Cornerstone Land Associates, LLC
Civil & Structural Engineering • Permitting
Land Surveying • Land Development & Planning
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October 10, 2024

Scott Malory, Chairman
Dracut Zoning Board of Appeals
62 Arlington Street
Dracut, MA 01826

Ref: Greenmont Commons
Comprehensive Permit Peer Review
135 Greenmont Avenue
Dracut, MA 01826

Dear Mr. Malory,

Cornerstone Land Associates, LLC (CLA) is in receipt of a letter from GCG Associates (GCG) dated September 30, 2024 addressed to Ms. Alison Manugian, Community Development Director of the Town of Dracut. The letter is an Engineering Peer Review of the Site Plans, Details and Stormwater Management Report submitted to the Town of Dracut Zoning Board of Appeals for the above referenced project. The purpose of this letter is to provide CLA Responses to the GCG Peer Review Comments within the aforementioned received letter. To better clarify the Comments and Responses, GCG's latest Comments will be in *italicized red* format and CLA's latest Responses will be in **Bold Black** format.

## REVIEW SUMMARY:

***3.*** *This latest plan set has lowered the site by approximately a foot. However, the two proposed Water Quality Units (WQU) top of concrete is above the driveway finish grade. The proposed retaining wall enclosed wetland forebay has created an accessibility issue for maintenance. This wetland forebay should be cleaned once per year per MSH requirements. The sole maintenance access path would be through the 10 feet wide path along the eastern property line. The applicant should demonstrate a valid accessible path to the southeasterly lot corner and turnaround area without encroaching to the abutting properties.*

**Our office has revised the location of SWQU#1 to the landscape island in front of Unit 26 with a Rim elevation of 162,60 which is consistent with the finish grade in that location. The location of SWQU#2 has also been relocated to the landscape island in front of Unit 13 with a Rim elevation of 162.50 which is consistent with the finish grade in that location. Access to the Sediment Forebay for maintenance can be completed from the upper area of the maintenance access with the use of a Vacuum Truck as necessary and outlined in the Operations & Maintenance Manual.**

***11.*** *The proposed 6" diameter water service main is being proposed 5 feet from the face of the building stairs along the back side of the two larger buildings. GCG recommends increasing the separation between the water service main and stair structures to 10 feet minimum. (There is not enough room for the construction equipment's swing radius, same situation with the sewer line to the retaining wall separation). Flow tests for water supply and fire protection capacity should be performed to ensure there is sufficient capacity to support this project. The system should be reviewed and approved by the Water District. The proposed water services remain too close to the accessible building (6') and the multi-unit building rear stairways (5'). GCG recommends providing 10 feet separation between the proposed 6" water services to any permanent structures to allow construction equipment maneuverability. Not addressed.*

As requested, the sewer main and the water service main at the rear of the proposed dwellings will be 10 feet from the structures, decks, and stairs, 10 feet apart from each other, and 10 feet from the retaining wall allowing sufficient room for future repairs. Upon approval of the Comprehensive Permit, the project team will work with the Dracut Water Supply and the Dracut Fire Department to determine sufficient water capacity.

*GCG recommends incorporating the Flow Tests to satisfy Fire Department and Dracut Water Supply District/Water Department requirements as part of the approval conditions.*

**Upon approval of the Comprehensive Permit, the project team will work with the Dracut Water Supply and the Dracut Fire Department to determine sufficient water capacity.**

***12.*** *The proposed water line at the southerly end of the building should be relocated further southward to provide 10 feet separation to the sewer services for units 14 and 15. Resolved. The southern end of the proposed water line does not meet the minimum 10 feet setback to the newly proposed Units 13 and 14's subsurface roof drain chamber system.*

The subsurface roof drain chamber system has been removed. The sewer main line and the water main line have been relocated to provide 10 feet from all structures, decks, stairs, and retaining walls.

*GCG recommends centering the water line next to Units 13 and 14 building corners to provide a minimum of 10 feet clearance from any structures.*

**The rear access stairwell has been relocated to the northern side of Units 13 & 14 to allow for the recommended 10 foot clearance from all structures.**

***14.*** *Proposed sewer connection core invert at the existing sewer manhole on Greenmont Avenue should be specified.* Resolved.

