11-0509-006-03
July 11, 2018

Jeremy Ginsberg, Director
Planning & Zoning
Darien Town Hall
2 Renshaw Road
Darien, CT 06820

Re: **Responses to Engineering Review Comments - Corbin Block**

Dear Mr. Ginsberg:

We have reviewed comments from Redniss & Mead dated June 27, 2018, and have responded to their comments below. A revised Engineering Report has been provided with these responses reflecting the changes made related to the review comments. The subsequent plan revisions will be provided at a later date once we have received all the Town comments and revised the plans accordingly.

The following summarizes our responses in **bold** text:

1. Corbin Block Engineering Drawings

a. Review soil conditions in the vicinity of Stormwater Retention System #2. Groundwater was observed Test B12 at elevation 43.00 and ledge was observed in Test RP10 at elevation 42.50. The crushed stone of the retention system is set at elevation 41.00.

   Response: While the groundwater reading at B12 is at elevation 43.0, the surrounding test boring and rock probe locations show significantly lower elevations to groundwater. For example, B1/B1A a few feet to the north has groundwater at elevation 37, RP 11 shows it at 38.8, and B9 and B4/4A both have it at 40.1. Additionally, RP 12, which is located near the middle of the system, has the top of bedrock at elevation 37.6 with no groundwater detected. Based on these readings we feel the groundwater elevation identified for B12 is likely a perched water table and an anomaly not corroborated by the other test locations in the area. Lastly, the crushed stone bed for the system was elevated to be roughly 1-ft above the highest of the prior noted groundwater readings (40.1) with the perforated pipes nearly 2-ft above same. This should provide adequate separation from the highest observed representative groundwater elevations for the infiltration system to function as designed.

   RP10 is immediately adjacent to the large rock outcrop that will be removed in this area as part of the project proposal. In addition, the existing rock profile for the bedrock drops off quickly as you go from south to north away from the I-95 non-access line. Therefore, the system should not be adversely impacted by the adjacent ledge and we will continue to evaluate this as the design advances and more information becomes available.

b. Review soil conditions in the vicinity of Stormwater Retention System #3. Ledge was observed in Test B7 and B9 at elevation 47.70 and 48.90, respectively. The crushed stone of the retention system is set at elevation 45.10.

Response: Due to restrictions from the current tenant, we did not have the ability to fully access this portion of the site for exploratory testing. Because of this the boring locations in this area were limited. As noted in the prior response, the existing ledge on site drops off significantly from south to north as you move away from the I-95 non-access line. In order to install the required utility trenching, storm drainage, and building foundations, the rock elevation in the area will need to be lowered as part of the project proposal providing greater separation from our system. It is our opinion that the other test borings and rock probes provided to the east will be representative for the rock elevations near System #3. We will confirm our assumptions during construction documents as a more detailed boring program is established.

c. Two hydraulic conductivity tests need to be performed within the footprint and at the bottom elevation of each system to confirm the soil can adequately infiltrate the designed stormwater volumes.

Response: The requested tests will be performed during the construction documents phase once each systems size and location are finalized. Tighe & Bond will verify that the observed rates are consistent with the design assumptions and make the necessary adjustments to the systems design to provide comparable results. Based on the soil conditions observed on site and the feedback provided by the geotechnical engineer, it is our opinion that an infiltration rate of 1.5 inches per hour is reasonable. Updated calculations are included with the revised Engineering Report that reflect the infiltration rate for the system being 1.5 in/hr.

d. Update the stormwater conveyance configuration to ensure that flows reaching MH 08 bypass Stormwater Retention System #2.

Response: This is the current design intent. MH-07 includes a diversion weir designed to direct site runoff into Stormwater Quality System #2. Once the system reaches capacity, flows will bypass the inlet to the system and top over the weir and be conveyed to MH-08. The top of the weir is set to coincide with the crown of the pipe diverting flow into Stormwater Quality System #2.

e. Are footing drains provided around the underground garage? If so, the infiltration capabilities of Retention System #2 may be minimal due to its proximity to the garage.

