliance with all applicable Massachusetts Stormwater Management Standards, based on predictive hydrologic modeling. The modeling demonstrates no increase in peak discharge to the on-site wetland or intermittent stream system for the 2-year and 10-year storm events, with only a de minimis incremental discharge occurring during the 100-year storm event. The project discharges to an enclosed municipal storm drainage system that conveys flows through existing pipe and culvert infrastructure prior to any downstream daylighting, providing attenuation and mixing with existing watershed contributions.

Based on the de minimis magnitude of the incremental discharge, the enclosed nature of the intervening conveyance, and the absence of any site-specific evidence or analysis demonstrating that the project's incremental flow would be reasonably likely to cause a material alteration of a downstream protected resource area, the Applicant respectfully submits that additional assessment of intermittent stream jurisdiction at the stone box culvert itself is not warranted. Accordingly, while the Conservation Commission may review activities where necessary to protect the interests of the Act, the record does not support a conclusion that the proposed project will materially alter a protected resource area or require additional discharge limitations beyond those already incorporated into the project design.

GCG Associates Comment #2: As stated in comment #4 above, the peak rates increase during the 100-year storm event do not qualify for de 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-foot buffer of the wetland resource area at 90 Waverly Street.

Response #2: Refer to Response #4 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.

Response: The Applicant acknowledges the peer reviewer's observations regarding the existing stone box culvert and notes that it is a long-standing component of the municipal drainage system that predates the proposed development and conveys runoff from multiple upstream sources. As discussed in Response to Comment 5, the relevant inquiry under the Wetlands Protection Act is whether the proposed project is reasonably likely to cause a material alteration of a protected resource area or interest.

The project's stormwater system has been evaluated through predictive modeling and demonstrates compliance with all applicable Massachusetts Stormwater Management Standards, including no increase in peak discharge to the on-site wetland or associated open swale system for the 2-year and 10-year storm events, and only a de minimis incremental discharge during the 100-year storm event, conveyed through an enclosed municipal drainage system prior to reaching the culvert. No site-specific analysis or evidence has been identified demonstrating that this de minimis incremental discharge would be reasonably likely to impair the structural integrity of the existing culvert or alter downstream wetland resources.

Accordingly, while the existing condition of the culvert is acknowledged, it does not establish a basis to prohibit discharge from an otherwise compliant stormwater system or to require additional wetland delineation beyond that already shown on the approved plans.

GCG Associates Comment #2: 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-foot buffer (per 90 Waverly Street NOI documents) any increased peak runoff rate entering the culvert would potentially exacerbate the culvert's structure integrity.

Response #2: The contributing drainage area to the existing stone box culvert includes discharge from many surrounding properties. The project's peak discharge rate to the stone box culvert from the 100-year 24-hour storm event is 1.98 cfs which is approximately 2.78% of the estimated culvert capacity of 71.2 cfs. The Applicant does not agree to perform structural reinforcement of the town's infrastructure based on the project's minimal contribution.

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).

Response: SH-TP-105 and SH-TP-106 were excavated to refusal on boulders at approximate depths of 5.5 and 7.5 feet, respectively. The determination of refusal as boulders (as opposed to ledge) at these two locations was based on the shallow depth relative to SH-TP-8 and SH-TP-7, as well as the excavator response/interaction with the in-place soil, such as localized pitting of the excavation bottom and/or movement of the excavation bottom and sidewall. In general, surface and buried boulders can also be commonly found across the site as noted in the subsurface logs. In particular, a large surface boulder up to 6 feet in diameter can be found west of SH-TP-106. Sanborn Head excavated at the base of this boulder to confirm this was not a surface bedrock protrusion and encountered a soil subgrade – refer to the photographs in Attachment 5.

Refusal on bedrock was encountered to the west of the proposed basin B-2B, renamed SIS-2, now at El. 225 feet in SH-TP-8. Bedrock is anticipated to climb in elevation as you move further west and toward the central ridge, and drop as you move east towards proposed basin B-2B.

Consistent with Sanborn Head's data report, SHA further notes that bedrock or large boulders encountered at the subgrade for the stormwater system at the time of construction will need to be removed to allow for installation of the system and to create

a uniform, free-draining layer below the system for even distribution of the groundwater recharge. Based on the absence of groundwater observations and redoximorphic features in the explorations within and in the vicinity of proposed B-2B, renamed SIS-2, ESHGW is anticipated to be at El. 225 feet, or lower.