*This latest plan has lowered the sewer connection core invert at the same elevation as the existing live sewer invert. Which would most likely interrupting the live sewer line during construction. The installation should comply with the Dracut DPW's sewer connection permit requirements.*

**The contractor will coordinate with the Town of Dracut DPW Department for the installation of sewer connection to ensure there is no interruption in the sewer line during construction.**

***15.*** *The proposed sewer main and services pipe sizes, pipe length and slope should be called out on the plan. Sewer service inverts at each building should be provided. The proposed finish floor elevations indicated stepped foundations for the two larger buildings.* *Resolved.*

*P-SMH#3 INVin and INVout elevations were transposed.*

**The inverts for P-SMH#3 have been corrected.**

***26.*** *The proposed westerly retaining wall is about 10.5 feet in height, and the proposed 240 feet length row type buildings are 3-story's with a physical height of 39.5+/- feet height (actual roof height should be adjusted with the stepped foundation, GCG is expecting some roof line break with the stepped foundations). The combined retaining wall and building structure will exceed 50 feet in height for 240 feet length at the westerly portion of the site and approximately 46.5 feet height at the easterly portion of the development. GCG recommends having the project Architect provide elevations renderings to demonstrate the visual impacts from the abutting neighbors. Since this is a massive structure (50' x 240' ~ 12,000 square feet) located closely next to the westerly property line (11+/- to 24+/- feet), the Architect should provide impact assessments to the sunlight, wind flow and any other natural impacts affected by this development.*
*GCG recommends providing elevation plans with the existing abutters buildings and proposed development in same scale. The lowered site with 40+/- feet building remains 48+/- feet higher than the adjacent lots. Shade, sound, and wind impacts to the abutting property analysis should be provided. Not addressed.*

*This latest plan has proposed to lower the finish grade by approximately a foot, which is lower than the two WQU concrete structures concrete tanks and should be addressed. The retaining wall proposed outside the constructed wetland sediment forebay, and wetland basin should be designed to allow BMP's maintenance access without encroaching the abutting properties*

**The two SWQU have been relocated with adjusted Rim elevations that are consistent with the finish grades in that area. The Constructed Wetland can be maintained from the rear parking area from the rear 10 ft maintenance access as maintenance for this BMP consists of vegetation removal and replacement if necessary which is completed by hand and not machine.**

***31.*** *The proposed subsurface infiltration chambers system is classified as Shallow UIC (Underground Injection Control) Class V Injection Well, which should comply with the MassDEP Energy and Environmental Affairs (EEA) Standard Design Requirements for Shallow UIC Class V Injection Wells. The proposed Cultec Recharger R-330XL chamber system (Class V well) requires a 10 feet minimum set back to the rain gardens (open, surface drain). The proposed separation between rain gardens is 24 feet (driveway width), the proposed 25 feet wide chambers infiltration system encroached into the rain gardens. The subsurface infiltration system has been replaced with a pipe detention system due to the poor draining soil conditions. The proposed detention system consists of 24" HDPE pipe (plan should call out/specify perforated pipe to allow water flow to crushed stone void) embedded in crushed stone bed. However, 2.39' out of the 3' height (elevations 156.0 to 158.0) detention system is below the ESHGW (158.39, TP#6). During the high seasonal groundwater months, the system should be filled with groundwater and deemed useless. GCG recommends raising the bottom of the detention system/crushed stone to one (1) foot minimum above the ESHGW. There were some major discrepancies with the proposed pipe detention system as shown on plan and the HydroCAD calculations, see additional comments under the Stormwater Report Review. The proposed 300 feet* *length x 6 rows of 30" pipe detention system (approximately 8,836 cubic feet volume) is partially submerged below the ESHGW. Based on the 7 soil test pits performed on site with ESHGW depth between 2.5' to 3.2' below surface. GCG recommends replacing the ADS pipes with either PVC pipe or ductile iron gasketed pipe. (The system should be tested to withhold up to 150 PSI pressure as a watertight system). System buoyancy calculations should be provided. Roof recharge system - 6 of the 7 test pits consisted of very fine sand like Test Pits 1 & 2, which consist of 39.5% and 39.0% high silt and clay content (particle passed the #200 sieve), respectively, per GeoTesting laboratory Particle Size Analysis reports. The soil pits were classified as sandy clay. Permeability tests were performed with reported 0.00031 cm/sec (0.0439in/hr) and 0.000078 cm/sec (0.1106 in/hr), respectively. (MassDEP requires a 50% reduction of the in-situ rate using "Dynamic Field" method, MSH Vol.3, Ch.1, Pg.23). Test Pit #3 test results show lower silt/clay content (21.3%) and a higher permeability rate, 0.00076 cm/sec (1.077 in/hr). However, Test Pit #3 was located within the 50 feet wetland (BVW) buffer and did not meet the infiltration system's minimum 50' setback requirements. Therefore, the two reduced (50%) exfiltration rates (TP-1 - 0.22 in/hr and TP-2 - 0.055 in/hr) should be utilized for the on-site infiltration systems design. Standard engineering practice would utilize the lowest infiltration rate as a conservative/safety approach. In addition, the applicant should analyze the roof recharge system releasing roof runoff next to (3 feet +/-) the pipe detention system over poorly drained soil with proximity to the ESHGW in the buoyancy calculations. Furthermore, the proposed roof drain recharge chambers did not meet the minimum 25 feet setback to other subsurface discharge structures, per MassDEP Energy and Environmental Affairs (EEA), Standard Design Requirements for Shallow UIC Class V Injection Wells (Pg. 2 of 4, footnote #5). The pipe detention system outlet pipe is shown ending at the edge of pavement, which was supposed to discharge to the wet forebay. Drainpipe size and slope should be labeled on the plan.*

