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October 11, 2023

Town of Wareham
Zoning Board of Appeals
54 Marion Road
Wareham, MA 02571

Attention: Nazih Elkallassi – Chairman

RE: Response to Second Peer Review
Special Permit, Variance, and Site Plan Review Application
176 Main Street
Wareham, MA
ZBA Case 31-23
G.A.F. Job No. 21-9822

Dear Chairman Elkallassi,

G.A.F. Engineering, Inc., on behalf of our client Warren 176 Main St QOZB, LLC, provides the following responses to the review comments received from Allen & Major Associates, Inc. by letter dated October 6, 2023. Revised plans dated October 11, 2023, are included with the submittal.

This letter has been formatted for clarity by listing the review comment followed by our response in ***bold italics***.

Wareham By-Laws and Zoning By-Laws

1. The proposed site improvements are located within the Floodplain Overlay District, therefore subject to subject to Article 4: Overlay Districts, subsection 420 Floodplain Overlay District. A permit is required for development in the Floodplain Overlay District, per §420.10. The applicant should provide an update to the ZBA on the status of the permit. The plans should be updated to note and refer to the overlay district. If the proposed improvements costs are equal to or greater than 50 percent of the structure's market value, then the project would be considered a substantial improvement and the structure would need to be brought into compliance with current FEMA and building code regulations. Has the applicant considered the impacts that may be required to the building and what the potential impacts will be to the site to accommodate the improvements?

   Updated comment: The Cover sheet zoning data does

not appear to include the Flood Plain overlay district as noted in the response letter. Additional issues regarding the applicability of permit have been addressed by the applicant's response.

The plan set included with this submittal dated October 11, 2023, lists the Flood Plain overlay district in the Zoning Data table.

3. The proposed project is located within the Wareham Village 1 and 2 Zoning District and is subject to Article 7: Design Standards and Guideline, subsection 730 Wareham Village Districts. No architectural plans have been submitted; therefore A&M is unable to review for compliance with subsection 730 Architectural Design Guidelines. The applicant/architect should provide a statement for the record on how the proposed modifications will conform to the applicable section of the Zoning By-Laws. The ZBA may consider a condition of approval requiring the architectural modification of the building to be in compliance with subsection 730 of the Zoning By-Laws.

   Updated Comment: A&M defers the need for architectural information to the Board to aid in rendering a decision to the application.

   We have provided elevation views of the building sufficient for the Board's review. No further action required.

4. Zoning By-Law Section 1031 requires "new projects or expansions exceeding 5,000 square feet of non- residential development or more than three multi-family dwelling units, the landscape plan shall be prepared by a registered landscape architect whose seal shall appear on the plan." Landscaping is currently shown on the site plans but has not been prepared by a Landscape Architect. A landscape plan should be provided in accordance with the Zoning By-Law. Please provide a landscape table showing the requirements and what is being provided.

Updated comment: The applicant had requested a variance from the landscaping requirements. However with some new material added to sheet 6 of 8, it is unclear if the variance request remains. Please clarify. The addition of landscaping, wherever possible, is an improvement to the prior plan.

We have, however, enhanced the landscaping at the rear property line to provide screening for the neighboring abutter. The variance for landscaping was approved at the September 27, 2023, meeting.

Site Plan & Drainage Calculations

8. The applicant proposes restriping an area of existing pavement to remain along the Main Street portion of the site adjacent to the 6 parking stalls. There appears to be an opportunity to remove the excess asphalt and reduce the impervious area on this site improving the overall runoff condition. Would the applicant be willing to remove the excess asphalt in this area? If the applicant is amenable to this suggestion, the opportunity exists to regrade the site driveway to promote runoff into the drainage trench versus directly into Main Street also improving the general runoff condition.

   Updated comment: A&M defers to the Zoning Board regarding the intended use for snow management in lieu of additional landscaping as suggested. No further comment.

   No action required.

9. Test pit symbols should be added to the Drainage & Grading plan. It appears that the drainage system is located within the estimated seasonal high-water table, based on redoximorphic features (elevation 5.5) in Test Pit #2. The design engineer should review the elevations and revise the plans accordingly.