Response: The proposed garage foundation is intended to be a 'boat' with a pressure slab designed to resist hydrostatic pressure, no footing drains are being proposed.

f. Does the pipe run from OCS 01 to MH 02 pass over or through the below grade parking? If the pipe is intended to pass through the structure then it should be adequately sized to pass the 100-year storm. Provide profile.

Response: The pipe leaving OCS-01 and conveying flows to MH-02 is intended to go thru the garage in a carrier pipe. A profile of this pipe run has been included for your review. The design intent is for the garage to step down in this area to provide adequate clearance below the pipe for vehicular traffic within the garage. Upsizing the pipe will not provide any added stormwater conveyance within the system since the balance of the pipe network will still be sized for a 25-year storm, per Section 4.2 of the Darien Drainage Manual.

g. Provide crushed stone dimensions consistent with the Hydraflow Model on the details for each retention system.

Response: The calculations and details have been updated to be consistent.

h. Review inverts of the Vortechnic (WQS 02) unit and associated bypass configuration. The inverts into the Vortechnic and bypassing it are set at the same elevation.

Response: The inverts for the Vortechnic's unit have been revised to show a 4" drop across the structure.

i. Confirm that the secondary electrical service originating from the transformers along the southern property line is intended to run below Stormwater Retention System #1. Provide cross section.

Response: The secondary electric does not intersect with Stormwater Quality System #1. The secondary electric system goes around it. If the comment was intended for System #2, the secondary electric is intended to go over the top of the system. The design for this system currently maintains roughly 4-feet of cover from the top of 24-inch perforated pipes to finished grade.

j. Please resolve inconsistencies between the plan sheets, details, and Hydraflow Model. Some examples include:

   i. Stormwater Retention System #1

      1. C4.0/C4.2 – 6 Rows of 64 LF of 48" HDPE

      2. C8.6 – 6 Rows of 64 LF of 48" HDPE with some portions of the detail showing 24" HDPE

      3. Hydraflow – 10 Rows of 52 LF of 24" HDPE

   Response: The correct value used in the revised calculations and shown on the drawings and details is 10 Rows of 52 LF 24" HDPE.

   ii. Stormwater Retention System #2

      1. C4.0/8.7 – 4 Rows totaling 1,250 LF of 24" HDPE

      2. Hydraflow – 4 Rows totaling 1,280 LF of 24" HDPE

Response: The correct system size used in the revised calculations and shown on the drawings and details is 5 Rows totaling 1,477 LF of 24" HDPE.

iii. OCS O2

1. C4.0 – Inv. Out = 37.58
2. C8.7 – Inv. Out = 41.00
3. Hydraflow – Inv. Out = 41.00
4. C8.8 – The outlets indicated on the detail of OCS 02 don't match the plan view.

Response: The revised invert reflected through the plans and calculations is 41.95.

iv. OCS 03

1. C4.0 – Inv. Out = 44.80
2. C8.8 – Inv. Out = 45.58
3. Hydraflow – Inv. Out = 44.80

Response: The correct invert reflected through the plans and calculations is 44.80.

2. Tilly Pond Hydrologic and Hydraulic Calculations

a. Provide watershed maps that identifies all subbasins used in both the existing and proposed conditions analysis model.

   Response: The previously provided maps have been updated to include additional watershed delineations to assist in your review.

b. Evaluate the 50-year model using the coinciding peak water elevation of the Goodwives River in the vicinity of Design Point B as the tailwater applied to the outlet.

   Response: The Tilly Pond hydraulic analyses of the existing and proposed drainage systems required a starting tailwater elevation at the outlet at the Goodwives River. Available information from the September 9, 2014 LOMR of the Goodwives River was utilized in determining a tailwater elevation at the outlet. See Appendix C for the LOMR Profile and the Joint Probability Analysis table.

   Due to the outlet's location at the Goodwives River, a Joint Probability Analysis determined the coincidental occurrence frequency between the watersheds of Tilly Pond and the Goodwives River. The ratio of drainage areas is 10:1, requiring a 25-year frequency of the Goodwives River when performing 50-year design calculations of the Tilly Pond drainage

system. See Appendix for the river elevation and Joint Probability Analyses.