Our office reached out to the ADS pipe manufacturer and they have provided a Technical Note TN 5.07 Post Installation Testing for HDPE Pipe regarding the Water Tight testing that is conducted on a system of this nature attached herewith. In addition, ADS has provided our office with an additional Technical Note TN 5.05 Pipe Flotation that identifies the buoyancy requirements of a pipe system within the water table attached herewith. The requirement to eliminate floating of a 30" HDPE pipe completely submerged in the water table is 22" of cover. The proposed system is 1.4' within the estimated seasonal high water table and has 30" of cover minimum.

*The Technical Note TN5.07 mentioned in the response letter was not included in the latest drainage report. GCG did find the TN5.07 Post Installation Testing for HDPE Pipe through the ADS pipe website dated May 2022. The document stated that "Allowable Leakage – The allowable leakage rate for HDPE is 200 gallons/in-dia/mi-pipe/day for both infiltration and exfiltration when tested in accordance with ASTM F2487" and the Conclusion stated that "HDPE pipes is intended for gravity flow drainage applications and may be tested for deflection and joint tightness as discussed in this technical document. It is important to note that the testing procedures are no different than for other gravity flow drainage products currently being used in the market." Based on the ADS pipe's allowable leakage standards, the proposed 1,800 feet (0.34 miles) 30" diameter pipe would allow 2,040 gallons of water infiltration and exfiltration through the system per day, when it is fully submerged. This proposed pipe system invert is approximately 1.5 feet below the ESHGW. GCG estimated over a 1,100 gallon of groundwater infiltration entering the storage per day during high groundwater conditions. This water would continuously discharge downstream through the outlet control structure during wet months. In comparison, a standard sewer pipe system leakage allowance is not to exceed 100 gallons/in-dia/mi-pipe/day for both infiltration and exfiltration with a positive head of 2 feet, approximately half of the HPDE pipe standard. Therefore, the HDPE pipes are not suitable for a watertight drainage storage system use. The HDPE pipe system should either installed above the ESHGW or to be watertight. The TN 5.05 Pipe Flotation Technical Note Table 2 dated May 2022,* *GCG downloaded from the ADS pipe website, stated that the 30" diameter HDPE pipe would require 22" minimum cover to prevent flotation of the pipe. The applicant should show the buoyancy calculations and certify the proposed pipe detention system meets the requirements.*

**In order to eliminate the potential of infiltration of the groundwater within the ADS pipe system, the proposed excavation area will be lined with a one piece 30 Mil PVC liner that will be seamed at the factory with electro-welding to ensure there will be no leakage. All connections or penetrations to and from the ADS pipe system will be installed with a waterproof boot supplied by SolMax Geosynthetics and EPI.**

***32.*** *Drainpipes from PDMH #2 to PDMH #5, RG#6 and RG#12 to PDMH #4 are back pitched. GCG recommends connecting a single outlet pipe from the Pipe Detention System to the OCS (outlet control structure) and installing a baffle wall with the specified cored outlet orifices inside the OCS to control the outflow rates, as modeled in the HydroCAD calculations. Roof drain chambers should be equipped with cleanout/inspection port for inspection and maintenance. Calculations utilized 4" outlet pipe connecting the roof chambers to the pipe detention system, plan should show the 4" pipe connection between roof recharge chambers and pipe detention system. Please provide connection details.*

Based on the Soils Analysis, our office is in agreement that underground infiltration on this property is not feasible. The proposed Roof Runoff downspouts will be directed to the landscape areas to promote infiltration of the clean stormwater through the lawn areas.