The wetland in the north corner of the site appears to be a 'losing' system with respect to groundwater, meaning it adds to the groundwater rather than be fed by it. The wetland generally conveys surface runoff and discharge from the upgradient wetland system to an outlet stream to the north, with small amounts of water entering the ground below and at the fringe of the system as groundwater, and develops a dipping (or losing) gradient away from the wetland edge. Over time, fine sediments and plant detritus build up a low permeability layer along the bottom and edge of the wetland, limiting groundwater recharge and causing the wetland to be 'perched' on the landscape. This is seen by the redoximorphic features in SH-TP-102 and SH-TP-103, at approximately El. 225 feet, being 'downgradient' from the wetland edge at approximately El. 228 feet. Similarly, this would be the likely condition further south along the wetland edge when approaching proposed B-2B, renamed SIS-2. This is further supported by the general lack of observed groundwater; the existing wetland cannot 'gain' from groundwater when the groundwater is not consistently present above the bedrock. We therefore believe that extending the wetland surface elevation (of El. 228 feet) 100 or more feet to the west, is not consistent with the hydrogeologic conceptual site model.

GCG Associates Comment #2: 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.

Response #2: The Applicant agrees to perform additional deep test pits at the southern SIS-2 corner prior to start of construction which shall be witnessed by the Conservation Agent or the Town's Peer reviewer.

C-131 – Grading and Drainage Plan

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.

Response: Drawings C-131: Grading & Drainage Plan and C-141: Utilities Plan are revised to shift the water main at least 10 feet from the subsurface systems. Corrugated metal pipe (CMP) is an industry standard material for infiltration systems and 48-inch diameter pipe can be buried to depths far greater than those proposed. Detail A5/C-504 is updated to provide an impermeable membrane liner over the top of CMP pipes to reduce the risk of corrosion from salts in the parking areas above the systems.

GCG Associates Comment #2: 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.

Response #2: The Applicant and design team have selected the corrugated metal pipe (CMP) with added geomembrane liner (to protect against salt corrosion) based on performance and life span (40 to 70+ years). The selected product is made from Aluminized Type II Steel which is superior to standard galvanized steel in hydraulic environments. The Applicant would like to preserve its options in the selection of the pipe material to address factors at the time of construction such as availability, labor, and cost (tariffs). However, alternative materials such as HDPE pipe will be considered prior to installation as these variables are subject to change.

17. **New GCG Associates Comment: 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.**

Response: As requested in order to avoid or minimize bedrock removal, the parking area has been re-graded to redirect runoff to CB 2-2 and CB 2-4. Drawing C-131: Grading & Drainage Plan is revised accordingly.

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

18. Detail C1 – OCS 1-2 “E” elevation should be 223.55.

Response: The details and hydrology model have been updated accordingly.

GCG Associates Comment #2: OCS 2-1 – should have only one 1” orifice at elevation 229.15’ to match HydroCAD report.

Response #2: The OCS 2-1 orifice elevation is coordinated between the details and the hydrology model.

Hydrologic Modeling and Supporting Information (NOI Package Appendix 4.02)

HydroCAD Report – Existing Hydro

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. Subcatchment 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).

Response: The hydrology model has been revised to incorporate the recommended Design Points. The enclosed calculations reflect these revisions.

GCG Associates Comment #2: 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.

Response #2: The hydrology model is updated to incorporate comments from GCG Associates. The project is in compliance with DEP MSH Standard 2 - refer to more detailed response in #4 above.

HydroCAD Report – Proposed Hydro

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.

Response: The bioretention area has been replaced with a subsurface detention system.

GCG Associates Comment #2: 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.)

Response #2: As recommended by GCG, SDS-1 is updated so the entire system is fully wrapped in a waterproof membrane. The enclosed buoyancy calculations are updated accordingly.

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.

Response: The hydrology model has been revised to incorporate the recommended time of concentration. The enclosed calculations reflect these revisions.

GCG Associates Comment #2: 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.

Response #2: The hydrology model is revised to provide a CN value of 98 for the surface of the surface detention basin (SDB-1). Enclosed are buoyancy calculations for the liner beneath SDB-1.

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.

Response: The hydrology model and drawings have been revised to incorporate the recommended weir design.

GCG Associates Comment #2: 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.

Response #2: The orifice within OCS 2-1, now 2”x2”, is downstream of the entire treatment train consisting of deep sump hooded catch basins, pre-treatment water quality units, and the subsurface infiltration system. It is not anticipated that debris that could clog the orifice will reach this end of the treatment train.