   Updated comment: The test pits have been added as suggested. The redox features noted at a depth of 70" (elevation 5.67) have not been reconciled and still

conflict with the elevation of the drainage field (elevation 3).

The redox feature is not noted as the Estimated Seasonal Highwater Table.

The redox features are associated with a textural break in the soil and are not the result of a fluctuating water table. There is a sump pit in the lower level of the building. The bottom elevation of this sump pit is EL 3.6. Note that there is no standing water in this sump pit. The bottom of which is at elevation 3.6.

Also note that the groundwater elevation at nearby 59 Main Street is 0.5 feet.

In our opinion the drainage system will function equal to or better than calculated given the conservative exfiltration rate specified by utilizing the Rawl's rate of 0.27 inches/hour.

10. After the estimated seasonal high groundwater elevation is confirmed under Comment 9, the applicant is required to provide a groundwater mounding assessment if less than four feet of separation is provided from the bottom of the stormwater system to the confirmed seasonal high water table.

Updated comment: Comment remains outstanding.

We have performed mounding calculations for the drainage system which indicate that the groundwater mound beneath the center of the system will be 0.93 feet. The system would therefore perform as calculated if seasonal high groundwater was present at any elevation below the crushed stone.

12. The design engineer should review the proposed watershed divide line between watersheds "1S" & "2S" on the northerly side of the building. The grading indicates the dividing line should be adjusted.

Updated comment: The applicant indicated a revision to the watershed divide and its de minimus nature. A&M is unable to confirm as no updated drainage calculations or watershed plans were provided as part of the revisions. Spot grades were added to the drainage plan but depict a level area at elevation 12.3 on the north side of the building.

The drainage calculations and watershed maps remain valid. Spot grades were added to the grading plan to clarify the finished grade on the north side of the building which will function as an overflow weir for the larger calculated storms.

14. It appears that the design engineer is utilizing pond storage in the HydroCAD model in addition to the proposed subsurface leaching pits. The plans do not depict the intended location of this storage on the plan. It appears the parking lot is being used as the ponding area in all storm events. Based on the peak elevations reported in the HydroCAD report, water will be ponding against the building during the 10-year event and greater storm events. This should be clarified or revised. The rear door elevation is listed at 12.5 while the 100 year ponding in this area is at 12.79.

Updated comment: The response letter notes the use of a broad crested weir at elevation 12.3 on the northerly side of the building. A&M concurs with this from the prior HydroCAD model. However, the model notes that with the weir discharging during the 100-year storm, the water elevation will rise to elevation 12.79 and conflict with the door elevation. Understanding that during a 100-year event there are large areas of ponding that do occur, but the model directly indicates the building entry will be unusable as well as the rear parking areas that includes the only accessible parking spaces onsite.

A&M acknowledges that the entirety of the property is within the floodplain and will be inundated during the 100-year event but drainage backups/parking lot flooding occurs on all storm events. Given the approach to

redevelopment, the intent should be to facilitate drainage to the maximum extent practicable which may entail 100% containment of lower events.

The applicant can elect to accept the conditions as proposed with the acknowledgment that a large portion of the site may be potentially unusable during rain events, including the minimum 2-year storm event.

We disagree with the above characterization of the condition of the parking lot during storm events. Peak inflow and discharge rates that are calculated for Type III storm events have a duration of less than an hour. The resulting peak storm elevations dissipate in under an hour. The design also includes an outlet pipe set at elevation 9.3 which will provide a primary means of discharge that is three feet below the surface overflow on the north side of the building and will assist in ensuring full use of the parking lot during all calculated storm events.

Also note that recently there have been several significant storm events over the last month. Even with no site improvements which will improve existing conditions there have been no circumstances of adverse impact.

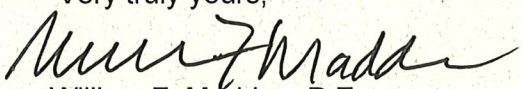
16. The proposed infiltration drawdown calculation reports that the system will take approximately 416 hours to completely drain after a 100 year storm event occurs. That would render the system unavailable to receive additional stormwater for nearly 17 days after significant precipitation. It is acknowledged that the design is based on redevelopment of a site where limited stormwater controls currently exist, however the potential for increased and repeated flooding of the property should be evaluated and the design revised accordingly. The design statement refers to the use of 0.27 inches per hour as the "conservative" rate of the soil as indicative of the drawdown time. Additional evaluative measures should be performed to further determine the underlying soils condition and consider the use of a double ring infiltrometer, or similar test, to determine the infiltrative ability of the soil.