Joint Probability Analysis:

Goodwives River Drainage Area = 1140 Acres
Tilly Pond Drainage Area = 123 acres

Drainage Area Ratio
1140 :123 = 9.2: 1
Use 10:1 Area Ratio

From Table 8-3 Joint Probability Analysis (ConnDOT Drainage Manual)

	10-year	**25-year**	**50-year**	**100-year**
Tilley Pond	**10**	**25**	**50**	**100**
Goodwives River	**10**	**10**	**25**	**50**

c. The hydraulic grade line should not be limited to the rim elevation of the structures to facilitate review (uncheck bolted covers in the model to facilitate review).

Response: The hydraulic grade line within the model is not being limited by the rim elevations for the structures.

d. Confirm that the input dimensions for the two 24″ elliptical RCP (Link-12) are correct. The model is showing a 2′ rise and a 4′ span for each pipe. Standard dimensioning for two-foot-tall elliptical concrete pipes include a span of 38″. Dimensions to be confirmed in the field.

Response: Tighe & Bond field measured the sizes of these pipes to be twin 29″(H) x 45″ (W) RCP elliptical culverts. The model has been updated accordingly for the existing and proposed conditions.

e. Provide an analysis of the compensatory storage provided by the open swale during 100-year rainfall conditions when the Goodwives River reaches a peak water elevation between 41.24 and 41.80. Confirm this amount of storage is replicated in proposed conditions.

Response: During a 100-year storm the channel is partially filled with the runoff volumes contributing from the Tilley Pond watershed. The channel would be at roughly 69% capacity based on tailwater elevations of the Goodwives River. This capacity equates to a volume of roughly 6,210 CF of storage. The 2′x6′ box culvert would provide approximately 5,300 CF of storage, which results in a net reduction of 910 CF. The proposed Stormwater Quality Basin at 33 Old Kings Highway South provides an additional 4,380 CF of compensatory storage not provided

under existing conditions, which results in a net gain of approximately 3,470 CF of compensatory storage volume for the Goodwives River.

f. There is concern that the flowable area of the system is reduced moving downstream. Under proposed conditions, starting at the Post Road, the 42" AC CMP (Area = 9.62 is reduced to the 4'x2' Box Culvert (Area = 8.0 s.f.) which is then increased with the twin 24" Elliptical RCP (Area = 12.56 s.f.). Consider upsizing the box culvert so its cross-sectional area is equal to or greater than that of the upstream 42" AC CMP.

   Response: The system was designed to convey the 50-year storm as requested by the CTDOT during our preliminary discussions; however, we have upsized the proposed box culvert to reflect a 2-ft (H) x 6-ft (W) cross-section to provide additional conveyance. This modification will result in a cross-sectional area of 12-S.F., a net increase of 2.38-S.F. from the upstream 42-in CMP. We have updated the plans and calculations to document this revision and included with these responses.

g. Provide a profile of the drainage system running from the Post Road to the Goodwives River for both existing and proposed conditions. The profile should depict the water surface elevation and hydraulic grade line.

   Response: The requested profiles have been provided with these responses for your review.

h. In the event a storm larger than the design event occurs, provide an analysis of the overland flow path taken by runoff that bubbles out of on-site structures during extreme rainfall events. Confirm that no damage will occur to on-site buildings and off-site properties as stormwater makes its way to the Goodwives River.

   Response: A sketch of the surrounding area (OV-1) has been provided with these responses depicting the anticipated overland flow paths for the various inlet structures in the event the system has reached its capacity.

i. Provide documentation depicting the inverts used for the existing conveyance network starting at the Post Road and finishing at Design Point B. Show on existing survey.

   Response: The model was updated to reflect the surveyed inverts for the existing conveyance system from the Post Road to the outlet at the Goodwives River. The survey was missing the inlet for the twin elliptical pipes at the end of the channel and an interpolated spot grade was used for this invert. The balance of the model north of the Post Road is taken from the existing Dewberry model.

j. Provide input data and the user defined cross-section used for the shape of the Existing Drainage Swale in the model.