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.

Response: Refer to response above regarding the geotechnical explorations at subsurface infiltration system #2 (formerly B-2B).

The hydrology model and drawings have been revised to incorporate the comments regarding orifices and weirs.

GCG Associates Comment #2: 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.

Response #2: The Applicant agrees to perform additional deep test pits at the southern SIS-2 corner prior to start of construction which shall be witnessed by the Conservation Agent or the Town’s Peer reviewer.

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 predevelopment 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.

Response: The hydrology model has been revised to incorporate the recommended Design Points. The enclosed calculations reflect these revisions.

GCG Associates Comment #2: 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.

Response #2: The hydrology model is updated to incorporate comments from GCG Associates. The project is in compliance with DEP MSH Standard 2 - refer to more detailed response 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.

Response: The bioretention area has been replaced with a subsurface detention system.

GCG Associates Comment #2: Does this project generated more than 1,000 traffic trips per day as a LUHPPL?

Response #2: Langan Engineering & Environmental Services, LLC, on behalf of the Applicant, prepared a Traffic Impact Assessment (TIA), to evaluate the impact to traffic of the proposed Residences at Ashland. The TIA concluded the proposed development will generate approximately 1,032 trips per day and therefore is designated a Land Use of Higher Potential Pollutant Load (LUHPPL).

Greater than 44% TSS pretreatment is provided for the infiltration basins by deep sump hooded catch basins and WQUs. Utilizing structural BMPs determined to be suitable for treating runoff from a LUHPPL (subsurface infiltration systems), in addition to the pretreatment BMPs, TSS removal of greater than 80% is achieved for the proposed site. The enclosed pollutant removal calculation worksheet confirms 63% TSS pretreatment is provided to the subsurface infiltration systems, SIS-1 and SIS-2, by deep sump hooded catch basins and WQUs (assuming a 50% TSS removal rate, as recommended by GCG). Additionally, 89% TSS removal for the site is provided by deep sump hooded catch basins, WQUs (assuming a 50% TSS removal rate, as recommended by GCG), and the subsurface infiltration systems, SIS-1 and SIS-2.

All BMPs are designed in accordance with the Massachusetts Stormwater Handbook Volumes 2 and 3 and are sized to treat a water quality volume of 1” times the proposed impervious area. Treatment provided by the proposed structural and non-structural BMPs are in compliance with Standard 5 for Land Uses with Higher Potential Pollutant Loads

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.

Response: The bioretention area is revised to a subsurface detention system – refer to detail C5/C-504: Subsurface Detention System (Stormtech SC-310). The bottom of system is at elevation 215.76 which is below the SH-TP-101 observed seasonal high ground water at elevation 217.5. Buoyancy calculations are enclosed that demonstrate a factor of safety against floatation of 6.0.

GCG Associates Comment #2: 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.

Response #2: Enclosed are buoyancy calculations for the liner beneath SDB-1.

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.

Response: Enclosed are TSS removal worksheets that demonstrate both the pre-treatment only TSS removal and the entire treatment train TSS removal for each stormwater BMP.

GCG Associates Comment #2: 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.

Response #2: Standard 4 of the Massachusetts Stormwater Handbook requires stormwater management systems to provide 80% removal of the average annual post-construction load of Total Suspended Solids (TSS). The enclosed pollutant removal calculation worksheet confirms 89% TSS removal is provided by deep sump hooded catch basins, WQUs (assuming a 50% TSS removal rate, as recommended by GCG), and the subsurface infiltration systems, SIS-1 and SIS-2.

Summary

The applicant should verify the ESHGW and boulders/ledge extent at the CMP infiltration system (B2B) 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.

Response: Responses to the above comments have been provided throughout the memo.

GCG Associates Comment #2: 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.

Response #2: The stormwater design and hydrology model have been revised as described in the responses throughout. The Project shows no increase in post-development peak discharge flow in the 2-year and 10-year 24-hour storm events and now has a net reduction in the site-wide 100-year 24-hour storm event. MSH Standard 2 is met.

Sincerely,



Will Park, PE

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CC: The Gutierrez Company

Enclosures: Revised Site Permit Plans, Existing Hydrology Report, Proposed Hydrology Report, Existing Hydrology Map, Proposed Hydrology Map, Peak Discharge Rate Summary, Pollutant Removal Worksheets, SDS-1 and SDB-1 Buoyancy Calculations, Rip-Rap Sizing Calculations