The roof drain connections have been eliminated. Roofs runoff will be discharged onto the pavement or lawn area and overland flow to the catch basins. The applicant should verify the catch basins #3 and #4 grate flow capacity and considering installing double grate inlet at these locations.

**The maximum flow of stormwater from the HydroCAD model to both Catch Basins #3 & #4 is approximately 2.5 CFS and therefore a single grate Catch Basin is sufficient.**

***33.*** *The plan should show buildings front roof drain connection to subsurface chamber infiltration system. Roof drain detail should be provided to collect and discharge the front and rear building roofs runoff to the pipe detention system. Additional roof drainpipes and leaders should be provided for the two 9-unit buildings. The two side yards are almost level (proposed contours show less than 0.5% slope along the longitudinal lawn surface. The proposed retaining wall should be equipped with a cap to channel the surface runoff to catch basins. High point/ridge spot grades should be provided between buildings to match the sub-catchment divide. Retaining wall capstone should be shown on all wall section views to avoid confusion during construction. The wall capstone provides the function to trap surface runoff and direct the flow to the designated drainage inlet. The architectural plan shows a roof ridge along the length of the two 9 units buildings. Plan needs to show how to connect the rear roof drain to the roof chambers system in the front of the building.*

The proposed drainage system and retaining wall have been revised. The buildings roof drains will drain into the proposed landscape areas in the front and rear of the buildings. Any overflow that may occur from the landscape area will be collected in the closed drainage system proposed.

*The roofs runoff has been revised to drain onto the lawn surface, due to the relatively flat lawn slope (0.5% slope). Water ponding over the lawn area would be expected with the poorly draining HSG 'D' soil and shallow groundwater conditions. The retaining wall drain tile day-light locations should be specified.*

**All areas where roof top runoff downspouts will be routed to will be newly landscaped areas having a minimum of 1-2 feet of clean granular fill over the existing grade which has a ESHWT depth of 2-3 feet allowing for sufficient volume in all storm events to eliminate surface ponding.**

***36.*** *The proposed detention basin Pond #1 earth berm is constructed in fill, not recommended. GCG recommends utilizing lowering the detention basin with impervious liner or utilizing a wet basin or construction wetland to provide the detention storage. If the applicant insisted to construct the earth berm in fill, the earth berm should be widened with a properly designed impervious core, the 4" or 6" outlet pipe (Drainage Chart called for 4" Low Flow Drain, plan called for 6" Pipe Outlet, need clarification.) should be constructed with an anti-seep collar. The 4" or 6" outlet pipe should be included in the drainage calculations. The outlet pond and the sediment basin/forebay bottoms are below ESHGW. Outlet Pond #1 bottom elevation at 154.0 is below the ESHGW at 154.47 (TP#3); the entire sediment basin, bottom elevation at 156.0 and spillway weir invert at 156.50 (per calculations) are both below the ESHGW at 157.34 (TP#2). The applicant should clarify the intension and function of the sediment basin. The plan should call out the basin outlet weir invert elevation, (156.5 was used in the calculations). The sediment basin is below ESHGW and does not provide any stormwater storage volume. If the sediment basin is intended to collect sedimentation, it should be designed and sized as a wetland forebay according to the MSH Table CSW.1 (Constructed Stormwater Wetland), Vol. 2, Ch. 2, Pg. 43, requirements. Pond #1 has a proposed outlet broad crested weir invert at 155.50, the bottom of the basin is below ESHGW and with little to no exfiltration available, once the basin filled with surface runoff, it would not be able to drawdown within 72 hours for back-to-back or multiple storm events. Therefore, storage volume should not be accounted below the weir invert at 155.50. The stormwater storage within the forebay and construction wetland basin should not be credited. There is no other outlet other than the spillway weir, there would be no drawdown of the basin water other than evaporation, which is not reliable for the storage volume available for the next storm event. See additional comments in the stormwater report below. Maintenance access path should be provided.*

The proposed Sediment Forebay and Extended Detention Wetland Basin have been modeled with 6" Low Flow Drain culverts. No storage credit was utilized in the Drainage Modeling below the elevation of the Low Flow Drain. Both areas below the culvert have been modeled as a water surface with a CN of 98.