Updated comment: Acknowledging that the site is located within the floodplain for the higher storm events, but the stormwater system ponds within the parking area in all storm events. As noted above, the applicant may accept this condition but should be aware of the potential for adverse ponding during rain events. The stormwater regulations allow for compliance to the maximum extent practicable for redevelopment but standards 2 and 3 are required to be met for compliance.

Stormwater will not pond within the parking area during all storm events. The drainage report and calculations as submitted confirm compliance with standards 2 and 3.

Please contact me directly should you have any questions about this project.

Very truly yours,



William F. Madden, P.E.
bill@gafenginc.com

WFM/jlc

Enclosures

cc: Warren 176 Main St QOZB LLC
Jilian Morton, Esq
Allen & Major Associates, Inc.

Groundwater Mounding Calculations

Groundwater Mounding Analysis based on a 2 inch volume 24 hour storm event.
Basin Exfiltration Rate entered into HydroCAD is 0.27 inches/hour for HSG C soils.

Pond 1P – Leaching Pits/Crushed Stone

Discarded Storm Volume = 0.018 acre-feet
Crushed Stone Bottom Area = 644 square feet
Duration of Exfiltration = 15.0 hours (Hydrograph)

Calculate the infiltrated volume of runoff to a rate in feet per day

0.018 af x 43,560 sf/acre = 784 cf

784 cf ÷ 644 sf = 1.22 feet

1.22 feet ÷ 15.0 hours x 24 hours/day = 1.95 feet/day

Using the Hantush calculator input R = 1.95 feet/day and K = 19.5 feet/day

Groundwater Mound = 0.93 feet at the center of the basin

This spreadsheet will calculate the height of a groundwater mound beneath a stormwater infiltration basin. More information can be found in the U.S. Geological Survey Scientific Investigations Report 2010-5102 "Simulation of groundwater mounding beneath hypothetical stormwater infiltration basins".

The user must specify infiltration rate (R), specific yield (Sy), horizontal hydraulic conductivity (Kh), basin dimensions (x, y), duration of infiltration period (t), and the initial thickness of the saturated zone (hi(0), height of the water table if the bottom of the aquifer is the datum). For a square basin the half width equals the half length (x = y). For a rectangular basin, if the user wants the water-table changes perpendicular to the long side, specify x as the short dimension and y as the long dimension. Conversely, if the user wants the values perpendicular to the short side, specify y as the short dimension, x as the long dimension. All distances are from the center of the basin. Users can change the distances from the center of the basin at which water-table aquifer thickness are calculated.
Cells highlighted in yellow are values that can be changed by the user. Cells highlighted in red are output values based on user-specified inputs. The user MUST click the blue "Re-Calculate Now" button each time ANY of the user-specified inputs are changed otherwise necessary iterations to converge on the correct solution will not be done and values shown will be incorrect. Use consistent units for all input values (for example, feet and days)

use consistent units (e.g. feet & days **or** inches & hours)

Input Values		
1.9500	*R*	Recharge (infiltration) rate (feet/day)
0.200	Sy	Specific yield, Sy (dimensionless, between 0 and 1)
19.50	K	Horizontal hydraulic conductivity, Kh (feet/day)*
7.000	x	1/2 length of basin (x direction, in feet)
23.000	y	1/2 width of basin (y direction, in feet)
1.000	t	duration of infiltration period (days)
20.000	hi(0)	initial thickness of saturated zone (feet)

20.930	h(max)	maximum thickness of saturated zone (beneath center of basin at end of infiltration period)
0.930	Δh(max)	maximum groundwater mounding (beneath center of basin at end of infiltration period)

Conversion Table

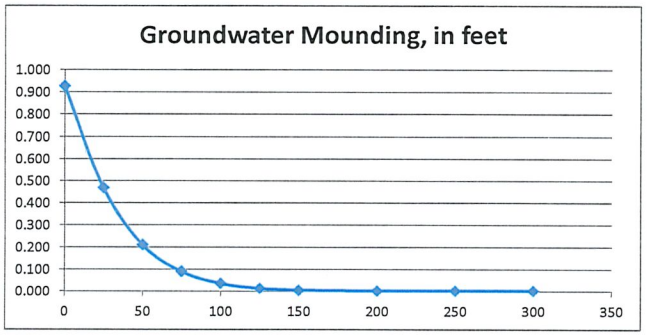
inch/hour	feet/day
0.67	1.33
2.00	4.00

hours	days
36	1.50

In the report accompanying this spreadsheet (USGS SIR 2010-5102), vertical soil permeability (ft/d) is assumed to be one-tenth horizontal hydraulic conductivity (ft/d).

Ground-water Mounding, in feet	Distance from center of basin in x direction, in feet
0.930	0
0.470	25
0.212	50
0.092	75
0.038	100
0.016	125
0.007	150
0.004	200
0.003	250
0.003	300

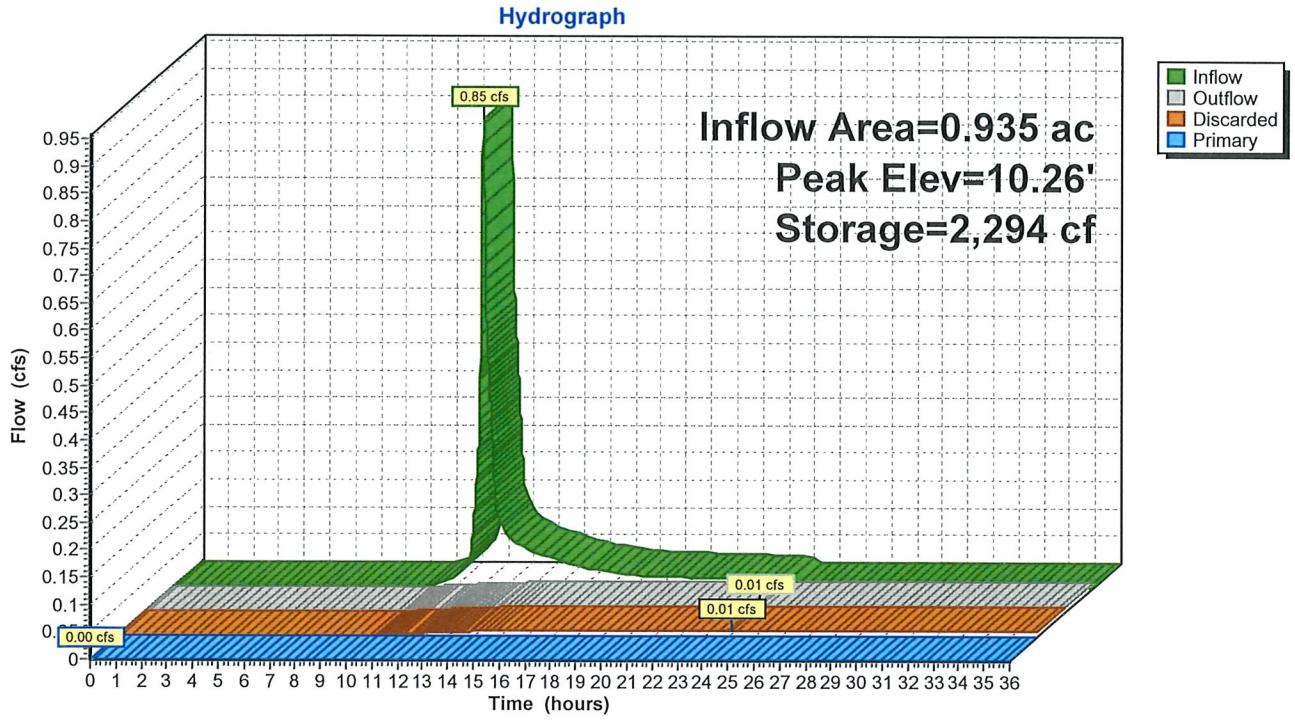
Re-Calculate Now



Disclaimer

This spreadsheet solving the Hantush (1967) equation for ground-water mounding beneath an infiltration basin is made available to the general public as a convenience for those wishing to replicate values documented in the USGS Scientific Investigations Report 2010-5102 "Groundwater mounding beneath hypothetical stormwater infiltration basins" or to calculate values based on user-specified site conditions. Any changes made to the spreadsheet (other than values identified as user-specified) after transmission from the USGS could have unintended, undesirable consequences. These consequences could include, but may not be limited to: erroneous output, numerical instabilities, and violations of underlying assumptions that are inherent in results presented in the accompanying USGS published report. The USGS assumes no responsibility for the consequences of any changes made to the spreadsheet. If changes are made to the spreadsheet, the user is responsible for documenting the changes and justifying the results and conclusions.

Pond 1P: Leaching Pits/Crushed Stone



Summary for Pond 1P: Leaching Pits/Crushed Stone

Inflow Area = 0.935 ac, 47.99% Impervious, Inflow Depth = 0.80" for 2 Inch Storm event
Inflow = 0.85 cfs @ 12.09 hrs, Volume= 0.062 af
Outflow = 0.01 cfs @ 24.06 hrs, Volume= 0.018 af, Atten= 99%, Lag= 717.8 min
