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(508) 903-2000
(508) 903-2001 fax
www.rizzo.com

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May 4, 2006

RECEIVED
Via Email
MAY 4 2006
JS

Mr. Jay Szklut
Planning and Economic Development Manager
Office of Community Development
19 Moore Street
Belmont, MA 02478

Re: The Residences at Acorn Park
Belmont, Massachusetts

Dear Mr. Szklut:

Rizzo Associates, Inc. (Rizzo), on behalf of AP Cambridge Partners II, LLC is providing detailed responses to the comments raised in the April 27, 2006 letter prepared by FST with regard to their review of the Drainage Report prepared by Rizzo in support of The Residences at Acorn Park. Listed below are each of the comments raised in FST's letter followed by our detailed response.

Please note that in response to other comments received (particularly the Belmont Fire Department) the site plan layout, grading and drainage has been revised to accommodate a 20-foot fire access roadway around the sides of Buildings B and D. This revision has impacted the stormwater management system and the drainage calculations.

General Civil Review

Comment 1: "*Ingress and egress to the site is shown via (3) three roadway curb cuts located off of Acorn Park Drive. FST recommends the intersection and/or stopping site distance at the three entrance/exit locations onto Acorn Park Drive be evaluated, documented and included in the Comprehensive Permit submittal.*"

Response: Refer to Vanasse Associates, Inc. (VAI) traffic study dated January 2006 regarding site distance analysis. This was submitted to your office by VAI. Attached is an excerpt of the study related to sight distances.

Comment 2: "*FST questions the length of each proposed entrance/exit drive located off of Acorn Park Drive in providing the required vehicle storage to avoid the potential conflict with vehicles entering and exiting the site. We recommend additional documentation addressing this issue be submitted by the Applicant.*"

Response: We have forwarded (via VAI) a copy of the VAI traffic study dated January 2006 to your attention. Based on the Future Build study year (2010) 95th percentile queue, no more than one (1) vehicle will be queued at each driveway exit during the a.m. peak hour per minute. Refer to attached Table 14 of the VAI study.

Comment 3: *"As shown on the submitted site plan, parking for the site will be provided under each building and on the surface in small block areas surrounding the buildings. FST recommends a typical cross-section of the surface parking areas identifying items such as location of sidewalk, guard rail, grass strip, pavement width, median, curbing, driveway crown and side slopes be provided on the site plan."*

Response: We will provide cross section.

Comment 4: *"Due to the layout of the buildings and parking lots, we recommend turning movements for an SU-30 vehicle (e.g. fire truck/delivery vehicle) within the on-site parking lot areas be analyzed and submitted for review. Also provisions for emergency vehicle access along the rear of Building Nos. B and D need to be addressed by the Applicant."*

Response: We will submit fire truck turning maneuver plans to the ZBA, Belmont Fire Department, and FST for review.

Comment 5: *"FST recommends the limits and layout of the proposed sidewalk be reviewed by the applicant. We note that no sidewalks are currently shown within the front entrance parking lot areas associated Building Nos. A and E. Limits of the proposed sidewalk located adjacent to Acorn Park Drive needs to be further detailed on the site plan."*

Response: We have revised the plan to reflect appropriate sidewalk locations. Sidewalks along the west side of Acorn Park Drive will be pervious (i.e. stone dust) in order to be in keeping with the natural surroundings and similar to the material proposed for the trail system and kiosk. The proponent will connect to the existing walk at the frontage road intersection with Acorn Park Drive and construct the walk up to the project parcels terminus in Cambridge.

Comment 6: *"Provisions for trash removal and location of dumpsters with proper screening need to be provided on the site plan."*

Response: Trash will be collected in the underground garages and will not be shown on the site plans. Detailed architectural plans will be developed once permit approvals are obtained.

Comment 7: *"Applicant indicates that snow will be stored on pervious areas to promote infiltration. These areas are not shown on the plans and we recommend they be shown on the plans."*

Response: Snow storage areas will be shown on Sheet C-2.

Comment 8: *"We recommend a property line plan of the subject property, stamped by a Professional Land Surveyor (PLS) be provided to the Board."*

Response: We will provide a final property line survey sealed by a PLS.

Stormwater Management

Comment 1: *"For existing conditions, Figure 1 in the report indicates the sheet flow calculation for Subcatchment 2S is based on a 2 percent slope. The supporting HydroCAD calculations for sheet flow are based on a 1 percent slope."*

Response: Calculations have been revised. This modification will not impact pre-development (existing) runoff or have an impact on the stormwater management system design.

Comment 2: *"We note that for proposed conditions, the times of concentration for Subcatchments 1S, 2S, 4S, 5S, and 6S are less than five minutes. Typically, five minutes is used as the minimum value for a time of concentration. However, in this case, by using a value of less than five minutes, the peak flows that are generated are higher than those that would be generated for a time of concentration of five minutes. Therefore, more conservative results are being generated."*

Response: No action required.