*The proposed retaining wall outside the wet sediment forebay has blocked the maintenance access. This now requires passing through 100 feet of the constructed wetland to provide maintenance and should be addressed. See additional Stormwater Report comments below.*

**Access to the Sediment Forebay for maintenance can be completed from the upper area of the maintenance access with the use of a Vacuum Truck as necessary and outlined in the Operations & Maintenance Manual. The Constructed Wetland can be maintained from the rear parking area from the rear 10 ft maintenance access as maintenance for this BMP consists of vegetation removal and replacement if necessary which is completed by hand and not machine.**

***52.*** *Provide an Outlet Control Structure detail drawing. The structure should be designed to fit two (2) – 6" diameter orifice outlets (per plan C-103) and/or additional 2" outlet orifice, which is modeled in the HydroCAD calculations but not mentioned on the plan. The HydroCAD calculations called for three* *outlet pipes (4”, 6”, and 12”) from the pipe detention system to the Outlet Control Structure (OCS). The calculations have also modeled the outlet pipes as vertical orifices. (Pipe outlet should be modeled as culvert, which has a different flow entrance coefficient). GCG recommends installing the outlet control orifices inside the OCS with a baffle wall and cored openings. The Outlet Control Structure detail’s 12” core invert is a foot lower than the invert used on the calculations. Vortechs model should be specified to the associated WQU unit and provide support sizing calculations.*

The Outlet Control Structure Detail has been revised to match the HydroCAD model. The Vortechs units have been identified on the revised Grading & Drainage Plan and supporting documentation for the Votechs Units has been provided attached herewith.

*The two WQU units top of concrete is 4.67’ above the pipe inverts, and above the proposed pavement finish grade and should be addressed*

**The two SWQU have been relocated with adjusted Rim elevations that are consistent with the finish grades in that area.**

***7.*** *Outlet Det. Pond did not include the 4” low flow drain (applicant to clarify the size of the outlet pipe, plan drainage chart called 4” pipe, but plan label specified 6” pipe) in the calculations. The proposed Sediment Basin is below ESHGW, storage volume credit is invalid. Outlet Pond #1 bottom is below ESHGW, hence, no infiltration credit. The storage volume below the outlet weir invert without infiltration should not be counted, as it would not meet the 72-hour drawdown requirements. The HydroCAD calculations should ignore the stormwater storage volume below the spillway for the wet forebay and constructed wetland, since both sumps do not have any drawdown capacity. (The bottom of constructed wetland and wet forebay are designed to set below ESHGW).*

The proposed Sediment Forebay and Extended Detention Wetland Basin have been modeled with 6” Low Flow Drain culverts. No storage credit was utilized in the Drainage Modeling below the elevation of the Low Flow Drain. Both areas below the culvert have been modeled as a water surface with a CN of 98.

*The wetland basin outlet pipe invert at 155.00 is below the existing wetland boundary grade (above 155 contour). The wet basin is within the 25-feet no disturbance buffer, which requires Conservation Commission approval. Furthermore, no stormwater storage credit should be taken below the board crest weir elevation 155.50*

**The area at the outlet pipe of the wetland basin has a surveyed elevation of 154.7 which is sufficient for the daylighting of this pipe. The HydroCAD model has been revised to ensure that no storage credit is taken below elevation 155.50.**

***10.*** *Provide detention Pond emergency spillway sizing calculations, at brimful conditions. Spillway sizing calculations with brimfull conditions should be provided. The calculations are necessary to ensure the emergency overflow would contain within the erosion protected armor channel and would not overtop and washout the earth berm. Construction Wetland and wet forebay surface area should be modeled as water surface (CN=98) in sub-catchments 5S and 36S.*

The proposed Sediment Forebay and Extended Detention Wetland Basin have been modeled with 6” Low Flow Drain culverts. No storage credit was utilized in the Drainage Modeling below the elevation of the Low Flow Drain. Both areas below the culvert have been modeled as a water surface with a CN of 98.

*Emergency spillway sizing calculations should be provided, with brimful conditions. The spillway shall be sized to avoid any overtopping through the top of berm.*

**Spillway Sizing calculations have been provided in the revised Stormwater Report.**

***11.*** *Operation and Maintenance (O&M) plan (during construction period and long-term) should be included in the stormwater report. O&M plan should identify the responsible party of the O&M plan, with estimated annual operation budget and sample O&M log. A new O&M plan for the revised system should be provided. A new O&M plan should be updated for the revised system.*

A Revised Operation & Maintenance Manual has been provided with this submission.

*No new O&M plan provided.*

**A revised O&M Manual has been provided with this submittal.**

***12.*** *The HydroCAD ADS Pipe Detention Basin calculations were based on six (6) rows of 240' - 24" diameter round pipe storage embedded within a 22.00'W x 203.00'L x 3.00'H stone bed. The calculations were based on the bottom of stone at elevation 156.50, with the 4" outlet orifice at elevation 157.00. Since the system does not provide exfiltration, the storage below the 4" outlet would be filled with water and not available for any future storm. In addition, most of the system is below the ESHGW.*

*The revised pipe detention system should be pressure tested to prove watertight. There is insufficient data to support the proposed roof recharge systems are not in the HSG 'D' soil. The NRCS Webb Soil Survey identified the western portion the site consists of 71B Ridgebury fine sandy loam, 3 to 8 percent slopes, extremely stony, HSG 'D' soil. And the eastern portion of the site consists of 310A Woodbridge fine sandy loam, 0 to 3 percent slopes, HSG "C/D" soil. The soil test pit logs indicated consistent soil material as identified in TPs-1 and -2. The laboratory tests reported high (39% - 39.5%) silt/clay contends and classified as HSG 'D' soils. (TP-3 was the only test hole, which found sand in the C-layer.) Based on the soil tests data available, there is not sufficient evidence that the proposed roof recharge/infiltration systems would function as designed. Furthermore, the roof recharge systems as shown do not meet the minimum 25' setback to each system.*

The revised Plan Set and Drainage model has removed the underground roof recharge systems. The proposed downspouts will be routed to the Landscape areas around the project and are modeled to enter the closed drainage system.

*The revised HDPE pipe detention system does not meet the watertight requirements. See TN5.07 technical Note comment #31 above.*

**In order to eliminate the potential of infiltration of the groundwater within the ADS pipe system, the proposed excavation area will be lined with a one piece 30 Mil PVC liner that will be seamed at the factory with electro-welding to ensure there will be no leakage. All connections or penetrations to and from the ADS pipe system will be installed with a waterproof boot supplied by SolMax Geosynthetics and EPI.**

***13.*** *The Grading and Drainage plan showed a 203' length bed only, which was 40-foot short of the 6 rows of 240' -24" diameter pipes storage used in the calculations. The plan also called for the bottom of stone at 156.00 with the 24" pipes invert at 157.00, where the ESHGW is at 158.39 (TP#6). The detention system must be set above the ESHGW. See comment #12 above.*

The revised Stormwater Report and Plans have been corrected and shows the underground pipe system as 300 feet in length. The underground pipe system has been lowered to elevation 157.0', placing the system 1.4 feet into the water table at its highest. Calculations have been provided by the manufacturer stating that a pipe system of this nature fully submerged in the water table would need a minimum cover of 22" to ensure that the system would not float. The proposed pipe detention system has 30" of cover at the lowest proposed grade.

*The revised HDPE pipe detention system does not meet the watertight requirements. See TN 5.07 technical Note comment #31 above. TN5.05's 22" minimum cover appeared to be valid. However, calculations should be certified and provided to show this site design meets the requirements for liability purpose.*

**In order to eliminate the potential of infiltration of the groundwater within the ADS pipe system, the proposed excavation area will be lined with a one piece 30 Mil PVC liner that will be seamed at the factory with electro-welding to ensure there will be no leakage. All connections or penetrations to and from the ADS pipe system will be installed with a waterproof boot supplied by SolMax Geosynthetics and EPI.**

***14.*** *HydroCAD report Pond 11P – Sediment Forebay, there should not be any valid stormwater storage volume available in the Forebay. The bottom of the forebay at 156.00 with the spillway weir at 156.50 are both below the ESHGW elevation at 157.34 (TP#2). The whole volume would be submerged under seasonal high groundwater during the wet months. The wet forebay does not provide any stormwater storage volume. The ponding water eventually would be replaced by sedimentation, which should be cleaned once a year under an Operation and Maintenance plan.*

The proposed Sediment Forebay and Extended Detention Wetland Basin have been modeled with 6" Low Flow Drain culverts. No storage credit was utilized in the Drainage Modeling below the elevation of the Low Flow Drain or the Estimated Seasonal High Water Table. Both areas below the culvert have been modeled as a water surface with a CN of 98.