   Response: A typical channel cross-section for the model has been provided with these responses. See attached Figure XS-01 in Appendix C.

k. The existing conditions report is missing information for the 50-year storm. Ensure the complete report is provided for all storms.

Response: The additional missing sheets have been included with the revised calculations for your review.

3. Corbin Block Hydrologic and Hydraulic Calculations

a. Provide topography along the southern edge of the drainage areas abutting the Connecticut Turnpike and within the area of the two offsite drainage areas.

   Response: Additional topographic information has been added to the watershed maps as requested and they have been included for your review.

b. Review time of concentrations for existing and proposed drainage areas WS 01, WS 03A, and WS 04. The severity of the grades and flow patterns may result in flows transitioning from sheet to shallow concentrated flows in a shorter distance than assumed.

   Response: Tighe & Bond has reviewed the flow paths and made minor adjustments to the time of concentration calculations. These updated values and the subsequent spreadsheets have been included with the revised Engineering Report.

c. What is drainage area EX WS 11 and where does it go?

   Response: Drainage area EX WS 11 is no longer delineated on the existing watershed map and the remaining drainage areas have been revised to incorporate the balance of this area. The revised calculations reflecting the corrected areas is included with these responses for your review.

d. Review drainage area PR WS 02 and PR WS 03B. A roof leader originating from Building B and tying into the Post Road is not accounted for in the current drainage area configuration.

   Response: The roof leader in question has been routed to Water Quality System 1. This modification has been reflected in the revised Engineering Report included for review.

e. Confirm all data is consistent between the Existing Conditions Comparative Hydrology Map, CN & Tc Calculations, and Hydraflow Model. Some examples include:
   i. Existing CN & Tc Calculations/Existing Conditions Comparative Hydrology Map (WM-01)
      1. Total Area = 15.304 acres
      2. EX WS 05 = 0.555 acres
      3. EX WS 10 = 0.214 acres
      4. EX WS 11 = N/A
   ii. Existing Conditions Hydraflow Model
      1. Total Area = 17.393 acres
      2. EX WS 05 = 0.758 acres
      3. EX WS 10 = 1.124 acres
      4. EX WS 11 = 0.976 acres

Response: The calculations and watershed maps have been updated to reflect the same values and included with the revised Engineering Report. The correct values for the noted areas above are as follows:

1. **Total Area = 15.34 acres**
2. **EX WS 05 = 0.56 acres**
3. **EX WS 10 = 0.21 acres**
4. **EX WS 11 = deleted**

f. Confirm all infiltration system inverts, lengths, and crushed stone bedding dimensions are consistent between the Hydraflow Model and Site Plans.

Response: The Hydraflow model and site plans have been coordinated to reflect the same values. Revised calculations and drawings are included with these responses.

g. Use a consistent void ratio to model crushed stone. The report varies between 0.3 and 0.4.

Response: The calculations have been revised to provide a consistent value of 0.4 for the void space within the crushed stone.

h. Provide supporting documentation for the constant flowrate (Q) used in Diversion Hyd. No. 18 – TO WQS & 19 – BYPASS and coordinate between hydrologic model and conveyance calculations.

Response: The constant flow rate (Q) used for the proposed bypass was determined based on the capacity of the 24" diversion pipe to the system. The capacity of the 24" pipe at a slope of 0.34% is equal to approximately 15cfs.

i. An exfiltration rate of 3" per hour is too high for soils falling in Hydrologic Soil Group D. Use the rate measured during the Hydraulic Conductivity Testing with an applied factor of safety of 2 with consideration for a sustainable long term acceptable rate.

Response: The initial soil exploration and testing program performed on site was limited, and consisted primarily of determining general characteristics of the soils, along with identifying approximate groundwater, and bedrock elevations. Portions of the site were also not available for testing since they are being used by the current tenants and permission was not granted at the time testing was performed. Based on the soil conditions observed on site and the feedback provided by the geotechnical engineer, it is our opinion that an infiltration rate of 1.5 inches per hour is reasonable. As the design advances and additional testing can be performed, we will revisit the estimated rate and confirm it is consistent with the design. If the rate differs from our estimate we will make adjustments as needed to the systems design. Revised calculations are included with the supplemental calculations that reflect the revised infiltration rate for the system of 1.5 in/hr.

j. Include all flows reaching Design Point B in the proposed Hydraflow Model similar to what is shown in the existing conditions model.