Discarded = 0.01 cfs @ 24.06 hrs, Volume= 0.018 af
Primary = 0.00 cfs @ 0.00 hrs, Volume= 0.000 af
Routed to Link 1L : Main Street

Routing by Stor-Ind method, Time Span= 0.00-36.00 hrs, dt= 0.01 hrs / 2
Peak Elev= 10.26' @ 24.06 hrs Surf.Area= 644 sf Storage= 2,294 cf

Plug-Flow detention time= 715.7 min calculated for 0.018 af (29% of inflow)
Center-of-Mass det. time= 579.6 min (1,428.8 - 849.2)

Volume	Invert	Avail.Storage	Storage Description
#1	11.50'	3,393 cf	**Custom Stage Data (Conic)** Listed below (Recalc)
#2	3.00'	1,896 cf	**14.00'W x 46.00'L x 8.50'H Crushed Stone** 5,474 cf Overall - 735 cf Embedded = 4,739 cf x 40.0% Voids
#3	4.00'	735 cf	**6.00'D x 6.50'H Leaching Pits** x 4 Inside #2
		6,024 cf	Total Available Storage

Elevation (feet)	Surf.Area (sq-ft)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)	Wet.Area (sq-ft)
11.50	650	0	0	650
12.00	3,000	841	841	3,001
12.50	3,950	1,732	2,573	3,957
12.70	4,250	820	3,393	4,260

Device	Routing	Invert	Outlet Devices
#1	Primary	12.30'	**5.0' long x 20.0' breadth Broad-Crested Rectangular Weir** Head (feet) 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60 Coef. (English) 2.68 2.70 2.70 2.64 2.63 2.64 2.64 2.63
#2	Primary	10.90'	**6" Emitters X 3.00** 6.000" Diameter, C= 0.600 3.4' Long Tube, Hazen-Williams C= 130 Inlet / Outlet Elev. = 7.53' / 10.90'
#3	Device 2	9.30'	**6.0" Round Culvert** L= 177.0' Ke= 0.500 Inlet / Outlet Invert= 9.30' / 7.53' S= 0.0100 '/' Cc= 0.900 n= 0.012, Flow Area= 0.20 sf
#4	Discarded	3.00'	**0.270 in/hr Exfiltration over Wetted area**

Discarded OutFlow Max=0.01 cfs @ 24.06 hrs HW=10.26' (Free Discharge)
└**4=Exfiltration** (Exfiltration Controls 0.01 cfs)

Primary OutFlow Max=0.00 cfs @ 0.00 hrs HW=3.00' (Free Discharge)
├**1=Broad-Crested Rectangular Weir** (Controls 0.00 cfs)
└**2=6" Emitters** (Controls 0.00 cfs)
└**3=Culvert** (Controls 0.00 cfs)