*The constructed wet sediment forebay should be designed as a forebay pretreatment to receive the constructed wetland's 80% TSS removal credit. The proposed 6" culvert allows sediment discharged out of the forebay and should be removed. The forebay volume should be reserved for sediment storage only. No stormwater storage volume below the spillway weir (157.50) should be credited*

**The 6" culvert has been removed from the sediment forebay as requested. No storage volume has been utilized below the elevation of 157.50.**

***15.*** *HydroCAD report Pond 12P – Outlet Detention Pond's bottom at elevation 154.00 are also below the ESHGW at 154.47 (TP#3). The storage volume below the outlet weir at 155.50 would be filled with water with no exfiltration function. Hence, the storage volume is invalid. The proposed extended* *detention stormwater wetland does not provide any stormwater storage volume without a drawdown/outlet device.*

The proposed Sediment Forebay and Extended Detention Wetland Basin have been modeled with 6" Low Flow Drain culverts. No storage credit was utilized in the Drainage Modeling below the elevation of the Low Flow Drain or the Estimated Seasonal High Water Table. Both areas below the culvert have been modeled as a water surface with a CN of 98.

*Wetland basin is within the "25-feet No Disturb" zone, which requires Conservation Commission approval. The proposed 6" outlet culvert invert at 155.00, which is below the existing ground along the wetland boundary and requires grading within the resource area. This will require Conservation Commission approval. No stormwater storage volume should be allowed below the Test Pit #3's ESHGW at 154.5. The applicant needs to revise the outlet culvert invert and adjust the storage volume accordingly.*

**The area at the outlet pipe of the wetland basin has a surveyed elevation of 154.7 which is sufficient for the daylighting of this pipe. The HydroCAD model has been revised to ensure that no storage credit is taken below elevation 155.50 as requested in previous GCG Comment 7.**

***16.*** *There appeared to be two errors on the Post-Development (Proposed Conditions) summary of Flow to Rear Wetlands (DP#2), the 25-yr event peak flow rate should be 5.11 cfs with volume at 0.605 acre-ft. (see HydroCAD Prop-Conditions Revised 041024 page 47); and the 100-yr event peak flow rate should be 10.05 cfs with volume at 0.828 acre-ft. (see HydroCAD Prop-Conditions Revised 041024 page 62); Therefore, the calculations shown increased runoff volumes flow to the rear wetlands during the 25-yr and 100-yr storm events, net increases of 0.043 acre-ft and 0.091 acre-ft, respectively. Since the existing wetland is surrounded by Greenmont Avenue and Spring Park Avenue. The increased runoff volume would most likely create some adverse impacts to the downstream properties. GCG recommends recalculating the post-development runoff peak and volume with the above comments and re-compare the pre-and post-development conditions for the 4 study storm events.*

The proposed study area has been extended to the surrounding neighborhood that contributes to the Wetland area to the southwest of the property. Our office utilized Massachusetts GIS Topography to show the existing conditions topography within the neighborhood area contributing to the Existing Wetland Area identified. In addition, our office verified elevations in the rear of a number of dwellings within the neighborhood. Lastly, the 12" outlet pipe was survey located along with the Town of Dracut Drainage System in Spring Park Avenue that it connects to so as to provide the most accurate model of the offsite drainage.

The proposed Stormwater Report has been revised to include this information in the overall drainage model. The results of the model show that the proposed project reduces the peak flow but does increase the peak volume in all storm events. To ensure that this project will not create any adverse effects on the neighborhood area, the maximum elevation of the wetland area was analyzed and it was determined that this project will not raise the elevation of the wetland area in all storm events.

*The applicant has provided an off-site pre-development and post-development HydroCAD report to analyze the downstream flooding impacts imposed by this development. GCG has the following comments for the off-site analysis report:*

*a) GCG found a major discrepancy in the stormwater report which causes the analysis results invalid. The pre-development HydroCAD modeled approximately 7.9+/- acres (combined project site and off-site) of single-family developed area with ½ acre lots, with 25% impervious surface, with Hydrologic Soil Group (HSG) 'D', CN value = 85 in sub-catchment 1S. However, the post-development HydroCAD sub-catchment 4S (the remaining off-site watershed, 5.7+/- acres) was modeled with ½ acre lots, with 25% impervious surface, with HSG 'C', CN value = 80, which has a lower runoff rate than the pre-development conditions. Hence, the calculations are invalid. Furthermore, the project site's 2.24+/- acres (pre-development sub-catchment 1S minus post-development sub-catchment 4S) was included as ½ acre lots, with 25% impervious surface, with HSG 'D', CN=85. This is the site area with surveyed details and appeared to have an impervious percentage of 5.7+/-%. GCG recommends utilizing the exact calculated project site impervious area (CN valid) in the pre-development modeling.*

**The curve number of 4S-Remaining Ex Neighborhood WS Area in the Post Development analysis has been corrected to a CN value of 85 consistent with the Pre Development analysis. The analysis of the off site Neighborhood area was agreed upon at two meetings with GCG to utilize the Mass Mapper topography and an assumed value of the Curve Number utilizing ½ acre single family lots. This assumed curve number states that all lots have 25% of the property as impervious surface which is not the case for a number of the lots within the neighborhood.**

*b) Based on the EX-WS map contours as shown, there appeared to be a road crown along Greenmont Avenue and Spring Park Avenue. Additional stormwater runoff drains into the existing wetland area. The applicant should verify the off-site watershed boundary, there appeared to be additional drainage structures on Greenmont Avenue, which should be shown on the watershed plan. The existing drainage network and outfall may also clarify the watershed boundary.*

**Our office would agree that there is a crown in both Greenmont Avenue and Spring Park Avenue. The stormwater from these roadway surfaces flows along the gutter line and into the Town of Dracut's closed drainage system and therefore is not accounted for in the offsite analysis. The additional drainage structures on Greenmont Avenue collect the roadway stormwater and direct it to an outfall on Ontario Avenue and has no bearing on the neighborhood analysis.**

*c) The pre-development Pond 2P's 12" outlet culvert's pipe slope 2.78% should be verified. The model should utilize the 12" pipe (L=35'+/-, S=0.4+/-%) between the two existing catch basins on Spring Park Avenue, as the restricted point with the stormwater runoff surcharging out of the Ex-CB (rim=153.07) at the northern side of Spring Park Avenue.*

**The culvert elevation has been corrected as requested.**

*d) Based on the calculations as presented, the wetland's water ponding elevation would encroach the northern side of the existing house at #67 Spring Park Avenue during the 100-year storm event. GCG recommends using the same Time Span period with the pre- and post- development HydroCAD calculations and extending the time span (more than 24 hours) to cover the full volume of the rainfall events. Due to the sensitive flood situations, the Time increment/Hydrograph points (dt) should be reduced to 0.1 or 0.2 to increase the calculations points to show the detail peak water ponding elevations.*

**The timespans of both the Pre & Post Development have been made the same in the revised Stormwater Report.**

***17.** Extended Detention Wetland Basin Specifications as shown on plan sheet C-103 were based on the contributing watershed area of 58,470 square feet (s.f.). The watershed area for the entire site consists of 106,730 square feet. Based on the Prop-WS plan, only post sub-catchments 4S and 6S do not flow through the settlement forebay and constructed wetland. Hence, the wet forebay and constructed wetland sizing should be based on the (total site area minus sub-catchments 4S and 6S) 95,000+/- s.f. of watershed area. WQV volume calculations and percent distribution calculations to the extended wetland component according to Table CSW.1 should be provided.*

The proposed Extended Detention Wet Basin has been designed as a Constructed Stormwater Wetland per table CSW.1. The contributing area and calculations have been revised as necessary and revised in the Table provided on Sheet C-103.

*There are grading issues (proposed culvert invert below existing grade), storage volume issues (volume below ESGW), and maintenance access issues to be address as stated above.*

**All items outlined in the previous comments have been addressed and corrected.**

It is our opinion that this submission meets the performance standards as prescribed in the referenced statue and Zoning Code. Please schedule a Public Hearing for this submission at your earliest convenience. Thank you for your time and consideration on this matter. Should you have any questions or require additional information please do not hesitate to email or call our office.

Sincerely,

Kenneth M. Lania, E.I.T.
Senior Project Manager