Response: The Hydraflow model has been updated as requested to match the information provided with the existing model. The results are included with the revised Engineering Report.

k. Check all tributary areas. The total tributary area covered in the existing conditions model is 17.393 acres. The total tributary area represented in the proposed model and CN & Tc Calculations is 15.448 acres. Additionally, in Appendix D, the tributary area to Design Point B is showing a decrease of 0.913 acres from existing to proposed conditions. In the Tilly Pond Model, a decrease of 0.03 acres is shown.

Response: The tributary areas have been updated to be consistent throughout the calculations and model. The correct area is 15.34 Acres.

l. Drainage reaching Design Point C should be evaluated using the "fresh meadow" existing conditions analysis since associated runoff is not part of the Goodwives River Watershed.

Response: Stormwater Quality Basin #3 was designed to fully infiltrate the stormwater quality volume from the referenced area. Based on the limited size of the watershed in relation to the total storage provided by the proposed system, Basin #3 will reduce runoff flows to the Post Road consistent with those of the 'Fresh Meadow' condition. The existing results for the anticipated runoff flows from this watershed under the 'fresh meadow' condition are summarized below and have been provided for review in the revised Engineering Report.

DESIGN STORM	2-YEAR	10-YEAR	25-YEAR	50-YEAR	100-YEAR
FRESH MEADOW	1.066	1.943	2.494	2.924	3.354
PROPOSED	1.064	1.901	2.409	2.818	3.245

m. We believe the conveyance network starting at OCS 01 and running to Design Point A (via MH 02-05, MH 07 & 08, OCS 02, MH 15 & 16) is a critical trunk line and should at a minimum be sized to convey the 50-year storm. Special consideration will need to taken for the two 18" pipes crossing Old Kings Highway South since capacity is exceeded in the 25-year storm.

Response: Section 4.2 of the Town of Darien drainage manual specifies that storm drainage systems should be sized for a 25-year design storm. The proposed hydraulic design is consistent with the Darien Drainage Manual. Section 880 of the Darien Zoning Regulations requires that the hydrologic model demonstrate conformance up to the 50-year storm, which is demonstrated with the design calculations.

n. Include the Stormwater Quality Basin in the Hydraflow Model with all supporting calculations. There is no freeboard between the high concrete weir and riprap spillway and no analysis of depth of flow in overflow conditions.

Response: The contributing drainage area for the Stormwater Quality Basin is outside the proposed development site and is not designed to attenuate peak flows or mitigate volumes for runoff; therefore, a Hydraflow model of this basin and weir wall was not prepared. The sole intent of the system is to provide compensatory storage for the Water

Quality Volume (WQV) of the proposed site area north of Corbin Drive. The diversion weir will divert flows from the drainage system in Old Kings Highway South into this basin until the WQV volume is met and then allow runoff to discharge over the weir wall. A low-level orifice is provided at the base of the weir wall to maintain low level flows to the swale and keep the channel wet during less significant storm events. When the basin is full the runoff will no longer bypass into the basin and the channel will function hydraulically in much the same way as it does under existing conditions. The Discharge pipe from Old Kings Highway south is a 24-inch pipe with a cross sectional area of 3.14 S.F. The overflow weir is set at the same elevation as the invert for the 24-inch pipe and has a cross sectional area of roughly 13 S.F., which is more than 4 times greater than that of the 24-inch pipe discharging to it. In regards to the noted freeboard elevations, we have revised the elevations for the berm, the basin, the spillway, and the top of weir to provide 6-inches of freeboard. We have added the Stormwater quality Basin to the HEC model (labeled as 'Storm Water Quality Basin') to confirm it's addition will result in no adverse impacts to the peak flows in the Goodwives River. The plans and calculations have been updated accordingly in the revised Engineering Report and provided for review.

o. Provide sizing information for the Vortechnic Unit (WQS 02).