Comment 3: *"On Figure 3, the grassed area in proposed area Subcatchment 1S is located in a D soil, not in a B soil, as used in the HydroCAD calculations."*

Response: Figure 3 has been revised (See attached). HydroCAD calculations will be revised accordingly and forwarded for review. This revision will have little impact on the final stormwater management system design.

Comment 4 *"We requested the Applicant provide a soil survey map to verify the delineation of the B and D soils. We note that the aerial map provided to us indicates that B soils terminate east of Acorn Park Drive. Figure 3 in the stormwater management report indicates B soils terminating west of Acorn Park Drive."*

Response: Previously provided to FST. Figure 3 in the Stormwater Management Report will be updated.

Comment 5: *"The stormwater runoff from the impervious area to the west of the proposed pool in Subcatchment 9S, sheets off towards the wetland without receiving treatment."*

Response: The grading and drainage plan (Sheet C-3) has been updated and shows a retention basin between buildings B and D. Runoff from the area described will sheet flow over the pervious area (geo-block) and into the basin.

Comment 6: *"In proposed Subcatchment 8S, the grading in the parking lot adjacent to the flood compensation area indicates that stormwater runoff from the parking lot will flow directly towards Acorn Park Drive and will not travel through the grass-filter strip. In addition, all the stormwater runoff from paved areas that is directed towards Acorn Park Drive does not appear to flow into the grassed swale adjacent to Acorn Park Drive, but instead appears to continue flowing down Acorn Park Drive."*

Response: The grading and drainage plan (Sheet C-3) has been updated and shows 1-foot contours and positive cross slope drainage to the grass filter strip.

Comment 7: *"The package submitted by the Applicant does not contain detailed calculations for the closed drainage systems. We recommend submittal of final design calculations to demonstrate system adequacy inclusive of, but not necessarily limited to, gutter flow capacity, width of gutter flow spread, inlet capacity / percent interception versus percent bypass for the selected inlet grate configurations, and pipe sizing calculations including the hydraulic grade line determination. The Town's Zoning By-Laws, Section 6B. Belmont Uplands District, require the storm drain system to be designed to convey the 10-year storm frequency. However, we recommend the Applicant demonstrate that flows in excess of the 10-year storm frequency will be conveyed to the stormwater management systems for treatment and attenuation."*

Response: The storm drain piping has been designed for the 25-year storm using StormCad. We will forward calculations with HydroCAD analysis once completed.

Comment 8a: *"Detention Basins 1 and 2 are located in D soils, which are not conducive to infiltration. Percolation tests have not been performed for this submittal and the Applicant states that these tests will be performed as part of final design. We stress that these tests must be done for both detention basins and the retention basin as part of the project's design efforts to verify that these basins, particularly the ones located in D soils, will be capable of infiltrating the runoff."*

Response: Percolation tests will be performed and HydroCAD Analysis updated to reflect results. In order to complete these tests, notification to the Belmont Conservation Commission is required since some test pits will be within the 100-foot buffer zone to wetland resource areas. We intend on filing a Request for Determination of Applicability to obtain approval. Once results are available the stormwater management systemt will be modeled with the test results and forwarded to the town and FST for final review.

Comment 8b: *"Based on the existing groundwater testing presented in the report, there is less than the recommended 2 feet of separation between the bottom of the detention basin and the high groundwater elevation for Detention Basins 1 and 2. The percolation tests done during final design will also determine the high groundwater elevation at the detention basins."*

Response: Once the test pitting program is completed, final design results will be submitted for review.

Comment 8c: *"For Detention Basins 1 and 2, the downstream inverts of the outlet pipes are 7.8 feet and 7.7 feet, respectively, and they discharge within the 100-year floodplain. The HydroCAD calculations were performed assuming free discharge and an empty detention basin. The 100-year flood is at elevation 9.8 feet; therefore, the outlet pipes will be submerged during the 100-year storm and water will backflow into the detention basins to an elevation of 9.8 feet. We recommend revising the model for the 100-year storm to reflect these conditions in order to ensure that these basins will be capable to attenuating the peak flow for the 100-year storm. Additionally, we recommend the Applicant estimate the flood elevations for the lesser storm events to determine whether the outlet pipes will be submerged. If so, the HydroCAD model should be revised to reflect these conditions to verify that these basins will attenuate the peak flows."*

Response: We will forward modified plans and supporting calculations once completed.

Comment 8d: *"No overflow spillway is provided for Detention Basin 2. We recommend the Applicant provide some method of conveying flows should the outlet control structure become clogged."*

Response: We agree to provide a proper detail.

Comment 8e: *"For Detention Basin 1 and the Retention Basin, details of the overflow spillway were not provided, except to indicate that the spillways are located at elevation 10.0 feet, which also appears to be the top of the basins. The spillways were also not included in the HydroCAD model. We recommend the Applicant demonstrate that these basins will be capable of directing flows over the spillways in a controlled manner in the event that the outlet control structure clogs (in the case of the detention basin) or infiltration proves to be less effective than anticipated."*

Response: We will forward modified plans and supporting calculations once completed. Preliminary analysis shows that the inclusion of the spillway in the calculations has no impact on the post-development runoff rates.

Comment 8f: *"The detention basins and retention basin do not provide a defined forebay area at the discharge points. In order to take the 70 percent credit in the TSS removal, defined sediment forebays must be shown on these basins."*

Response: Stormwater detention basin design is under modification due to site layout revisions. Forebays and/or water quality structure(s) will be included in the design.

Comment 9a: *"The plans indicate that an oil/grit separator will be used to treat the pavement runoff prior to discharge into an infiltration chamber. No details of the oil/grit separator were provided. On April 14, we requested details of the oil/grit separator from the Applicant for our review. As of the writing of this letter, we have not received this information."*

Response: We will forward modified plans once completed.

Comment 9b: *"We note that the infiltration chambers are located less than the recommended 20 feet away from the building foundations."*

Response: We will move chambers a minimum of 20 feet from proposed buildings.

Comment 9c: *"We note that Infiltration Chamber 2 is located in D soils, which is not conducive to infiltration and not recommended in the DEP standards because they will not function adequately in those soils. In addition, the boring log for OW-5, which is closest to this chamber, indicate the soils in that area are peat and organic silt situated over blue clay and inorganic silt. As previously mentioned, percolation tests have not been performed for this submittal and the Applicant states that these tests will be performed as part of final design. We stress that these tests must be done for the infiltration chambers to verify that these chambers, particularly the one located in D soils, will be capable of infiltrating the runoff."*

Response: We will be performing percolation tests and soil horizon analysis in the vicinity of the infiltration chambers to confirm system viability.

Comment 9d: *"Based on the existing groundwater testing presented in the report, there is less than the recommended 2 feet of separation between the bottom of the infiltration chamber and the high groundwater elevation for Infiltration Chamber 3. The percolation tests done as part of the project design process will also determine the high groundwater elevation at the infiltration chambers."*

Response: Once the test pitting program is completed, final design results will be submitted for review.

Comment 11: *"The measurement date of the groundwater elevations in the observation wells is not shown on the plans. At the request of FST, the Applicant provided the groundwater monitoring reports for the observation wells. These reports indicate that the readings were taken on April 2, 2001 after a heavy rain event. Therefore, the elevations shown on plans appear to represent high groundwater levels. We recommend the measurement date of the groundwater elevations be added to the plans."*

Response: Table to be added to C-3 that will present groundwater observation well #, date of measurement, and groundwater elevation.

Comment 12: *"The TSS calculation for Subcatchment 8S appears to count the grass filter strip twice in the calculations. Table 9 in the report only counts the grass filter strip once."*

Response: Individual calculation revised. Table 9 is ok.

Comment 13: *"There is an inconsistency between the text and the supporting calculations on the amount of runoff required to be infiltrated because the amount of impervious area over B soils is different. The text indicates the area is 3.866 acres, but the hand calculation uses an area of 2.79 acres. In addition, review of the soils map indicates that there may be more areas in B soils, than originally estimated. The Applicant should confirm the amount of B soils being impacted by this project."*

Response: All updated documentation and plans will be reviewed for consistency. This will not affect the design results.

Comment 14: *"The peak storage volumes and 100-year peak water surface elevations in Table 5 do not always match those values shown in the HydroCAD calculations. The Applicant should resolve these inconsistencies."*

Response: All updated documentation and plans will be reviewed for consistency. This will not affect the design results.

Comment 15: *"As a further point, when estimating the volume for recharge, the Applicant does not consider the impervious surfaces over the D soils because it is not conducive to infiltration. However, the Applicant assumes those same soils will be conducive to infiltration for Infiltration Chamber 3 when demonstrating the effectiveness of this chamber. We note these assumptions are contradictory."*

Response: Noted. Updated documentation and plans will be reviewed for consistency.

Comment 16: *"The Applicant acknowledges that the proposed project will result floodplain filling between elevations 5.0 and 9.8 feet. The Applicant provides floodplain compensation on a foot-by-foot basis and indicates that the compensatory*

Comment 9e: *"Given that the high groundwater elevations are close to the bottom of the infiltration chambers and that during the 100-year storm the groundwater elevation will rise, we recommend the Applicant prepare a mounding analysis for the infiltration chambers to ascertain their effectiveness under various storm events."*

Response: The Applicant will performing additional analyses as part of the Belmont Conservation Commission review of the Notice of Intent permit process. This will not commence until the Comprehensive Permit is approved.

Comment 9f: *"The HydroCAD model does not include the 8-inch overflow pipe within the infiltration chambers and these elevations are not provided within the documentation. We recommend the Applicant provide the elevation of these overflow pipes. We also recommend the Applicant consider the condition where the infiltration chambers fail to perform as proposed and water flows through the overflow pipes because it will impact the peak flows at the point of analysis."*

Response: The 8-inch overflows have been added to the plan. Refer to Sheet C-3.

Comment 9g: *"In the HydroCAD calculations, a flow rate of 0.02 cfs was used for all the infiltration chambers, as well as the detention and retention basins, to quantify the amount of exfiltration. The hand calculation provided in the documentation indicates a different flow rate was determined for each structure. On April 14 we asked the Applicant to clarify the reasoning for the infiltration calculations. As of the writing of this letter we have not received that clarification and we are therefore unable to fully evaluate the adequacy of the infiltration calculations. We also note, however, that the flow rate was derived from an infiltration rate 0.52 inches per hours, based on the assumption of a loam in a B soil group. With the exception of OW-5, this is a reasonable initial estimate for an infiltration rate because the boring logs for OW-1 through OW-4 indicate the soils are generally sandy. We also note that for HydroCAD, an infiltration rate can be directly entered into the model, rather than a flow rate."*

Response: We have modified the HydroCAD based on the hand calculated values and the system works effectively where used. When the actual percolation rates have been determined we will revise accordingly.

Comment 10: *"Based on the groundwater elevation shown in OW-4, the garage elevation appears to be beneath the groundwater elevation, which may result in seepage of water into the garage."*

Response: The architect has been notified and will take appropriate caution during their design.

storage volume will increase the available volume by 10 percent. However, we note that Town's Zoning By-Laws, Section 6B. Belmont Uplands District, require floodplain compensation at 1.5 times the volume impacted."

Response: As part of the Comprehensive Permit process the applicant has requested a waiver from the requirement of mitigating 1.5 times the floodplain impact. We meet all other local, state and federal floodplain mitigation requirements.

Comment 17: *"The erosion and sedimentation controls and schedule of inspections outlined for the construction period are reasonable. However, we note that the Applicant will need to obtain an NPDES General Permit for Stormwater Discharges from Construction Activities at the time of construction because this project will disturb greater than one acres of land. As a requirement of this permit, the Applicant will need to prepare a Stormwater Pollution Prevention Plan, which should incorporate the erosion and sedimentation controls and inspections discussed in Rizzo Associate's stormwater management report."*

Response: Noted. The applicant is aware of the need for a NPDES General Permit.

Comment 18: *"The Applicant proposes to inspect catchbasins, area drains and drop inlets on a quarterly basis and clean them on a semi-annual basis. We find this maintenance schedule to be reasonable, but we note that the DEP Stormwater Management Policy recommends monthly inspections and quarterly cleanings."*

Response: Proponent is in agreement.

Comment 19: *"The Applicant proposes to inspect and clean the sediment forebays at least once per year. The DEP Stormwater Management Policy recommends sediment forebays be inspected monthly and cleaned quarterly. We recommend the sediment forebays be inspected quarterly and cleaned semi-annually as a minimum, or more frequently, if necessary, based on the amount of accumulated sediments. We also note that the sediment forebays are not well-defined in the detention and retention basins."*

Response: Proponent is in agreement. The sediment forebays and/or water quality structures will be used to mitigate TSS.

Comment 20: *"The maintenance measures for the detention and retention basins are reasonable. However, the DEP Stormwater Management Policy recommends a minimum 10-foot-wide access way for maintenance that does not cross the emergency spillway. For the Retention Basin and Detention Basin 1, this recommendation is not met because of the proximity of the basins to the proposed buildings."*

Response: Refer to revised plans for 10-foot wide access to basins.

Wastewater Management

Comment: *"As of this draft report, we have requested, but yet not received, any conceptual information regarding the proposed pumping station, specifically, the peak hourly flow (PHF) and proposed pumping rate. This key information is essential for determining the impact on the Town's existing sewerage system. Consequently, FST's review of the wastewater component of the project cannot advance any further until this information is received."*

Response: We have forwarded to FST (Justin Gould) via email on May 2, 2006 the wastewater pump station calculations.

If you should have any questions or would like to discuss these responses, please feel free to contact me at 508-903-2350.

Sincerely,

David Albrecht

David M. Albrecht, P.E.
Senior Project Manager

Attachments

cc: S. Corridan – O'Neill Properties
J. Ward, Esquire – Nutter, McClennen & Fish, LLP
R. Engler – Stockard Engler & Brigham
J. Dirk-Vanasse & Associates, Inc.
File

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SIGHT DISTANCE MEASUREMENTS

Sight distance measurements were performed at the intersections of Acorn Park Drive with the site driveways in accordance with MassHighway and American Association of State Highway and Transportation Officials (AASHTO)[1] standards. Both stopping sight distance (SSD) and intersection sight distance (ISD) measurements were performed. In brief, SSD is the distance required by a vehicle traveling at the design speed of a roadway, on wet pavement, to stop prior to striking an object in its travel path. ISD or corner sight distance (CSD) is the sight distance required by a driver entering or crossing an intersecting roadway to perceive an on-coming vehicle and safely complete a turning or crossing maneuver with on-coming traffic. In accordance with AASHTO and MassHighway standards, at a minimum, sufficient SSD must be provided at an intersection. Table 17 presents the measured SSD and ISD at the site driveway intersections with Acorn Park Drive.

Table 17
SIGHT DISTANCE MEASUREMENTS

Intersection/Sight Distance Measurement	Required Minimum (Feet)[a]	Desirable (Feet)[b]	Measured (Feet)
Acorn Park Drive at the North Site Driveway			
Stopping Sight Distance:			
Acorn Park Drive approaching from the north	250	--	295
Acorn Park Drive approaching from the south	250	--	598
Intersection Sight Distance:			
Looking to the north from the north site driveway	250	335[c]/390[d]	295
Looking to the south from the north site driveway	250	335[c]/390[d]	607
Acorn Park Drive at the Center Site Driveway			
Stopping Sight Distance:			
Acorn Park Drive approaching from the north	250	--	440
Acorn Park Drive approaching from the south	250	--	449
Intersection Sight Distance:			
Looking to the north from the center site driveway	250	335[c]/390[d]	440
Looking to the south from the center site driveway	250	335[c]/390[d]	459
Acorn Park Drive at the South Site Drive			
Stopping Sight Distance:			
Acorn Park Drive approaching from the north	250	--	600
Acorn Park Drive approaching from the south	250	--	320
Intersection Sight Distance:			
Looking to the north from the south site driveway	250	335[c]/390[d]	600
Looking to the south from the south site driveway	250	335[c]/390[d]	328

[a]Recommended minimum values obtained from *A Policy on Geometric Design of Highways and Streets, Fifth Edition*; American Association of State Highway and Transportation Officials (AASHTO); 2004, and based on a 35 mph design speed for Acorn Park Drive.
[b]Values shown are desirable intersection sight distances for vehicles exiting a roadway under STOP control such that motorists approaching the intersection on the major street should not need to adjust their travel speed to less than 70 percent of their initial approach speed.
[c]Recommended minimum value for vehicles turning right exiting a roadway under STOP-sign control.
[d]Recommended minimum value for vehicles turning left exiting a roadway under STOP-sign control.

[1] *A Policy on Geometric Design of Highways and Streets*, Fifth Edition; American Association of State Highway and Transportation Officials (AASHTO); 2004.

As can be seen in Table 17, the measured sight lines both approaching the site driveway intersections along Acorn Park Drive and for motorists exiting the site were found to meet or exceed the minimum sight distance requirements for the appropriate design speed along Acorn Park Drive.

Table 14 (Continued)
UNSIGNALIZED INTERSECTION LEVEL-OF-SERVICE AND QUEUE SUMMARY

	2005 Existing				2010 No-Build				2010 Build			
Unsignalized Intersection/Peak Hour/Movement	Demand[a]	Delay[b]	LOS[c]	Queue[d] Avg.	Demand	Delay	LOS	Queue Avg.	Demand	Delay	LOS	Queue Avg.
Acorn Park Drive at Center Site Driveway:												
Weekday Morning:												
Acorn Park Drive NB LT/TH	--	--	--	--	--	--	--	--	109	0.0	A	0
Acorn Park Drive SB TH/RT	--	--	--	--	--	--	--	--	707	0.0	A	0
Center Site Driveway EB LT/RT	--	--	--	--	--	--	--	--	46	17.5	C	1
Weekday Evening:												
Acorn Park Drive NB LT/TH	--	--	--	--	--	--	--	--	491	0.0	A	0
Acorn Park Drive SB TH/RT	--	--	--	--	--	--	--	--	163	0.0	A	0
Center Site Driveway EB LT/RT	--	--	--	--	--	--	--	--	24	12.8	B	0
Acorn Park Drive at South Site Driveway:												
Weekday Morning:												
Acorn Park Drive NB LT/TH	--	--	--	--	--	--	--	--	86	0.0	A	0
Acorn Park Drive SB TH/RT	--	--	--	--	--	--	--	--	709	0.0	A	0
South Site Driveway EB LT/RT	--	--	--	--	--	--	--	--	32	16.6	C	1
Weekday Evening:												
Acorn Park Drive NB LT/TH	--	--	--	--	--	--	--	--	479	0.0	A	0
Acorn Park Drive SB TH/RT	--	--	--	--	--	--	--	--	124	0.0	A	0
South Site Driveway EB LT/RT	--	--	--	--	--	--	--	--	17	12.2	B	0

[a]Demand in vehicles per hour.
[b]Average control delay per vehicle (in seconds).
[c]Level-of-Service.
[d]Queue length in vehicles.
[e]Not calculated.
[f]Assumes a single approach lane. Field observation indicate that the approach functions as two lanes.
EB = eastbound; WB = westbound; NB = northbound; SB = southbound; LT = left-turning movements; TH = through movements; RT = right-turning movements.

Table 14
UNSIGNALIZED INTERSECTION LEVEL-OF-SERVICE AND QUEUE SUMMARY

Unsignalized Intersection/Peak Hour/Movement	2005 Existing				2010 No-Build				2010 Build			
	Demand[a]	Delay[b]	LOS[c]	Queue[d] Avg.	Demand	Delay	LOS	Queue Avg.	Demand	Delay	LOS	Queue Avg.
10. Acorn Park Drive at Alewife Station Off-Ramp												
Weekday Morning:												
Alewife Station Off-Ramp TH	2,044	0.0	A	0	2,235	0.0	A	0	2,235	0.0	A	0
Alewife Station Off-Ramp RT	12	0.0	A	0	39	0.0	A	0	39	0.0	A	0
Acorn Park Drive NB RT	68	>50.0	F	10	86	>50.0	F	NC[e]	119	>50.0	F	NC
Weekday Evening:												
Alewife Station Off-Ramp TH	1,172	0.0	A	0	1,277	0.0	A	0	1,277	0.0	A	0
Alewife Station Off-Ramp RT	4	0.0	A	0	34	0.0	A	0	34	0.0	A	0
Acorn Park Drive NB RT	22	23.4	C	1	101	>50.0	F	5	118	>50.0	F	6
13. Lake Street at Cross Street:												
Weekday Morning:												
Lake Street EB TH/RT	305	0.0	A	0	541	0.0	A	0	548	0.0	A	0
Lake Street WB LT/TH	488	9.7	A	3	539	13.2	B	4	584	14.5	B	5
Cross Street NB LT/RT[f]	315	>50.0	F	23	358	>50.0	F	NC	367	>50.0	F	NC
Weekday Evening:												
Lake Street EB TH/RT	117	0.0	A	0	189	0.0	A	0	215	0.0	A	0
Lake Street WB LT/TH	250	7.0	A	1	399	5.8	A	1	422	6.1	A	1
Cross Street NB LT/RT[f]	585	47.7	E	14	624	>50.0	F	30	661	>50.0	F	38
14. Lake Street at Concord Turnpike EB On-Ramp:												
Weekday Morning:												
Lake Street EB LT/TH	477	0.0	A	0	749	0.0	A	0	765	0.0	A	0
Lake Street WB TH	488	0.0	A	0	539	0.0	A	0	584	0.0	A	0
Lake Street WB RT	22	0.0	A	0	23	0.0	A	0	23	0.0	A	0
Weekday Evening:												
Lake Street EB LT/TH	508	0.0	A	0	610	0.0	A	0	673	0.0	A	0
Lake Street WB TH	250	0.0	A	0	399	0.0	A	0	422	0.0	A	0
Lake Street WB RT	18	0.0	A	0	19	0.0	A	0	19	0.0	A	0
Acorn Park Drive at North Site Driveway:												
Weekday Morning:												
Acorn Park Drive NB LT/TH	--	--	--	--	--	--	--	--	142	0.0	A	0
Acorn Park Drive SB TH/RT	--	--	--	--	--	--	--	--	705	0.0	A	0
North Site Driveway EB LT/RT	--	--	--	--	--	--	--	--	38	17.6	C	1
Weekday Evening:												
Acorn Park Drive NB LT/TH	--	--	--	--	--	--	--	--	508	0.0	A	0
Acorn Park Drive SB TH/RT	--	--	--	--	--	--	--	--	194	0.0	A	0
North Site Driveway EB LT/RT	--	--	--	--	--	--	--	--	19	13.4	B	0

See notes at end of table.

1-OBJECTIVE

Sewage (Wastewater) pump station design

2-METHOD OF ANALYSIS

- Calculation for force main is ruled by Hazen-Williams formula for flow in pipes under pressure
- The wet well should be large enough to prevent pumps motor from over heating due to extensive cycling, but small enough to accommodate cycling times that will reduce septicity and odor problems. Typically submersible pumps can cycle 4-10 times/hour
- The wet well capacity should contain sufficient wastewater to permit the pump to run for at least 2 min and restart no more than once in 5 min (Steel and McGhee 1979)
- The smallest capacity pump should be able to pump from the wet well and discharge at a self-cleaning velocity of 2 ft/sec (minimum velocity)

TETRA TECH, INC.
One Grant Street
Framingham, MA 01701-9005
(508) 903-2000

JOB 7128
SHEET NO. 1 OF 8
CALCULATED BY Damir Uzelac DATE 4/12/06
CHECKED BY TMH DATE 4/14/06
SCALE

3 – INPUT DATA

$Q_{AVR} = 25{,}500$ gpd

$n = 5$ – peak factor

$Q_{MAX} = Q_{AVR} \times n = 25{,}500 \times 5 = 127{,}500$ gpd

$L = 1{,}600$ feet – force main length

$dH = 10$ – feet – elevation difference

Inflow pipe invert depth ≈ 5 ft

4 – OUTPUT DATA

t_r – pump running time

t_f – pump filling time, pump off

V – storage volume of wet well

Q – Inflow

D – pump discharge

$$t_f = \frac{V}{Q} \qquad t_r = \frac{V}{D-Q}$$

$t = t_r + t_f$ – total cycle time

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SCALE			

Step 1 - Pump capacity for peak flow

$D = 127{,}500 \text{ gpm} = 88.54 \text{ gpm}$

Step 2 - Minimum volume for 2 min running time

$V_1 = 88.54 \text{ gpm} \times 2 \text{ min} = 177.08 \text{ gal}$

Step 3 - Volume V_2 for 5-min cycle

$t = t_L + t_f = \dfrac{V_2}{D - Q_{AVR}} + \dfrac{V_2}{Q_{AVR}}$ $\qquad Q_{AVR} = 25{,}500 \text{ gpd} = 17.71 \text{ gpm}$

$5 \text{ min} = \dfrac{V_2}{88.54 - 17.71} + \dfrac{V_2}{17.71} = \dfrac{V_2}{70.83} + \dfrac{V_2}{17.71} =$

$17.71 V_2 + 70.83 V_2 = 5 \times 70.83 \times 17.71$

$88.54 V_2 = 6271.9965$

$V_2 = 70.83 \text{ gal}$

Step 4 - Control factor determination

Since $V_1 > V_2$, therefore the pump running time is the control factor.

$V = V_1 = 177.08 \text{ gal} \approx 200 \text{ gal} \Rightarrow \boxed{V = 200 \text{ gal} = 26.7 \text{ cf}}$

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Step 5 - Actual time of pumping cycle

$$t = \frac{200 \text{ gal}}{(88.54 - 17.71) \text{ gal/min}} + \frac{200 \text{ gal}}{17.71 \text{ gal/min}}$$

$$t = \frac{200}{70.83} + \frac{200}{17.71} = 2.82 + 11.29 = 14.11 \text{ min}$$

$\boxed{t = 14.11 \text{ min}}$ - pumping cycle

Step 6 - Determination of wet well size

A submergance of 1 foot above the top of the suction pipe is required for an intake velocity of 2 ft/s. The depth between the well bottom and the top of submergance is 1.6 feet. If 4 ft diameter of wet well is chosen, surface area is 12.56 ft²

$$A = \frac{D^2}{4}\pi = \frac{4^2 ft^2}{4}\pi = 12.56 \text{ ft}^2$$ - surface area of the wet well

$$V = 200 \text{ gal} = 26.7 \text{ ft}^3$$

$$V = A \cdot h \Rightarrow h = \frac{V}{A} = \frac{26.7 ft^3}{12.56 ft^2} = 2.12 \text{ ft}$$

$$h = 2.12 \text{ ft}$$

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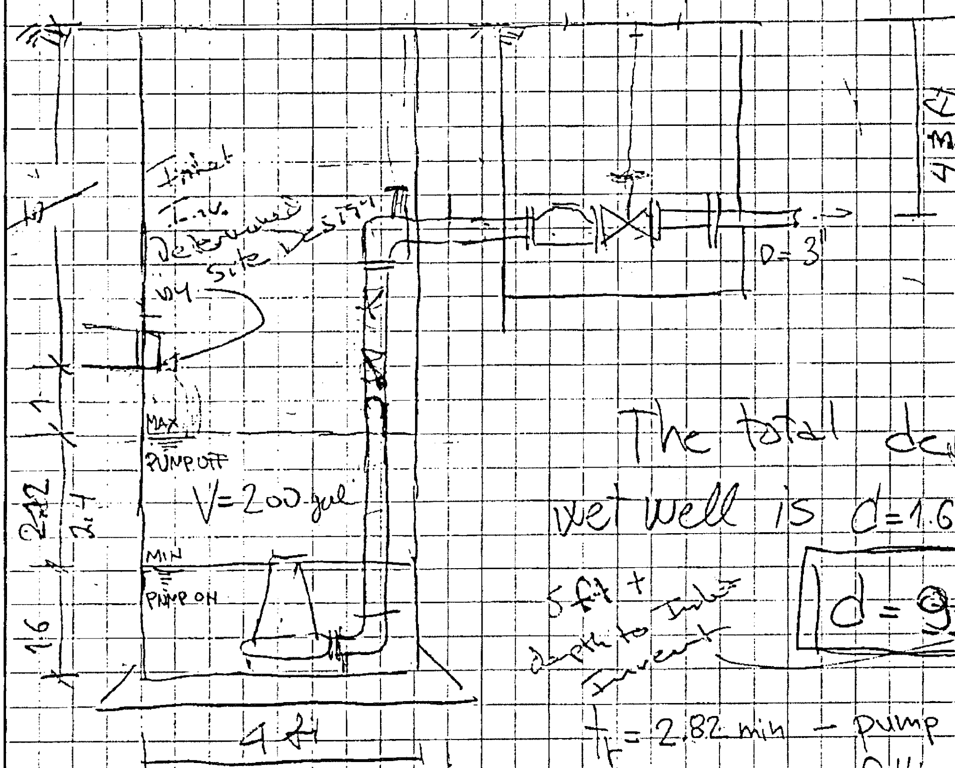
Typically 2 ft of freeboard is required. Thus minimum total depth of the wet well is

$d = 1.6\,ft + 2.12\,ft + 2\,ft = 5.72\,ft$

$d = 5.72\,ft$ minimum total depth of the wet well



The total depth of wet well is $d = 1.6 + 2.12 + 1 + 5$

$\boxed{d = 9.72\,ft}$ — OK

5 ft + depth to inlet invert

$t_r = 2.82$ min – pump running time

$t_f = 11.29$ min – filling time

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FORCE MAIN DESIGN

$L = 1600$ ft

$D = 3$ in $= 0.25$ ft

$dH = 10$ ft - elevation difference

- Static head calculated from minimum wastewater level in wetwell to discharge point (SH)

$SH = \Delta H +$ min level - below ground discharge

$= 10 + 8.12 - 3 = 15.12$ ft

$SH = 15.12$ ft - static head

- Friction head will be calculated with Hazen-Williams (HW)

$$H_L = L \times 10.439 \frac{Q^{1.85}}{C^{1.85} D^{4.8655}}$$

$$H_L^{max} = 1600 \times 10.4397 \frac{88.54^{1.85}}{130^{1.85} \cdot 3^{4.8655}} = 16703.52 \frac{4001.36}{8143.2 \times 209.62}$$

$$\boxed{H_L^{max} = 39.15 \text{ ft} - \text{friction head}}$$

$$H_L^{Avr} = 1600 \times 10.4397 \frac{17.71^{1.85}}{130^{1.85} \times 3^{4.8655}} = 16703.52 \frac{203.80}{8143.2 \times 209.62}$$

$H_L^{Avr} = 2$ ft , $v = \frac{Q_{MAX}}{A} = \frac{88.54 \text{ gal/min}}{\frac{3^2}{4}\pi} = \frac{0.197 \text{ cfs}}{0.049 \text{ ft}^2} = 4.02$ ft/s

$\boxed{v = 4 \text{ ft/s}}$

velocity

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- Minor Head loss (MH_L)

Exit	$K = 1.0$			1.0
3 × 90° Elbow	$K = 0.25$	× 3	=	0.75
check valve	$K = 2.5$	× 1	=	2.50
plug valve	$K = 0.5$	× 1	=	0.50
3 × 45° Elbow	$K = 0.2$	× 3	=	0.60

$\Sigma K = 4.35$ [5.85]

$$MH_L = K \frac{V^2}{2g} =$$

$$MH_L = 4.35 \text{ [5.85]} \times \frac{4.00^2 \text{ ft/s}}{2 \cdot 32.17 \text{ ft/s}^2} = \frac{16}{64.34} \times 4.35$$

$MH_L = 1.08$ ft [1.45]

- Total dynamic head (TDH)

$TDH = SH + H_L + MH_L = 15.12 + 39.15 + 1.08 = 55.35$ ft [1.45]

$\boxed{TDH = 55.35 \text{ ft}}$ – total dynamic head

PUMP SELECT

88.54 gpm @ 55 ft TDH

~~use 90 gpm @ 55 ft TDH~~

Two pump will be provided, but for redundancy only. Manufactur: GOULDS PUMPS. Submersible sewage pump, model 3888D3, series 3" SD. Enclosed data following. 3" discharge.

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CHECKED BY [signature] DATE 4/14/06
SCALE

5) CONCLUSION

Usful volume storage of the wet well is 200 gal, based on average inflow of 25,500 gpd. Total depth of wet well is 9.72 ft with 4-ft diameter base. Pump running time is 2.82 min and filling time is 11.29 min. Total pumping cycle (from turn-on to turn-on again) is 14.11 min. Check valve and plug valve will be placed in valve chamber that is 4-5 ft deep. 3"-force main (velocity is 4 ft/s) shouldn't be burried less then 4.3 feet below ground.

Two pump will be provided, but for redundancy only. Manufactur: GOULDS PUMPS. Submersible sewage pump, model 3888 D3, series 3SD. Pump data from the manufactury catalog is enclosed.

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