   Response: The sizing information for the Vortechnics model is included with the supplemental design calculations. All water quality structures will be reviewed and coordinated with the manufacturer prior to fabrication to confirm size and treatment capacities.

p. The CDS Model selected for WQS 03 is inadequately sized for the tributary water quality flow.

   Response: The WQF for WQS 03 is 1.588 cfs. The subsequent CDS unit for this flow is model CDS-3020-6-C.

q. Two of the three retention basins are sized to treat the tributary WQv. There sizes are as follows:
   i. Retention System #1 Storage Provided = 3,900 c.f. (115% of WQv)
   ii. Retention System #2 Storage Provided = 9,928 c.f. (88% of WQv)
   iii. Retention System #3 Storage Provided = 544 c.f. (229% of WQv)

Response: Stormwater Quality System #2 has been enlarged based on prior comments included with this memo. The associated modifications resulted in additional storage for this system, which is now in excess of the associated water quality volume for the contributing area. A summary of the current WQV's is provided below:

i. **Stormwater Quality System #1 - Storage Provided = 3,872 c.f**
ii. **Stormwater Quality System #2 - Storage Provided = 11,757 c.f.**
iii. **Stormwater Quality System #3 - Storage Provided = 698 c.f**

4. Goodwives River Hydrologic and Hydraulic Calculations

a. Provide input information used to build the model so a full evaluation of the existing and proposed results can be performed.

Response: The requested information has been provided with these responses. It was not previously provided due to the significant number of sheets for both the existing and proposed models.

b. Include the Stormwater Quality Basin in the model.

Response: The Stormwater Quality Basin has been added to the proposed HEC model for the Goodwives River and included with the revised Engineering Report for review.

c. Update model to reflect changes made per previous comments

Response: So noted.

5. 26 East Lane

a. Provide an NRCS Soil Map depicting which Hydrologic Soil Groups exist on-site.

Response: An NRCS soil map was includes in Appendix D of the provided Engineering Report.

b. Soil testing needs to be performed in the vicinity of each infiltration system.

Response: Percolation tests were performed at the site to the rear of the existing building in the vicinity of the proposed swale above the stormwater management system on 5/29/18. The field testing resulted in an observed infiltration rate of 3"/hr at one location and 6"/hr at the other. We based our design on the lower perc rate and utilized a conservative factor of safety of 3. The current design is based on a rate of 1"/hr, which is significantly lower than the observed rates. A copy of the perc test results has been included with the supplemental calculations included with these responses.

c. Hydraulic conductivity tests need to be performed in the footprint of each infiltration system.

Response: Tests were performed for the rear system (see response 5.b.), but the rate for the front system will be verified during construction documents. Should the observed rates differ from the estimated rates, the system design will be revised accordingly.

d. The exfiltration rate of 1" per hour is too high for soils categorized as "very poorly draining." Use the rate measured during the Hydraulic Conductivity Testing with an applied factor of safety of 2 with consideration for a sustainable long term acceptable rate.

Response: Field testing has verified our assumptions for the rear system. A perc rate of 3 in/hr was observed within the footprint of the proposed

system and a conservative factor of safety of 3 was incorporated into the design.

e. Confirm crushed stone encasement dimensions for Pond No. 1 match the detail on Sheet C5.

Response: The detail and design model have been updated to be consistent. The representative section for each pipe was input into the hydraflow model based on the revised details. Due to the limited input options within the hydraflow model there will be a slight difference in storage, but this should be negligible. The revised calculations and detail sheet have been provided for your review.

f. Provide supporting calculations for the drywell storage (Pond No. 2) used in the Hydraflow Model.

Response: The drywell sizing sheet has been updated to show the geometric equations used for sizing the drywell. The values calculated from this sheet were manually entered into the hydraflow model.

g. An error in the Hydraflow Report resulted in some of Pond No. 1 inputs being illegible. Please update.

Response: The hydraflow report has been reprinted and provided for your review.

If you have any questions, please feel free to contact us at 203-712-1100.

Very truly yours,

TIGHE & BOND, INC.

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Enclosures:
Copy: