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CAPACITY ASSESSMENT REPORT

Stormwater, Sanitary Sewer, Water and Irrigation Systems

University at Albany (Uptown Campus)

Prepared for:

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COMMITMENT & INTEGRITY DRIVE RESULTS

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Executive Summary

In partial fulfillment of SUCF Program Study 01834, Woodard & Curran has prepared this Capacity Assessment Report. This report includes a capacity assessment of the Water System, Sanitary Sewer System, Storm Sewer System, and Irrigation System at the University of Albany based on capacity modeling efforts. The results of these capacity assessments are summarized below. This report represents items developed based on our observations as part of this project. The actual condition and capacity of the infrastructure items may have changed since the time of our investigations.

Water System

The capacity of the water system was assessed using the GIS-based water network modeling software, Infowater®. The fire flow availability at fire hydrants across the campus during peak system demands was used to assess the capacity of the water system. The following three peak demand scenarios were modeled:

- Peak hour demands during current demand conditions;
- Peak hour demands for an expanded demand scenario that includes anticipated building projects over the next five years; and
- Peak hour demands for the expanded demand scenario, with the addition of a second water supply source.

The results from the model simulations are that the available hydrant flow at hydrants varies greatly across the campus. In the current demand scenario, the available fire flows ranged from 835 GPM to 11,139 GPM. For the expanded demand scenario, the available flow generally decreased and ranged from 739 GPM to 9,733 GPM. If the Washington Avenue interconnection was supplemented with a booster pump, as simulated in the third scenario, the available fire flow generally increased, and ranged from 914 GPM to 12,966 GPM. The highest available fire flows were located at hydrants in close proximity to the 12-inch water main loop that runs around the academic podium. The hydrant with the lowest available fire flow was the hydrant near the Chemistry Building, a hydrant supplied off a 4-inch diameter service lateral. The next lowest available flows were located in Freedom Quad, the area furthest from the existing water supply source, and in an area served by a long length of 8-inch water main. The modeled results also determined that the Alumni House and Freedom Quad will most likely experience the lowest pressures in the system during fire flows and may need either larger diameter pipes or booster pumps to increase the supply pressure in these locations.

Sanitary Sewer System

The capacity of the sanitary sewer system was assessed by constructing a capacity model using Manning's Equation, conducting flow metering, and incorporating pipe condition information. The capacity design flow for each individual pipe was assessed and cross-checked with the information from the Condition Assessment Report.

It was concluded that there is no additional flow capacity in the southern interceptor because of pipe blockages of up to 90%. Pipe cleaning and root removal is necessary to increase capacity of this section of the sewer system. The northern interceptor has additional flow capacity available; the flow metering data indicated that the maximum instantaneous flow through this section of the sanitary sewer system was significantly less than the modeled flow capacity. However, the capacity model assumes that the pipe is in good condition. The northern interceptor pipes were generally in poor condition and should therefore be cleaned and replaced as recommended in the Condition Assessment Report to restore the actual capacity to the modeled flow capacity.

Stormwater Collection System

The capacity of the storm sewer system was assessed by developing a flow model using the SewerGEMs® modeling software. This modeling software estimates the stormwater generated during a rainfall event and the quantity and hydraulic grade line of flow through the system.

Model results indicate that there were four areas with limited capacity. Flooding could occur in State Gold Lot and Collins Circle area, the Colonial Quad area, and the Dutch Gold Lot area. The area near University Drive West closest to the entrance of Western Avenue could experience overflow from structures, consequently impacting traffic. It is recommended that the above areas be investigated further to evaluate the necessity and measures required to improve the capacity of the subsystems in these areas.

Irrigation System

The capacity of the campus irrigation system was assessed by calculating the irrigation demand, which is a function of the type of plant material being irrigated, the rainfall conditions, the evapotranspiration potential, the irrigation water supply capacity, and the efficiency of the irrigation system. This irrigation demand was compared to the capacity of the stormwater pond used as the source for irrigation water supply.

It is estimated that an area of approximately 100 acres of mixed turf and plant material, or 75 acres of turf only, could be irrigated with the current irrigation system. Currently, approximately 45 acres are outfitted for irrigation. If the current pond dredging activities take place at the proposed magnitude, approximately 9.1 million gallons of storage is available. During drought conditions in the month of July, if the maximum potential area of turf is irrigated (75 acres), the stormwater pond would be able to provide fewer than 19 days of irrigation capacity. During similar conditions in June and August, the pond could provide fewer than 20 and 22 days of irrigation water supply, respectively. The actual number of days of irrigation supply available will be dependent on the usable water from the retention pond including factors such as intake elevation and turbidity. Therefore, there is adequate capacity in this system.

The following report goes into detail on the capacity methodology and results for each of the four infrastructure systems included in this study.

1. WATER SYSTEM

1.1 METHOD

The GIS-based water network modeling software, Infowater®, was used to assess the capacity of the water system. The water model was developed using system maps provided by University at Albany, field investigations, fire flow testing, and system pressure data obtained with hydrant-mounted data logging pressure transducers. The water model simulates conditions in the actual water distribution system by iteratively calculating the hydraulic conditions in the pipes, junctions, and water storage features that are represented in the model.

The capacity of the water system was determined by estimating the available fire flow at a series of hydrants across the University at Albany campus. Fire flows are typically the highest flow condition that occurs in a water distribution system. The model was used to simulate the current water system operating conditions and the impacts of the proposed campus expansion projects on the availability of water supply.

The development of the model can be described as three steps:

1. Representation of the actual physical structures in the water system in the capacity model (Section 1.1.1);

2. Representation of the water supply demands in the model (Section 1.1.2); and

3. Calibration of the model to water system data (Section 1.1.3).

Once the model was developed as described above, capacity assessment scenarios were developed, as described in Section 1.1.4, to determine if the current system has adequate capacity to support current and future projects.

1.1.1 Modeling of Physical Structures

The piping network, including pipes, valves, junctions (intersections of pipes), water storage tank, and supply sources were developed using system maps provided to Woodard & Curran by University at Albany. These system maps included information such as location, material, size, and, in some cases elevation, of the features. The information included for each feature is described below.

1.1.1.1 Pipes

Each pipe in the model has assigned information that includes its location, length, diameter, estimated roughness, and connecting junctions. The location, length, diameter, and information on connecting junctions of the piping were derived from the system maps. The roughness of the pipe was estimated through the model calibration, described in Section 1.1.3.

1.1.1.2 Junctions

Junctions in the model represent valves, water hydrants, plugs, connections between pipes, or nodes where the water demands of the system are assigned. Each of the junctions contains location and elevation information derived from system maps and the aerial mapping conducted as part of the infrastructure assessment process. Demand nodes, the nodes used for water system demand assignment, are also assigned a water demand rate and diurnal curve, as described in Section 1.1.2. These demand nodes represent points in the water system where water is removed from the pipes. They are located adjacent to buildings and other water consuming structures, such as water fountains and hydrants.

1.1.1.3 Water Storage Tank

The University at Albany water distribution system has one water storage tank. The modeled tank is assigned the following information: location, base elevation, diameter, minimum level, and maximum level. The location and elevation information were derived from system maps and the aerial mapping conducted as part of the infrastructure assessment process. The diameter and maximum level were determined are from the November 13, 1964, Academic Group Part-2 plans by Edward Durell Stone Architects. The minimum level of the tank was set at zero (empty).

1.1.1.4 Supply Sources

The University at Albany campus has two supply connections – one on the eastern side of campus with the OGS campus, and the other along Washington Ave. from the City of Albany. In the modeled water system, these connections are termed “reservoirs” and are assigned location, elevation, hydraulic grade elevation, and a diurnal curve. The location and elevation are derived from system maps and the aerial survey conducted as part of the infrastructure assessment process. The hydraulic grade elevation and reservoir curves are described in Section 1.1.2. The interconnection along Washington Ave. is currently inactive due to a low supply pressure.

1.1.2 Modeling of Water Demands

Once the physical structures had been represented in the model, the water supply demands were added to the water model. This was done in a way that allowed the model to represent:

- The average in-session water system demands;
- The variability of usage volumes for different building types; and
- The diurnal usage pattern typical of the university.

The methods used to represent the model demands are described below.

1.1.2.1 Calculation of Average System Demand

The system water demand was estimated from monthly water meter records. From this data, the average day water usage rate was calculated for the time period when the campus is in-session. Using monthly water meter readings, the September 2006 – May 2007 school year was taken as a representative time period. The total water usage for that period was divided by the number of days in that period to obtain the average day demand (ADD) in terms of gallons per day (GPD) and gallons per minute (GPM). This calculation is illustrated below in Table 1-1. This average day, in-session water usage rate was used as the base scenario for the capacity model.

Table 1-1: Average Day Demand

Month	Gallons
September	20,562,520
October	27,682,732
November	27,923,837
December	26,188,477
January	23,151,597
February	21,505,000
March	14,922,600
April	22,552,200
May	21,205,800
Total:	205,694,764
Average GPD:	753,461
Average GPM:	523

1.1.2.2 Distribution of Demand Between Nodes

Different volumes of water need to be supplied to different areas of the campus based on the usage patterns in those areas. For example, dormitories or dining halls likely require significantly more water than classroom buildings. To estimate the spatial variation in demand across the University at Albany distribution system, the water usage assigned to each demand node in the capacity model was estimated based on the type of building or structure served by that demand node.

For the water usage types on the campus, it is likely that the wastewater generation rates and water usage rates are very similar. Therefore, wastewater generation patterns were used as the basis for estimating the relative volume of water required by the different building types. The estimated per capita wastewater generation rates specified in the New York State Department of Environmental Conservation (NYSDEC) Design Standards for Wastewater Treatment Works, 1988, were used as weights applied to the ADD calculated in Section 1.1.2.1 to estimate the average day demand for each demand node, as shown in Table 1-2.

Table 1-2: Weights Assigned to Different Demand Node Types

Demand Node Classification	1988 NYSDEC Standards GPD/Person
Classroom	10
Residence/Dining	75
Office Building	15
Community Center	25
Water Fountain	1
Gymnasium	5
Power Plant	25

Using these weights, the ADD for the water system was divided up amongst the demand nodes, as detailed in Figure 1-1.

$$DemandNode = [y_i] \times \left[\frac{ADD}{\sum_i x_i y_i} \right]$$

x_i = total number of demand nodes of classification i

y_i = weight assigned to classification i

ADD = average day demand

Figure 1-1: Calculation of Demand for Each Demand Node

It should be noted that some structures had one demand node serving them, while others had multiple demand nodes. For example, each water fountain had only one node serving it, while the high-rise dormitories like State Quad had nine demand nodes serving them.

1.1.2.3 Diurnal Curve Development

For the water usage types on the campus, it is likely that the wastewater generation rates and water usage rates are very similar. Therefore, data from the sewer flow metering performed by Savin Engineers, PC, as part of the sanitary sewer capacity assessment described in Section 2, was used as the basis for the diurnal water usage pattern. The sewer flow data from the terminal locations of the northern and southern interceptors were averaged and normalized to create demand factor data that could be used with the water system capacity model. By doing this, water system demand at each hour through the day can be calculated by multiplying the demand factor by the ADD. The diurnal curve demand factors are shown in Figure 1-2.

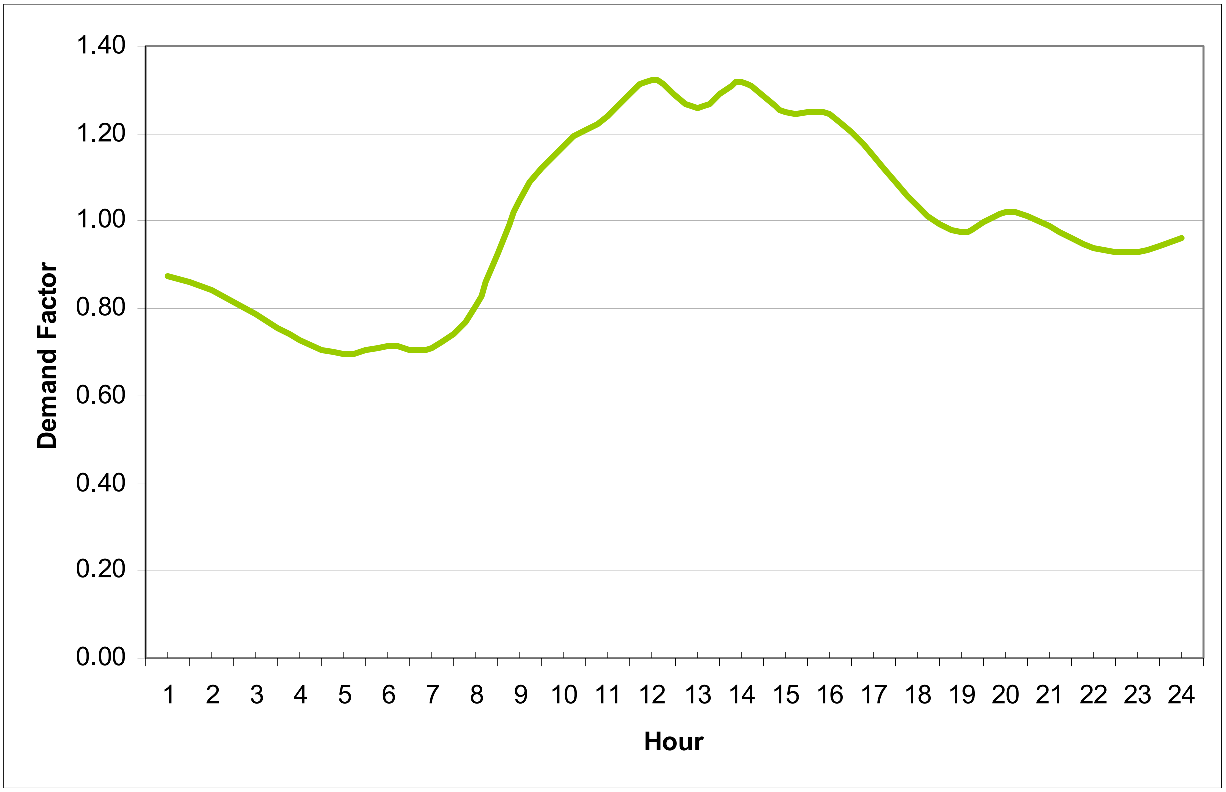


Figure 1-2: Demand Diurnal Curve

1.1.3 Model Calibration

The calibration of the water system model serves to optimize the model performance to closely match conditions observed in the actual water system. This is done through estimating the roughness of the pipes in the model, accurately representing the hydraulic characteristics of the water sources, and validating the model's performance by comparing the model output to observed system conditions.

1.1.3.1 Pipe Roughness Estimation

The roughness of the pipes in the University at Albany water system was estimated using the results of the fire flow testing conducted by Woodard & Curran and University at Albany staff on March 27, 2008. The Data Calibration Module of Infowater® is an optimization tool that adjusts the roughness of the pipes based on the results of fire flow testing and constraints set by the modeler. The optimization minimizes the difference between the fireflow residual pressure and the modeled residual pressure by varying the pipe roughness coefficients of different pipe groups.

For the pipe roughness estimation performed for the University at Albany system, the following assumptions and estimations were used:

1. The fire flow testing results are assumed to be representative of conditions year-round;

2. The range of roughness in pipes was set to be between 50 and 140, where 50 is the roughness of a pipe in bad condition and 140 is the roughness of pipe in excellent condition using the Hazen-Williams roughness coefficient scale.

3. Pipes were grouped into four clusters based on their assumed hydraulic characteristics: 1) pipes immediately next to wHy4 and wHy5 where the highest drop in static to residential pressure was recorded during fire flow testing, 2) pipes in the adjoining area, 3) pipes in the system that are approximately 48 years old based on system mapping, and 4) pipes in the system that are less than 48 years old based on system mapping; and

4. The condition of the tanks, reservoirs, and controls is the same during fire flow testing as it is under normal conditions. There was no data collected on the tanks, reservoirs, or controls during the fire flow testing. We believe this is a reasonable assumption because the system is relatively small.

Using these assumptions, roughness coefficients for the capacity model averaged 129. The average difference between the fire flow residual pressure and simulated residual pressure was 3.1%. Less than 5% is typically acceptable for water system modeling.

1.1.3.2 Hydraulic Characteristics of Supply Sources

1.1.3.2.1 Hydraulic Grade Elevation

The hydraulic grade elevation for each supply source was calculated from data taken from data-logging pressure transducers deployed across the system for a week following the fire flow testing in March and April, 2008. The pressure transducers recorded the system pressure every few minutes. The average pressure of the pressure transducer closest to the reservoir was converted to feet of head, and then adjusted for the elevation difference between the pressure transducer and the supply location. The average pressure and adjusted elevation were added together to get the hydraulic grade elevation.

This hydraulic grade elevation was then given a pressure curve representing the variation in the hydraulic grade elevation over the course of a day, as described in Section 1.1.3.2.2.

1.1.3.2.2 Supply Pressure Curve

The supply pressure curve represents the hydraulic grade elevation of the supply at different hours of the day. For the University at Albany system, the hydraulic grade elevation of the water source governs the water level in the water tank and the pressures around the water distribution system. To derive the supply pressure curve, data taken from the week of pressure transducer deployment was used.

From the week of data obtained from the pressure transducers, several system pressure patterns were observed. These patterns closely followed campus population patterns, with less variation being noted during days when the student population was on break than on in-session days. Figure 1-3, below, shows the observed system pressures for the five pressure transducers deployed across the system for an in-session day. The curves shown exhibit the characteristic system pressure pattern that was observed for all in-session days. The day shown in Figure 1-3 had the largest variation in system pressure of all the days when the pressure transducers were deployed. The pressure variation observed during this day was used to set the supply pressure curve as it was the most conservative choice for predicting water supply capacity.

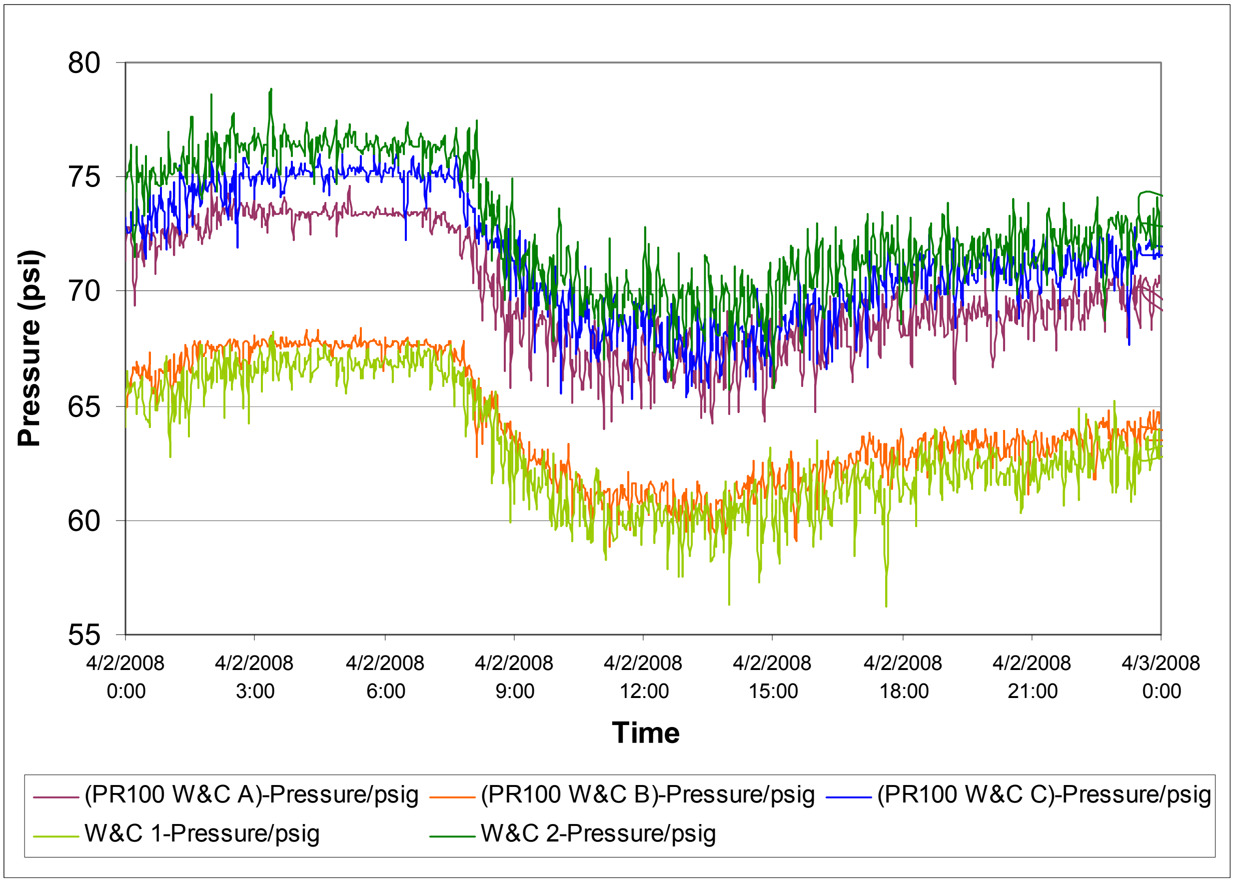


Figure 1-3: Characteristic Pressure Transducer Diurnal Curve

The data from the pressure transducers were averaged to derive the water supply pressure curve, shown in Figure 1-4, below.

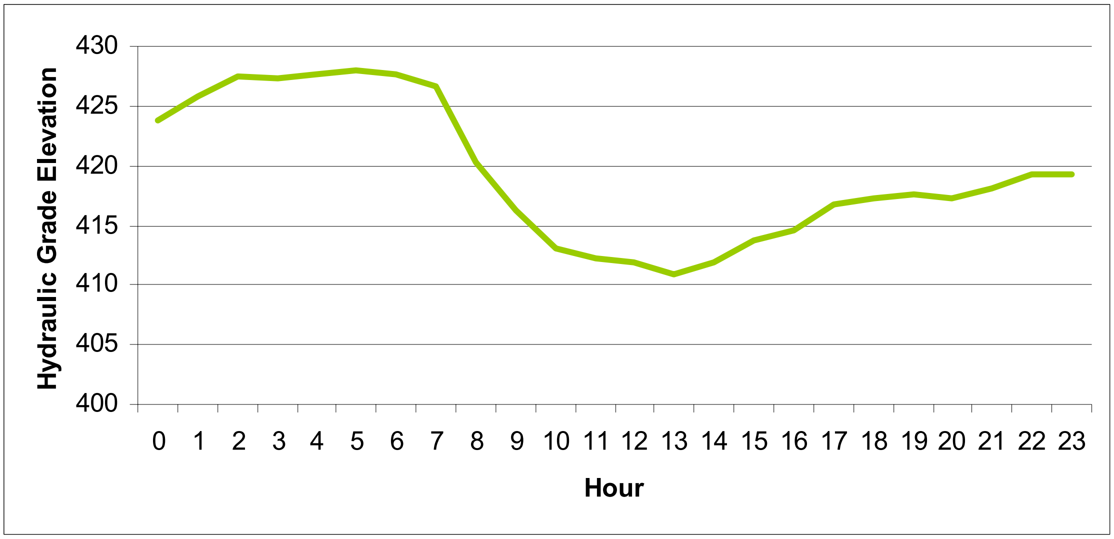


Figure 1-4: Water Supply Pressure Diurnal Curve

1.1.3.3 Model Validation

Several checks were used to validate the accuracy of the model results. As part of the pipe roughness estimation performed for the calibration process, the modeled fire flow availability was compared to the actual fire flow test results. The modeled fire flow results varied by 3.1% compared to the fire flow test results, as described in Section 1.1.3.1.

Additionally, the modeled system pressures were compared to the system pressures observed during the deployment of the pressure transducer and during fire flow testing. The pressures recorded by the five pressure transducers were normalized to the elevation of the flow hydrant for comparison. The average difference between the modeled pressures and hydrant static pressures was 2.3%. The average difference between modeled pressures and pressure transducers data was 0.7%, which are both acceptable variations. Additionally, since the fire flow testing was conducted over the course of the day, we see that the modeled pressures predict the system pressures observed not only for the elevation differences, but also for the diurnal pressure pattern observed in the pressure transducer data. These model validation comparisons are summarized in Table 1-3.

Table 1-3: Model Pressure Validation

Flow Hydrant Number	Time of Test	Hydrant Static Pressure	Average Pressure Transducer Static Pressure	Modeled Static Pressure
wHy5	9:30	75	78	76
wHy206	10:00	78	84	83
wHy202	10:16	75	79	79
wHy219	10:35	81	81	81
wHy15	10:50	72	73	73
wHy22	11:30	75	72	73
wHy28	12:56	68	67	67
wHy225	13:20	63	64	64
wHy6	13:53	75	73	73
wHy6	14:05	74	74	73
Average Difference Between Modeled Pressures and Hydrant Pressures:				2.3%
Average Difference Between Modeled Pressures and Pressure Transducer Pressures:				0.7%

The last check used to validate the model results was a comparison to the historic water levels recorded in the University at Albany water tank. Water tank levels recorded on March 3, 2008, presented in Table 1-4, were compared to modeled water levels in the tank. The modeled system levels are on average 1.5% different than the reported tank levels and qualitatively follow the same fill-draw pattern, which is an acceptable check of the model's validity. It should be noted that information was not available on the datum used for the tank level data provided by the University at Albany.

Table 1-4: Water Tank Levels, March 3, 2008

Time	Tank Level (ft)	Modeled Tank Level (ft)
1:00	171.0	164.6
2:00	172.1	166.0
3:00	172.1	168.8
4:00	172.2	170.6
5:00	172.3	170.6
6:00	172.2	170.4
7:00	172.2	170.5
8:00	-	169.7
9:00	165.4	164.9
10:00	163.4	160.3
11:00	163.1	159.7
12:00	-	160.7
13:00	-	159.8
14:00	163.3	160.4
15:00	163.1	161.4
16:00	164.6	163.2
17:00	165.1	163.4
18:00	165.9	164.8
19:00	167.0	164.8
20:00	167.0	165.4
21:00	167.0	164.7
22:00	167.8	166.1
23:00	169.1	166.7
0:00	169.7	166.0
	Average Difference:	1.5%

1.1.4 Capacity Assessment Scenarios

The intent of the water system model was to predict the available water supply across the University at Albany campus during high system stress scenarios. These high system stress events occur when the availability of water for fire flow demands is most limited. Also, the model was used to predict the supply impacts if a booster pump is added to the currently unused water supply connection at Washington Avenue. The water supply at this location is unused because its supply pressure is lower than the supply pressure of the currently used water source.

The available system capacity was modeled for four different scenarios. For each of these scenarios, the 2-hour fire flow availability was estimated by the model, such that at a 20 psi residual pressure was maintained at all demand nodes in the system. The requirement for a 20 psi residual pressure in the system is based on fire protection standards, including those of the Insurance Services Office, Inc. (ISO), which require a 20 psi residual pressure at the flowing hydrant. By modeling that a 20 psi residual is present throughout the water supply system, the estimated available fire flows are conservative, and protective of the integrity of the piping system.

The existing water usage rate, as described in Section 1.1.2, was used as a base comparison scenario, Scenario 1, to validate the model and to provide information on the average day system dynamics. Beyond this base scenario, three additional system stressors were modeled. They are:

- Scenario 2: Fire flow availability during the peak hour demand situation using the current campus ADD as a basis;

- Scenario 3: Fire flow availability during the peak hour demand situation using an expanded ADD that includes the University's proposed building expansion projects; and

- Scenario 4: Fire flow availability during the peak hour demand situation using the expanded demands and adding a booster station to the Washington Avenue supply to connect it to the University's water system.

The peak hour demands were estimated by applying a peaking factor of four (4) to the ADD. This peaking factor is typical for a water supply system the size of the University's. The fire flow availability during the peak hour demand represents the most extreme demand scenario for a system. The expanded demand was calculated by estimating the University's water usage rates after the planned building expansions are completed. Table 1-5 summarizes the expanded ADD with the planned campus expansion projects provided to Woodard & Curran by the University at Albany Office of Campus Planning. Only those campus expansion projects that would add building footprint or student population were included in the expanded ADD in addition to current demand. For example, the renovation to an existing building or construction of a parking lot would not be included, while the construction of a building addition would be included.

The estimated wastewater generation rates cited in the NYSDEC Design Standards for Wastewater Treatment Works, 1988, were used as a basis for the estimation of the added water demand from the campus expansion projects. The estimated wastewater generation rates were adjusted in two ways for use in the estimation of the expanded ADD. First, the wastewater generation rates were assumed to account for 90% of the water usage rate, estimating that 10% of the water usage would be consumptive uses and losses from the water system. Second, those wastewater generation rates that were not cited in the correct units for direct use in the expanded ADD estimate were adjusted based on other common usage ratios. For example, the NYSDEC Design Standards cite the wastewater generation rate of a classroom to be 10 GPD/capita and that of an office building to be either 15 GPD/capita or 0.1 GPD/square foot. The size of the proposed School of Business expansion, assumed to be largely a classroom use, is stated on a square-foot basis. To estimate the water usage of a classroom for a square-foot basis, the ratio of the two per-capita rates was used to adjust the per square foot usage rate of the office building to an estimated per square foot usage rate for a classroom. This is illustrated in the notes to Table 1-5.

Table 1-5: Campus Expansion Projects with Water Demand Impacts

Project Name	Added Units	Water Usage Rate	Estimated Added Water Usage (GPM)
1. Renovate Health Center	6,400 sf	0.074 GPD/sf*	0.33
2. SBA Renovation	6,400 sf	0.185 GPD/sf**	0.82
3. School of Business	75,000 sf	0.074 GPD/sf	3.86
4. Campus Center Addition	75,000 sf	0.185 GPD/sf***	9.65
5. Student Housing	1,000 beds	83.3 GPD/capita****	57.9
6. Stadium	57,000 seats	0.037 GPD/seat*****	1.46
7. Relocate Data Center	47,000 sf	0.074 GPD/sf	2.42
8. Science Surge Building	40,000 sf	0.074 GPD/sf	2.06
9. Fine Arts Studio	40,000 sf	0.074 GPD/sf	2.06
Total Estimated Added Water Usage (GPM):			**80.82**
Percentage of current ADD Usage:			*15%*

Notes: * Water usage rate for Projects 1, 3, 7, 8, and 9 are calculated as follows: 0.1 GPD/sf (office usage) x 10 GPD/capita (classroom usage) / 15 GPD/capita (office usage) / 0.9 (ratio of wastewater generation rate to water usage rate) = 0.074 GPD/sf (classroom usage).

** Water usage rate for Project 2 was calculated as follows: 0.1 GPD/sf (office usage) x 25 GPD/capita (power plant usage) / 15 GPD/capita (office usage) / 0.9 (ratio of wastewater generation rate to water usage rate) = 0.185 GPD/sf (power plant usage).

*** Water usage for Project 4 was calculated as follows: 0.1 GPD/sf (office usage) x 25 GPD/capita (community center usage) / 15 GPD/capita (office usage) / 0.9 (ratio of wastewater generation rate to water usage rate) = 0.185 GPD/sf (community center usage).

**** Water usage for Project 5 was calculated as follows: 75 GPD/capita (Boarding School usage) / 0.9 (ratio of wastewater generation rate to water usage rate) = 0.074 GPD/sf (classroom usage).

***** Water usage for Project 6 was calculated as follows: 0.1 GPD/sf (office usage) x 5 GPD/captia (gymnasium usage) / 15 GPD/capita / 0.9 (ratio of wastewater generation rate to water usage rate) = 0.037 GPD/sf.

Scenario 4 aims to predict the expanded demand peak hour system performance with the addition of a second water supply. The University at Albany has an interconnection with the City of Albany's water system along Washington

Avenue. According to conversations with University staff, the pressure available from this connection is lower than what is required by the University. To use this source, either as a second source or emergency supply, a booster pump station would be required. Woodard & Curran has modeled the impact on fire flow availability across the University at Albany's campus if a booster pump station matching the hydraulic grade elevation characteristics of its current supply source is added at the Washington Avenue interconnection. This model scenario uses the expanded system demand that includes the planned expansion projects and peak hour factor of (4) as the base demand for this model scenario. This is done because the planned expansion projects will likely be in place by the time a booster pump station is put in place.

These four model scenarios are summarized below in Table 1-6.

Table 1-6: Modeled Scenarios

Scenario Number	Description	Demand Multiplier	Rationale
1	Current Average Day Demand	1	Base comparison scenario
2	Current Peak Hour	4	High system stress scenario at current peak demand
3	Expanded Peak Hour	4.60 *(4 x 1.15*)*	High system stress scenario at expanded peak demand
4	Expanded Peak Hour, Expanded Demand	4.60	High system stress scenario at expanded peak demand, supply includes second water source

Note: * Demand multiplier with the inclusion of the planned campus expansion projects is summarized in Table 1-5.

1.2 CAPACITY ASSESSMENT RESULTS

1.2.1 Model Output

The modeled available fire flow, subject to the constraints described in Section 1.1, was used as the basis for analyzing the capacity of the water system. In Scenario 1, the average day demand was used for model calibration and validation. In Scenario 2, the peak hour scenario for the current demands, is used for analyzing the current fire flow availability. The results for model scenario 1 are therefore not included in the discussion below.

Table 1-7 contains the available fire flow for each hydrant during each of the three peak demand scenarios: current demand, expanded demand, and expanded demand with a second supply source. The hydrants have been grouped by area to discuss trends across the campus. See Section 1.2.2.1 for a discussion of peak demand results and Section 1.2.2.2 for a discussion of the implications of adding a second water source.

Table 1-7: Modeled Capacity Results

Hydrant Node Number	Hydrant Number	Area	Peak Current Demand Available Fire Flow (GPM)	Peak Expanded Demand Available Fire Flow (GPM)	Peak Expanded Demand, Second Source Available Fire Flow (GPM)
J0574	wHY25	Arts and Sciences	7,962	6,218	9,431
J0731	2613	Bohr Studio	2,407	2,370	2,420
J0736	2620	Bohr Studio	4,590	4,491	4,634
J0444	wHY12	Campus Center/Sci Library	9,104	7,362	10,225
J0549	wHY13	Campus Center/Sci Library	2,004	1,651	2,132
J0469	wHY201	Campus Center/Sci Library	6,414	5,287	6,960
J0439	wHY202	Campus Center/Sci Library	5,496	4,671	5,895
J0395	wHY204	Campus Center/Sci Library	9,373	6,801	10,509
J0428	wHY205	Campus Center/Sci Library	9,317	6,643	10,546
J0609	wHY14	Chemistry	871	710	921
J0484	wHY27	Colonial Quad	4,555	3,702	5,907
J0467	wHY28	Colonial Quad	3,913	3,073	4,575
J0626	wHY29	Colonial Quad	3,564	2,873	4,127
J0207	--	Dutch Quad	2,391	1,931	2,546
J0212	wHY210	Dutch Quad	5,144	4,183	5,777
J0126	wHY5	Dutch Quad	3,912	2,147	3,844
J0236	wHY6	Dutch Quad	7,430	4,547	8,693
J0269	wHY7	Dutch Quad	5,533	4,598	5,946
J0368	wHY8	Dutch Quad	7,869	5,721	8,899
J0264	wHY9	Dutch Quad - Pod. W Lot	6,359	4,713	7,613
J0143	2483	Empire Commons	2,475	1,650	2,790
J0270	2559	Empire Commons	2,824	1,901	3,240
J0294	wHY215	Empire Commons	1,472	1,145	1,638
J0177	wHY223	Empire Commons	2,497	1,665	2,821
J0120	wHY225	Empire Commons	2,567	1,743	2,895
J0193	wHY226	Empire Commons	2,677	1,831	3,041
J0185	wHY33	Empire Commons	1,172	912	1,271
J0241	wHY34	Empire Commons	1,642	1,269	1,790
J0154	24	Freedom Quad/Tri-Centennial	1,584	940	1,631
J0224	2442	Freedom Quad/Tri-Centennial	1,902	1,145	1,982
J0112	2446	Freedom Quad/Tri-Centennial	1,317	769	1,341
J0015	2771	Freedom Quad/Tri-Centennial	949	534	947
J0005	3182	Freedom Quad/Tri-Centennial	983	588	997
J0011	3184	Freedom Quad/Tri-Centennial	931	525	929
J0903	--	Indian Quad	11,648	7,752	13,018

Hydrant Node Number	Hydrant Number	Area	Peak Current Demand Available Fire Flow (GPM)	Peak Expanded Demand Available Fire Flow (GPM)	Peak Expanded Demand, Second Source Available Fire Flow (GPM)
J0546	wHY15	Indian Quad	6,846	5,590	7,410
J0536	wHY16	Indian Quad	4,272	3,564	4,433
J0661	wHY220	Justice Dr. - Grounds Bldg.	4,229	3,464	4,287
J0705	wHY219	Justice Dr. - Police Bldg.	4,170	3,422	4,227
J0631	wHy221	Life Sciences	2,119	1,779	2,258
J0699	wHY222	Life Sciences	5,578	4,563	6,087
J0657	wHY23	NE Pod. - Earth Sci & Math	3,000	2,412	3,262
J0619	wHY24	NE Pod. - Fine Arts	4,686	3,728	5,234
J0692	wHY20	State Quad	4,296	3,493	4,699
J0726	wHY21	State Quad	4,509	3,751	4,844
J0691	wHY22	State Quad	3,352	2,770	3,594
J0038	wHY1	Support Bldg	2,059	1,675	2,188
J0057	wHY2	Support Bldg	2,590	2,012	2,763
J0086	wHY3	Support Bldg	2,536	2,060	2,708
J0105	wHY4	Support Bldg	3,539	1,944	3,457
J0133	1043	University Field Area	2,832	2,432	2,967
J0205	wHY206	University Field Area	3,698	3,161	3,879
J0197	wHY207	University Field Area	3,593	3,108	3,755
J0293	wHY208	University Field Area	3,364	2,899	3,516
J0318	wHY209	University Field Area	3,329	2,852	3,484
J0431	wHY250	University Field Area	4,159	3,558	4,363
J0249	wHY30	University Field Area	3,857	3,300	4,050
J0369	wHY31	University Field Area	3,944	3,374	4,137
J0475	wHY26	West Pod. - Bus. Bldg.	2,636	2,069	2,968
J0379	wHY10	West Pod. - Soc. Sci. Bldg.	3,176	2,563	3,557

Figures 1, 3, and 5 in Appendix C show the hydraulic grade elevation at each hydrant junction under peak non-fire flow conditions and the estimated flow through the system's pipes during non-fire flow peak hour conditions. Figures 2, 4 and 6 in Appendix C show the estimated available design flow to each hydrant and the diameter of each pipe. It should be noted that in Figures 1 through 4, the Washington Avenue source is inactivated and is not showing.

The figures in Appendix C depict trends in the model results over large areas of the campus. One trend is the decrease in the modeled hydraulic grade elevation at points further away from the current source in the single source scenarios. In the scenario with the Washington Avenue source activated, the modeled hydraulic grade elevation shows a decrease in areas such as southeastern University Field, which is far from both sources. Reductions in the modeled hydraulic grade elevation were most significant at the extremities of smaller diameter pipe (for example, a 4-inch diameter pipe). This trend also leads to the model result that the fire flow availability is also closely tied to the diameter of the service main.

The model results are described for each area of the campus in Section 1.2.1.1.

1.2.1.1 Model Output Discussion by Area

Arts and Sciences Building Area

The fire hydrant serving the Arts and Sciences Building has a modeled available flow of 7,962 gallons per minute GPM during current peak conditions and 6,218 GPM during expanded demand conditions. An 8-inch diameter lateral connects the hydrant to a 12-inch main, near the water tower, which likely accounts for the high available flow.

Bohr Studio Area

The fire hydrants serving the Bohr Studio Area have modeled available flows ranging from 2,407 GPM to 4,590 GPM during current peak conditions and 2,370 GPM to 4,491 GPM during expanded demand conditions. The hydrants are connected to a 6-inch main by 6-inch laterals, near the currently used supply. The large range in available fire flow is likely due to the 6-inch main restricting available flow to hydrants as they get further from the supply source.

Campus Center and Science Library Area

The fire hydrants serving the Campus Center and Science Library Area have modeled available flows ranging from 2,004 GPM to 9,373 GPM during current peak conditions and 1,651 GPM to 7,362 GPM during expanded demand conditions. The hydrant with the lowest available fire flow, wHY13, is located on a 6-inch diameter lateral off of a 6-inch diameter main, which is restricting the flow to this hydrant. The rest of the hydrants are on 6-inch laterals off of 12-inch mains.

Chemistry Building Area

The fire hydrant serving the Chemistry Building has a modeled available flow of 871 GPM during current peak conditions and 710 GPM during expanded demand conditions. This is the lowest modeled design flow of all of the hydrants. This hydrant is located on a 4-inch lateral, which is limiting the flow to this hydrant.

Colonial Quad Area

The fire hydrants serving the Colonial Quad Area have modeled available flows ranging from 3,564 GPM to 4,555 GPM during current peak conditions and 2,873 GPM to 3,702 GPM during expanded conditions. The hydrant with the lowest flow is located on a 6-inch lateral off of an 8-inch main. The remaining hydrants in this area are located on 6-inch laterals off of 12-inch mains, resulting in their higher available flow.

Dutch Quad Area

The fire hydrants serving the Dutch Quad Area have modeled available flows ranging from 2,391 GPM to 7,869 GPM during current peak conditions and 1,931 GPM to 5,721 GPM during expanded demand conditions. The hydrants with higher available flow are on 6-inch laterals off of 12-inch mains, and the hydrants with lower available flow are on 6-inch laterals off of 8-inch mains.

Empire Commons Area

The fire hydrants serving the Empire Commons Area have modeled available flows ranging from 1,172 GPM to 2,824 GPM during current peak conditions, and 912 GPM to 1,901 GPM during expanded demand conditions. The hydrants with higher available flows are on 6-inch laterals off of 8-inch mains, and the hydrants with lower available flows are on 6-inch laterals off of 6-inch mains.

Freedom Quad and Tri-Centennial Dr. Area

The fire hydrants serving the Freedom Quad and Tri-Centennial Dr. have modeled available flows ranging from 931 GPM to 1,902 GPM during current peak conditions and 525 GPM to 1,145 GPM during expanded demand conditions. This area contains the second lowest available flows, which is most likely a result of them being the furthest away from the current water supply. The hydrants with higher available flows are the closest to the water supply. The hydrants in this area are on 6-inch laterals off of 8-inch mains.

Indian Quad Area

The fire hydrants serving the Indian Quad Area have modeled available flows ranging from 4,272 GPM to 11,648 GPM during current peak conditions and 3,564 GPM to 7,752 GPM during expanded demand conditions. The hydrant with highest available flow is on a 6-inch lateral off of a 12-inch main in the model, closest to the water source. The hydrants with lower available flows are on 6-inch laterals off of 8-inch mains.

Justice Drive Area

The fire hydrants serving the Justice Drive Area have modeled available flows ranging from 4,170 GPM to 4,229 GPM during current peak conditions and 3,422 GPM to 3,464 GPM during expanded demand conditions. The hydrants are both on 6-inch laterals off of the same 12-inch main.

Life Sciences Building Area

The fire hydrants serving the Life Sciences Building have modeled available flows ranging from 2,119 GPM to 5,578 GPM during current peak conditions, and 1,779 GPM to 4,563 GPM during expanded demand conditions. The hydrant with the lower flow is on a 6-inch lateral off a 6-inch main and the hydrant with the higher flow is on a 6-inch lateral off of a 12-inch main.

Northeast Podium Area

The fire hydrants serving the Northeast Podium have modeled available flows ranging from 3,000 GPM to 4,686 GPM during current peak conditions, and 2,412 GPM to 3,728 GPM during expanded demand conditions. The hydrant with the higher flow is on a 6-inch lateral off a 8-inch main and the hydrant with the lower flow is on a 6-inch lateral off of a 6-inch main.

State Quad Area

The fire hydrants serving the State Quad Area have modeled available flows ranging from 3,352 GPM to 4,509 GPM during current peak conditions, and 2,770 GPM to 3,751 GPM during expanded demand conditions. The hydrant with the highest flow is on a 6-inch lateral off a 12-inch main and the rest of the hydrants are located on 6-inch laterals off of 8-inch mains.

Support Building Area

The fire hydrants serving the Support Building have a modeled available flow ranging from 2,059 GPM to 3,539 GPM during current peak conditions, and 1,675 GPM to 2,060 GPM during expanded demand conditions. The hydrants with the highest flow are on 6-inch laterals off 12-inch mains and the other hydrants are located on 6-inch laterals off of 8-inch mains.

University Field Area

The fire hydrants serving the University Field Area have a modeled available flow ranging from 2,832 GPM to 4,159 GPM during current peak conditions, and 2,432 GPM to 3,558 GPM during expanded demand conditions. The hydrants with the highest flows are on 6-inch laterals off 8-inch mains and the other hydrants are located on 6-inch laterals off of 6-inch mains

West Podium Area

The fire hydrants serving the West Podium Area have a modeled available flow ranging from 2,636 GPM to 3,176 GPM during current peak conditions, and 2,069 GPM to 2,563 GPM during expanded demand conditions. The hydrant with the highest flow is on a 6-inch lateral off an 8-inch main and the other hydrant is located on a 6-inch lateral off of a 6-inch main.

1.2.1.2 Addition of Second Water Source – Scenario 4 Results

The hydrants in areas closest to the added water source had the largest increases in available fire flow. Accordingly, the hydrants furthest from the new water source had the smallest increase in available fire flow.

The areas listed below contained hydrants with the most significant increase in available fire flow (greater than 20%) with the addition of a second water source. The area with the greatest increase in flow was Colonial Quad, which is the area closest to the added water source, with a modeled increase in flow ranging from 21-35%. The Dutch Quad Area, which is immediately below Colonial Quad, had the next highest increase in available flow ranging from 12-25%. The next highest increases in available fire flow occurred at the hydrants in the Arts and Sciences Building Area, directly east of the Colonial Quad, at 24%. Finally, the Empire Commons, directly west of Colonial Quad, had a modeled increase of 14-22%.

Each of the following areas contains hydrants that experienced a moderate increase in available fire flow (between 10% and 20%) when the second source was added. The areas with the greatest increase in flow in this group were Freedom Quad and Tri-Centennial Dr. with increases ranging from 15-18%, the Campus Center and Science Library at had increases ranging from 10-18%, the West Podium area had an increase of 17%, the Northeast Podium Area had increases ranging from 13-17% and Indian Quad had increases ranging from 7-16%. The areas with the lowest increase in flow in this group were the Support Building with an 11-14% increase, the State Quad with a 10-14% increase, the Life Sciences Building with a 10-13% increase, and the Chemistry Building with a 10% increase.

The third group of areas contains hydrants that experienced the smallest increase in available fire flow (less than 10%) with the addition of a second source. The University Field Area had an increase of 7-8%. The Justice Drive area had an increase of 5%. The Bohr Studio Area had an increase of 1-1.5%. Note that these three areas are the furthest away from the second water source, and thus are affected the least by the addition of a second water source.

1.2.1.3 Additional Discussion

When the model was run to determine the available fire flow, one model output was the critical node. The critical node is the first node at which the pressure would fall below the 20 psi constraint if the available flow was increased. Most critical nodes are the nodes where a hydrant is flowing. However, in several instances, the two nodes J0008 and J0111 were determined to be the critical node rather than the flow hydrant node. Node J0008 is in Freedom Quad and most likely has a restricting pressure because it is in one of the furthest locations from the water supply and it is connected to a main by a 2-inch diameter pipe. Node J0111 is located at the Alumni House and most likely has a restricting pressure because it is also at one of the furthest locations from the water supply and is connected to a main by a 2-inch diameter pipe. In a fire event, if it is not important for pressure to remain at or above 20 psi at these two locations, some additional fire flow is likely available at certain nodes. See Appendix D for a chart including critical nodes; critical nodes which differ from the flow node are highlighted.

1.2.2 Conclusions

Based on model results, we recommend the following projects be completed:

- The modeled fire flows should be compared to the required fire flow for each building served;
- The size of pipe should be increased or a booster pump should be added going to the Alumni House and Freedom Quad junctions if it is important for the Alumni House and Freedom Quad to maintain a pressure of greater than 20 psi during fire flow under peak system conditions;
- The diameter of the pipe to the hydrant next to the Chemistry Building should be increased (hydrants require a 6-inch diameter line to meet fire hydrant design criteria); and
- A booster pump should be added to the Washington Avenue interconnection. Doing so will add a second supply source to the campus for supply interruptions during normal operating situations, and increase the available fire flow to the majority of the campus during emergency events.

2. SANITARY SEWER SYSTEM

2.1 METHOD

The sanitary sewer system capacity was assessed by building a capacity model, flow metering, and incorporating pipe condition information.

The capacity model estimates a capacity design flow for each individual pipe of the system, based on mapping data. In order to develop an understanding of how the current system is functioning, flow metering data was collected. The Condition Assessment Report developed by Woodard & Curran was used to further estimate current pipe capacity based on information on blockages and sags in pipes.

The capacity model, flow metering, and condition assessment information are discussed separately below. Overall estimated capacity is discussed in the Conclusions section, Section 2.5.

2.1.1 Capacity Model

A capacity model was developed to estimate the flow capacity of the sewer network. The capacity model uses Manning's Equation for partially-filled pipes to estimate the flow capacity for each pipe. The Manning's Equation estimates flow capacity based on the slope of the pipe, the diameter of the pipe, and the roughness of the pipe. The flow capacity of a pipe is directly proportional to the diameter and slope of the pipe, and inversely proportional to the roughness.

The source of data used to calculate the pipe slopes was the plan titled Site Utilities, by Edward Durell Stone, Architect, dated April 4, 1970, because it has the most accurate data available on manhole invert elevations. The diameters of the pipes were taken from field data collected by Woodard & Curran from June through August, 2007 and historic mapping. The roughness of the pipes was estimated based on industry standards for the pipe materials in use in the sanitary sewer system.

Manning's Equation was used to calculate the estimated flow capacities of each pipe. The Manning Equation relates the flow (Q), the roughness of the pipe (n), the hydraulic radius of the pipe (R), the slope of the pipe (S), and the cross-sectional area of the pipe which is full of water (A). The equation is as follows:

$$Q = \frac{1.486}{n} R^{2/3} S^{1/2} A$$

The hydraulic radius (R) and the cross-sectional area (A) can be combined and written in terms of the water depth ratio, d/D, which is equivalent to the depth of flow divided by pipe diameter. Thus, the equation can be rewritten in terms of d/D:

$$Q = \frac{1.486}{n} D^{8/3} S^{1/2} \frac{\left(\frac{1}{8}(2\cos^{-1}(1-2\frac{d}{D}) - \sin(2\cos^{-1}(1-2\frac{d}{D})))\right)^{5/3}}{(\cos^{-1}(1-2\frac{d}{D}))^{2/3}}$$

The standard graph of this Manning's Equation is the curve below, Figure 2-1. The ratio Q/Q_{max} relates the flow of water in the pipe to the maximum possible flow through the pipe. The maximum Q/Q_{max} occurs when the pipe is roughly 90% full of water.

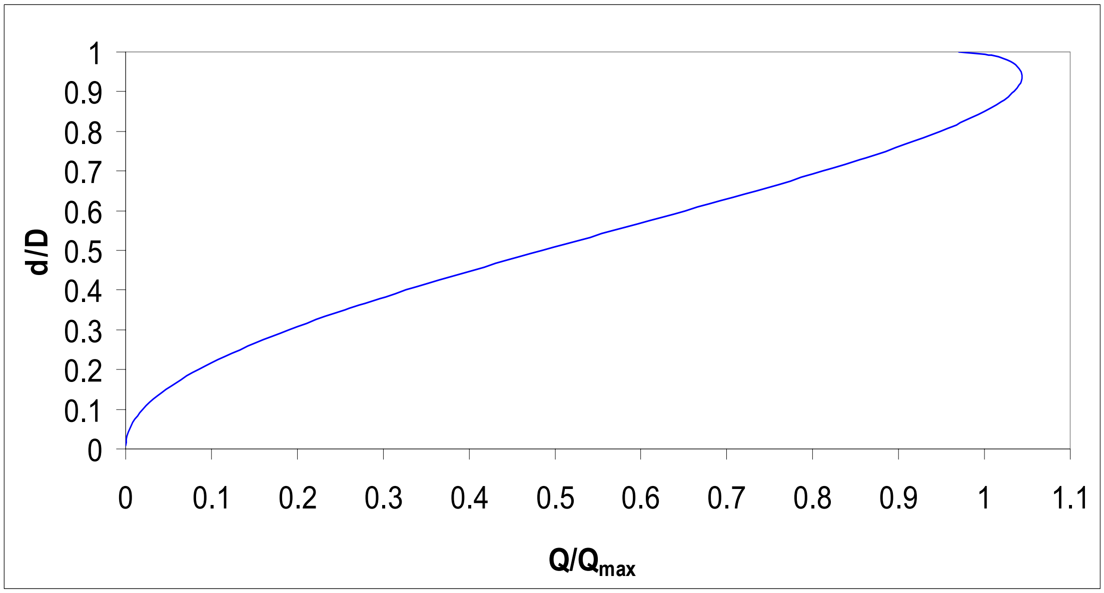


Figure 2-1: Manning's Curve

For a given water depth ratio, the curve in Figure 2-1 shows the maximum flow that can be conveyed through the pipe as a function of the maximum flow through the pipe. The Manning's Equation curve shows that the highest flow through a pipe happens when the pipe is approximately 90% full. This correlates with good design practice which recommends that the full pipe capacity is 90%.

The modeled flow for each pipe estimates the greatest possible flow through each pipe based on slope, diameter and estimated roughness. The model does not account for adverse pipe conditions such as debris in the pipe, sags, or other blockages, which could change reduce the maximum flow capacity of the pipe.

2.1.2 Flow Metering

The flow meter installation and monitoring was conducted by Savin Engineers, PC, of Pleasantville and took place from May 16, 2007 to June 14, 2007. The flow data was collected at four locations and then analyzed to determine the remaining flow capacity. Precipitation data was also collected during the flow metering period so that a correlation between precipitation and flow could be analyzed for potential infiltration/inflow issues.

The four flow metering locations were chosen to characterize contributing flows of the system. Location 1 was at MH145 (F11_sMH03) between the northeast softball field and Washington Avenue, and was chosen to characterize the flow at the end of the northern interceptor. Location 2 was at MH103, on the north side of Justice Drive between the University Police Building and University Drive East, and was chosen to characterize the flow at the end of the southern interceptor. Location 3 was at MH86 which is just south of Building 1 in the Indian Quad, and was chosen to characterize the flow contributions in the middle of the southern interceptor, after flows from the University Field and southeastern portion of the Podium have entered the system. Location 4 was at MH46 near Building 15 at the southwest corner of the Dutch Quad and was chosen to characterize the flow contributions upstream in the southern interceptor. At each of these metering locations, a flow meter was installed on May 16, 2007. The flow meters at Locations 1, 2 and 4 were installed on the inflow pipe. The meter at Location 3 was installed on the effluent pipe. The precipitation gauge was installed on the roof of the Power Plant. See Figure 2-2.

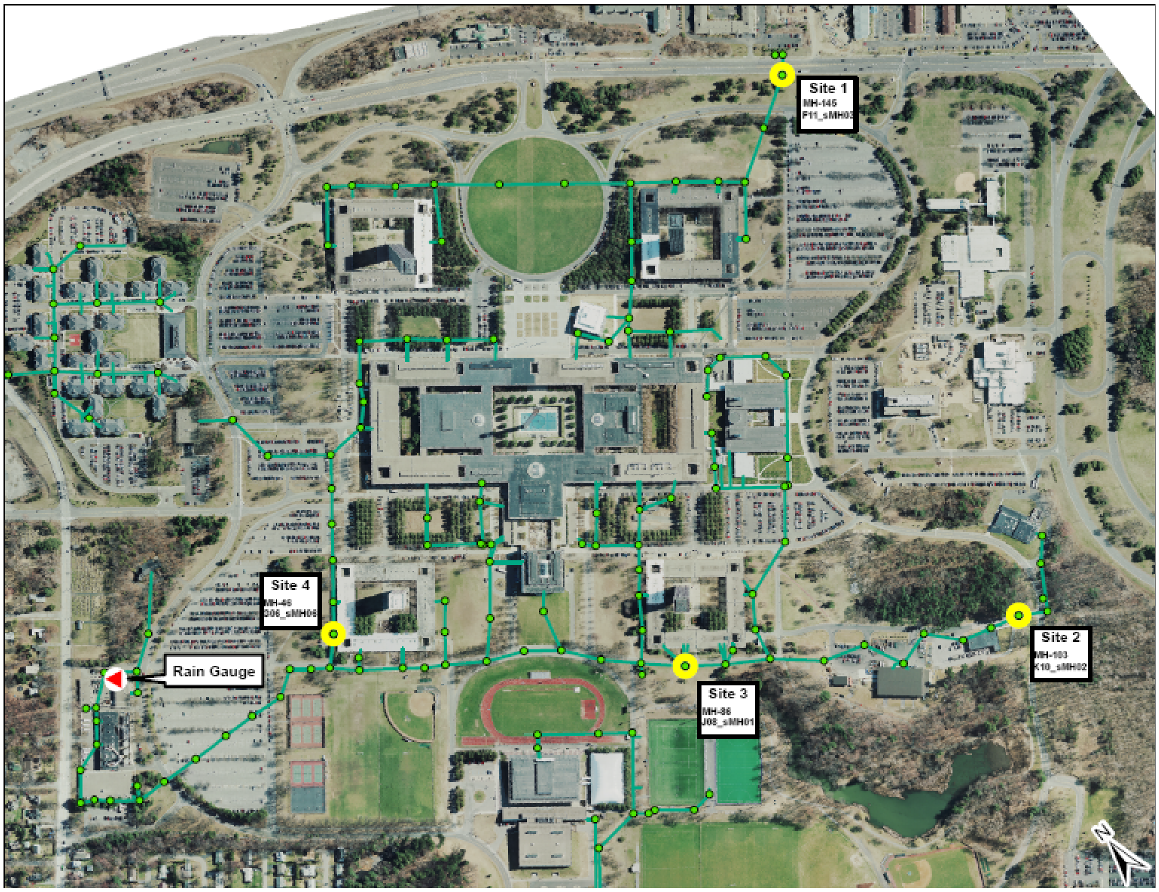


Figure 2-2: Flow Metering and Precipitation Gauge Locations

During the first two days of flow metering, May 16, 2007 and May 17, 2007, there were final exams taking place. The flow during these two days is reflective of the in-session flows. Commencement was during the weekend of May 19, 2007 and May 20, 2007, so a smaller population was on campus at this time, and thus a reduced flow. The remaining portion of the metering dates reflects a summer population, with the lowest flows.

The flow metering data collected at each of the four locations consisted of flow measurements and depth of flow in the pipe. From this data, the capacity of the sanitary sewer system was analyzed using several capacity indicators. One indicator was the maximum instantaneous flow in the pipe. This number indicates the peak flow in the pipe during the flow metering period. Another indicator was the average day flow (ADF), which is descriptive of typical flow conditions in the pipe. Another indicator was the correlation between flow in the pipe and precipitation, which is an indicator of inflow and infiltration to the sanitary sewer system. The final indicator was the trends observed in the flow through the pipe and the ratio of the depth (d) of the water in the pipe to the diameter of the pipe (D), which will be referred to as “d/D” or the water depth ratio.

Using Manning’s Equation for partially-filled pipes, the pipe flow capacity can be estimated, providing an estimate of the remaining flow capacity in the system. These estimates of flow capacity at each location were then analyzed with respect to the pipe conditions found during the Condition Assessment, as summarized in Section 2.1.3.

2.2 CONDITION ASSESSMENT SUMMARY

The Condition Assessment analysis conducted by Woodard & Curran was reviewed to analyze the impact of pipe conditions to the estimated flow capacity. Flow capacity can be constrained by condition issues, such as blockages and grease build-up that decrease the cross-sectional flow area of the pipe. Flow capacity is often also limited by pipe sags because they facilitate the build-up of material in the sag, and result in a decreased cross-sectional flow area of the pipe.

The details of the Condition Assessment activities are presented separately in the Condition Assessment Report, and summarized below. In this assessment, each pipe was assigned a condition by Woodard & Curran using the categories of "excellent," "good," "fair," and "poor." These conditions were assigned based on the presence of cracks/breaks, pipe blockages, grease, sags or fine roots.

2.2.1 Northern Interceptor

The northern interceptor is on average in poor condition. Blockages in the pipes range from 0-85%. Sags in the pipes range from 0-50%. Many pipes also contain grease and fine roots. Three of the pipes also contain cracks. The northern interceptor is shown in Figure 2-3, with the following color-coding:

- Red pipes = poor condition
- Orange pipes = fair condition
- Yellow pipes = good condition
- Green pipes = excellent condition

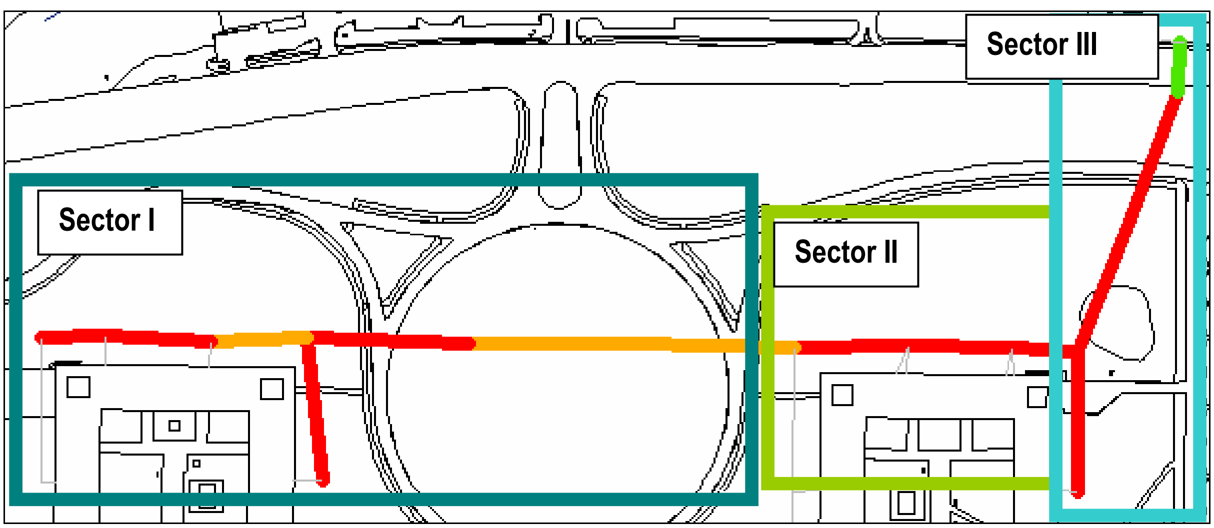


Figure 2-3: Northern Interceptor Condition

2.2.1.1 Sector I

Seven sanitary sewer pipe segments were inspected in Sector I. They ranged in condition from poor to fair. The pipes contained:

- Cracks – two pipes
- Blockages 20-70% – five pipes
- Sags 15-25% – five pipes,
- Grease 5% – five pipes, and
- Fine Roots – five pipes.

In summary, the average condition of the pipes in this area was poor. The poor condition of the pipes in Sector I could significantly reduce their flow capacity.

2.2.1.2 Sector II

Three sanitary sewer pipes were inspected in Sector II. All three pipes were in poor condition. The pipes contained:

- Blockages 20-60% – three pipes,
- Sags 30-50% – three pipes,
- Grease 5% – two pipes, and
- Fine Roots – two pipes.

In summary, the average condition of the pipes in this area was poor. The poor condition of the pipes in Sector II could significantly reduce their flow capacity.

2.2.1.3 Sector III

Four sanitary sewer pipes were inspected in Sector III. Three of the pipes were in poor condition and one pipe was in excellent condition. The pipes contained:

- Cracks – one pipe,
- Blockages 20-85% – two pipes,
- Sags 30% – two pipes, and
- Fine Roots – two pipes.

In summary, the average condition of the pipes in this area was poor. The poor condition of the pipes in Sector III could significantly reduce their flow capacity.

2.2.2 Southern Interceptor

The southern interceptor is on average in fair/poor condition. Blockages in the pipes range from 0-90%. Sags in the pipes range from 0-75%. Many pipes also contain grease and fine roots. Four of the pipes also contain cracks. The northern interceptor is shown in Figure 2-4, with the following color-coding:

- Red pipes = poor condition
- Orange pipes = fair condition
- Yellow pipes = good condition
- Green pipes = excellent condition

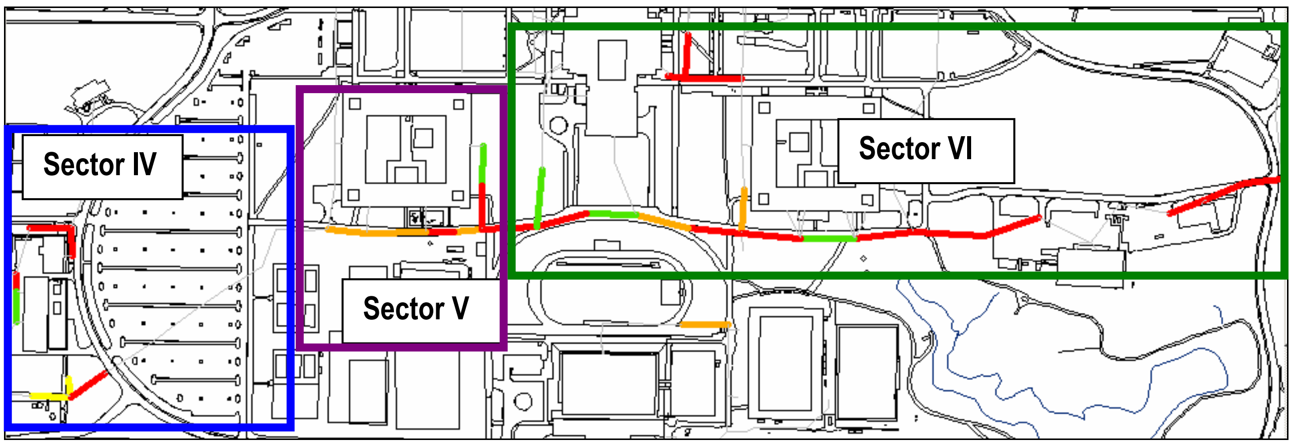


Figure 2-4: Southern Interceptor Condition

2.2.2.1 Sector IV

There were seven sanitary sewer pipes inspected in Sector IV. They ranged in condition from poor to excellent. They contained:

- Blockages 25-90% – three pipes
- Sags 25-75% – three pipes, and
- Fine Roots – three pipes.

In summary, the average condition of the pipes in this area was fair. The furthest downstream pipe inspected in Sector IV was in poor condition with 25% blockages. This may constrain the estimated flow capacity of Sector IV. The poor condition of the pipes in Sector IV could reduce their flow capacity.

2.2.2.2 Sector V

There were four sanitary sewer pipes inspected in Sector V. They ranged in condition from poor to fair. They contained:

- Cracks – one pipe,
- Sags 25-30% – four pipes,
- Grease 10% – one pipe, and
- Fine Roots – two pipes.

In summary, the average condition of the pipes in this area was fair. The fair condition of the pipes in Sector V could reduce their flow capacity.

2.2.2.3 Sector VI

There were thirty pipes inspected in Sector VI. They ranged in condition from poor to excellent. They contained:

- Cracks – three pipes,
- Blockages 25-90% – ten pipes,
- Sags 15-60% – fourteen pipes,
- Grease 20-35% – two pipes, and
- Fine Roots – twelve pipes.

In summary, the average condition of the pipes in this area was fair. The pipes north of University Field contain blockages of 30-45% that may constrain flow through the upstream portion of Sector VI. The pipes downstream along Justice Drive contain sags of 35-60% that may collect enough debris to significantly reduce flow capacity through the downstream portion of the southern interceptor. Overall, the fair condition of the pipes could reduce the flow capacity of Sector VI.

2.3 CAPACITY MODEL RESULTS

The estimated flow capacities of each pipe based on the capacity model are color-coded in Figures 2-3 and 2-4, with lighter blue being higher flow capacity and darker blue representing lower flow capacity. The northern interceptor and southern interceptor were each divided into three sectors each for analysis.

Sections below discuss six individual sectors. The interaction between sectors is discussed in Sections 2.4.1 and 2.4.2.

2.3.1 Northern Interceptor

- The Northern Interceptor sanitary sewer includes the length of sewer pipe beginning at Colonial Quad and ending northeast of the softball fields north of State Quad. This interceptor has been divided into three capacity sectors based on general capacity trends and areas of possible development, as shown in Figure 2-5. The northern interceptor is shown in Figure 2-5 with the lighter colors representing higher flow capacity and darker colors representing lower flow capacity. Grey indicates that the capacity was not able to be calculated for that segment.

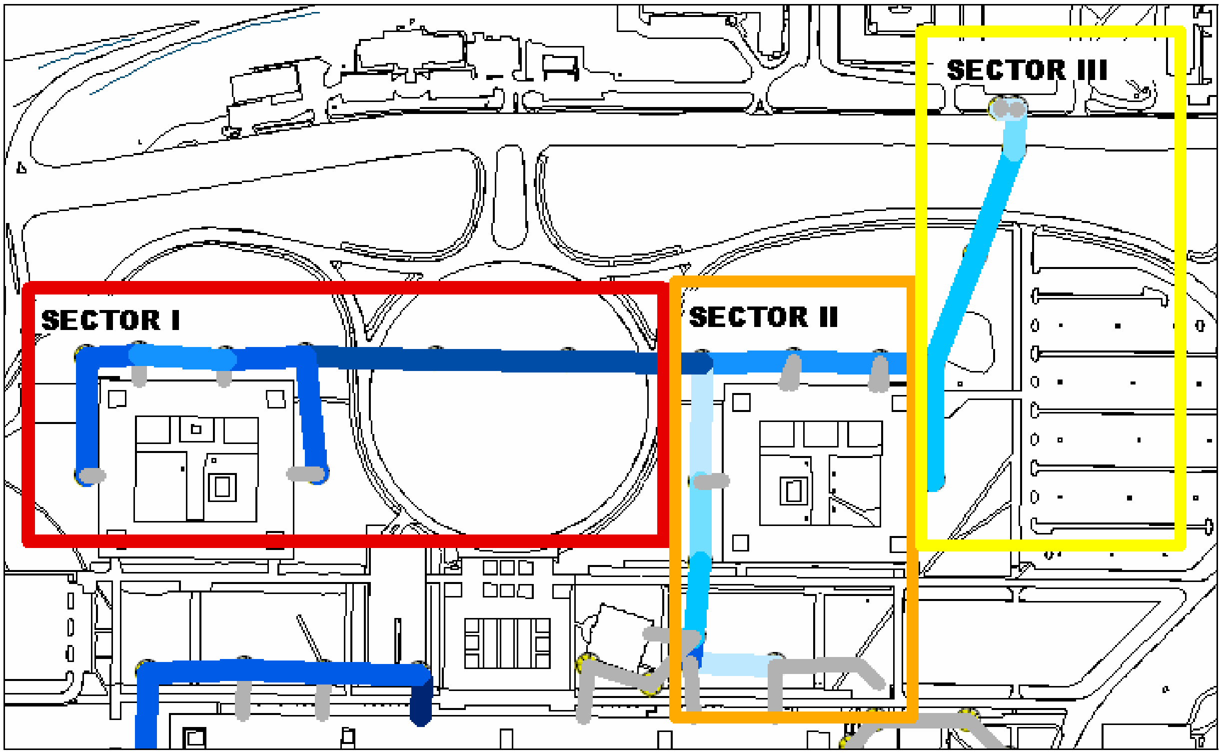


Figure 2-5: Northern Interceptor Capacity – Sectors I – III

2.3.1.1 Sector I

Sector I includes sanitary sewer pipes north of Colonial Quad and in Collins Circle. Estimated maximum flow capacities in this area range from 0.47 to 1.01 million gallons per day (MGD). The pipes all have the same diameter, so flow capacity is not diameter-driven. There are laterals from the east and west sides of Colonial Quad that flow into this section of the northern interceptor.

The pipes under Collins Circle, which are the furthest downstream in Sector I have the lowest estimated flows because of their low slopes. These pipes constrain the overall estimated flow capacity of this sector to 0.47 MGD.

2.3.1.2 Sector II

Sector II includes sanitary sewer pipes north and west of State Quad. Estimated maximum flow capacities in this area range from 1.32 to 1.47 MGD. There is a lateral that flows into this section of the northern interceptor from north of the Fine Arts Building and west of State Quad, which has a maximum capacity of 4.39 MGD. The average slope of pipe in the lateral is much larger than the slope of the section of interceptor, hence the much larger flow capacity in the lateral than in the section of interceptor.

The diameters of all of the pipes in Sector II are the same. The pipe with the lowest estimated flow in Sector II is the furthest pipe downstream. This pipe had the lowest slope and limits the flow capacity of this sector to 1.32 MGD.

The flow capacity of this sector, 1.32 MGD, is higher than its upstream sector, Sector I, which had a flow capacity of 0.47 MGD. Therefore, Sector II should not constrain flow from Sector I.

2.3.1.3 Sector III

Sector III includes sanitary sewer pipes northeast of State Quad flowing north up to Washington Street. Estimated maximum flow capacities in this area range from 1.62 to 6.84 MGD. The high estimated flows in this area are both diameter and slope driven. The average diameter of pipes in this area is approximately twelve inches. The slopes of pipes in this area are on average much higher than in other sections. There is a lateral that flows into this section of the northern interceptor from the east side of the State Quad, with a flow capacity of 1.83 MGD.

The flow capacities of the pipes increase from upstream to downstream, so there are no downstream capacity limitations in this Sector. The flow capacity of the lateral flowing into the interceptor, 1.83 MGD, is slightly higher than the pipe it flows into, 1.62 MGD, so the capacity of the lateral is limited to 1.62 MGD.

The upstream flow capacity of this sector, 1.62 MGD, is greater than the overall flow capacity of the upstream sector, Sector II, which had a flow capacity of 1.32 MGD. Therefore, Sector III should not constrain flow from Sector II.

2.3.2 Southern Interceptor

The Southern Interceptor includes the length of sanitary sewer pipe beginning at the Support Building and ending near the intersection of Justice Drive and University Drive East. This interceptor has been divided into three Capacity Sectors based on general capacity trends and areas of possible development, as shown in Figure 2-6. The northern interceptor is shown in Figure 2-6 with the lighter colors representing higher flow capacity and darker colors representing lower flow capacity. Grey indicates that the capacity was not able to be calculated for that segment.

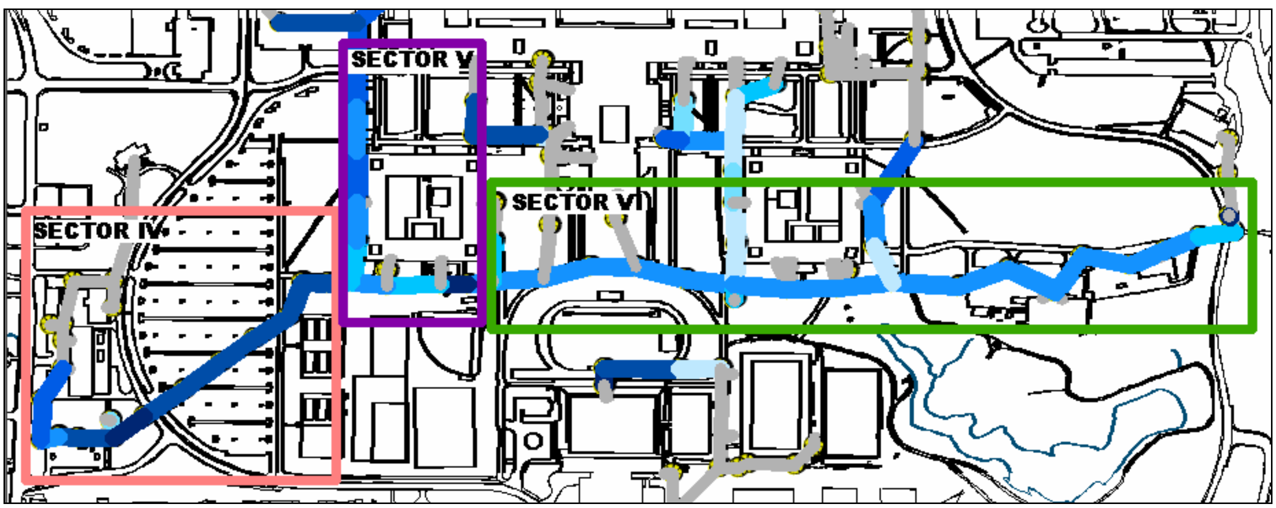


Figure 2-6: Southern Interceptor Sectors IV - VI

2.3.2.1 Sector IV

Sector IV includes sanitary sewer pipes from the Support Building area to the southeast corner of Dutch Quad. Estimated maximum flow capacities in this area range up to 1.22 MGD. The difference in capacity in the pipes is slope-driven, given that the pipes all have the same diameter and material. There is one lateral flowing into this section of the southern interceptor from the east side of Support Building C.

The pipe from the southeastern corner of Support Building C to the manhole next to University Drive West has an estimated gravity flow capacity of 0.0 MGD because it has no apparent slope. This pipe is a bottleneck for flow from the Support Building Area and the lateral from east of the Support Building C, which has an estimated flow of 2.36 MGD. Flow in the pipe will flow by momentum and pressure and not by gravity, since there is no slope to drive the flow. The pipe therefore will not necessarily achieve a self-cleaning velocity of 2 feet per second (ft/s), so settling of solids is likely to occur in this section of pipe.

The estimated maximum flow capacities of the remainder of Sector IV range from 0.38 to 0.44 MGD. The pipe with the lowest estimated flow in this section of Sector IV is the furthest pipe downstream. This pipe limits the flow capacity of this sector to 0.38 MGD.

2.3.2.2 Sector V

Sector V includes sanitary sewer pipes south of Dutch Quad. Estimated maximum flow capacities in this area range up to 1.54 MGD. The difference in capacity in the pipes is slope-driven, given that the pipes all have the same diameter. There are three laterals coming into this section of interceptor. Two of the laterals are from buildings on the southern side of Dutch Quad. The third lateral begins north of the Business Building, runs west of the Social Sciences Building and south along the west side of the Dutch Quad to the southern interceptor. This lateral has estimated maximum flow capacities ranging from 0.44 to 1.71 MGD.

The pipe south of Building 10 in Dutch Quad, under the UKids Daycare, has an estimated gravity flow capacity of 0.0 MGD because it has no apparent slope. The pipe is a bottleneck that will restrict flow through the southern interceptor from Sector IV, the upstream portion of the southern interceptor in Sector V, and the three laterals that flow into Sector V. Flow in the pipe will flow by momentum and pressure and not by gravity, since there is no slope to drive the flow. The pipe therefore will not necessarily achieve a self-cleaning velocity of 2 ft/s, so settling of solids is likely to occur in this section of pipe.

The pipe segment with the least flow capacity in Sector V that occurs downstream of the segment with zero slope is 1.28 MGD. However, as discussed below, the first pipe segment in Sector VI downstream of Sector V has a flow capacity of 1.17 MGD. The capacity of Sector V is therefore limited by the downstream condition to 1.17 MGD.

2.3.2.3 Sector VI

Sector VI includes sanitary sewer pipes from the northwest corner of University Field to where Justice Drive intersects University Drive East. Estimated flows in this area range from 1.17 to 1.55 MGD. The difference in capacity in the pipes is slope-driven, given that the pipes all have the same diameter. There are laterals into this section of southern interceptor from the east side of Dutch Quad; the Humanities and Education buildings; the Science Library; the east side of the campus center; the Physics Building; the Chemistry Building; the west, east and southern sides of Indian Quad; the Life Sciences Building, the Biology Building, the Grounds Building; and Boor Sculpture Studio.

The first pipe segment in Sector VI downstream of Sector V has a flow capacity of 1.17 MGD. As discussed above, the flow capacity of Sector V, neglecting the pipe segment with zero slope, is 1.28 MGD. This means that the capacity of Sector V is limited to the downstream (Sector VI) pipe segment capacity of 1.17 MGD. Two laterals enter Sector VI, one each on the east and west sides of the Indian Quad. These laterals have estimated flow capacities of 6.12 MGD and 4.72 MGD, respectively, higher than the downstream capacity of the Sector VI interceptor. Downstream of the 1.17 MGD capacity segment, which is the most upstream pipe in Sector VI, the Sector VI pipe segments all have flow capacities greater than 1.23 MGD. The pipe segment with the flow capacity of 1.28 MGD is at the downstream end of Sector VI, after all of the laterals have joined the Sector. The flow capacity of Sector VI is therefore limited to 1.23 MGD, with no more than 1.17 MGD able to enter Sector VI from Sector V.

2.4 FLOW METERING RESULTS AND DISCUSSION

2.4.1 Summary of Flow Metering Data

Table 2-1 includes the results of the flow metering for each of the four flow metering locations. The first column is the average daily flow for May 16, 2007 and May 17, 2007 when school was still in session. The second column is the average daily flow (ADF) for the metering days when school was not in session, including the days of commencement activities. The next two columns are the maximum and minimum instantaneous flows during the entire flow metering period. The last three columns are the average, maximum, and minimum water depth ratios of the pipes during the entire metering period.

Table 2-1: Summary of Results

	ADF – In Session (MGD)	ADF – Not in Session (MGD)	Maximum Instantaneous Flow (MGD)	Minimum Instantaneous Flow (MGD)	Average Water Depth Ratio	Maximum Water Depth Ratio	Minimum Water Depth Ratio
Location 1	0.09	0.03	0.32	0.00	0.11	0.28	0.00
Location 2	0.24	0.15	0.54	0.00	0.22	0.42	0.13
Location 3	0.43	0.31	0.94	0.00	0.37	0.75	0.27
Location 4	0.03	0.02	0.08	0.00	0.10	0.19	0.00

The flows and water depth ratios are the highest at Location 3 and the lowest at Location 4. Locations 2 and 3 both have minimum instantaneous flows of 0.0 million gallons per day (MGD), but minimum water depth ratios of 0.13 and 0.27. This means that there is still water in the pipe even when there is no flow, which indicates sags or blockages in the pipe that impede it from draining fully.

Location 2 is downstream from Location 3, but has a lower ADF, a lower maximum instantaneous flow, average water depth ratio, maximum water depth ratio and minimum water depth ratio. Upon an audit of field procedures by Woodard & Curran and Savin Engineers, there did not appear to be an issue with the flow meters, calibration procedures, or data recording. The Condition Assessment determined that while there were some condition issues, the types of issues that were found (e.g., sags and roots) would not typically be expected to cause a flow loss of this magnitude. Additional flow monitoring in the future may help clear up this discrepancy.

2.4.2 Inflow and Infiltration Analysis

A strong correlation between flow and precipitation is an indicator of inflow and infiltration to the sanitary sewer system. This would be observed if the metered flow consistently increases when there is a precipitation event. Figure 2-7, below, contains a plot of the average day flow at each location and the precipitation on each day, for each of the flow metering days.

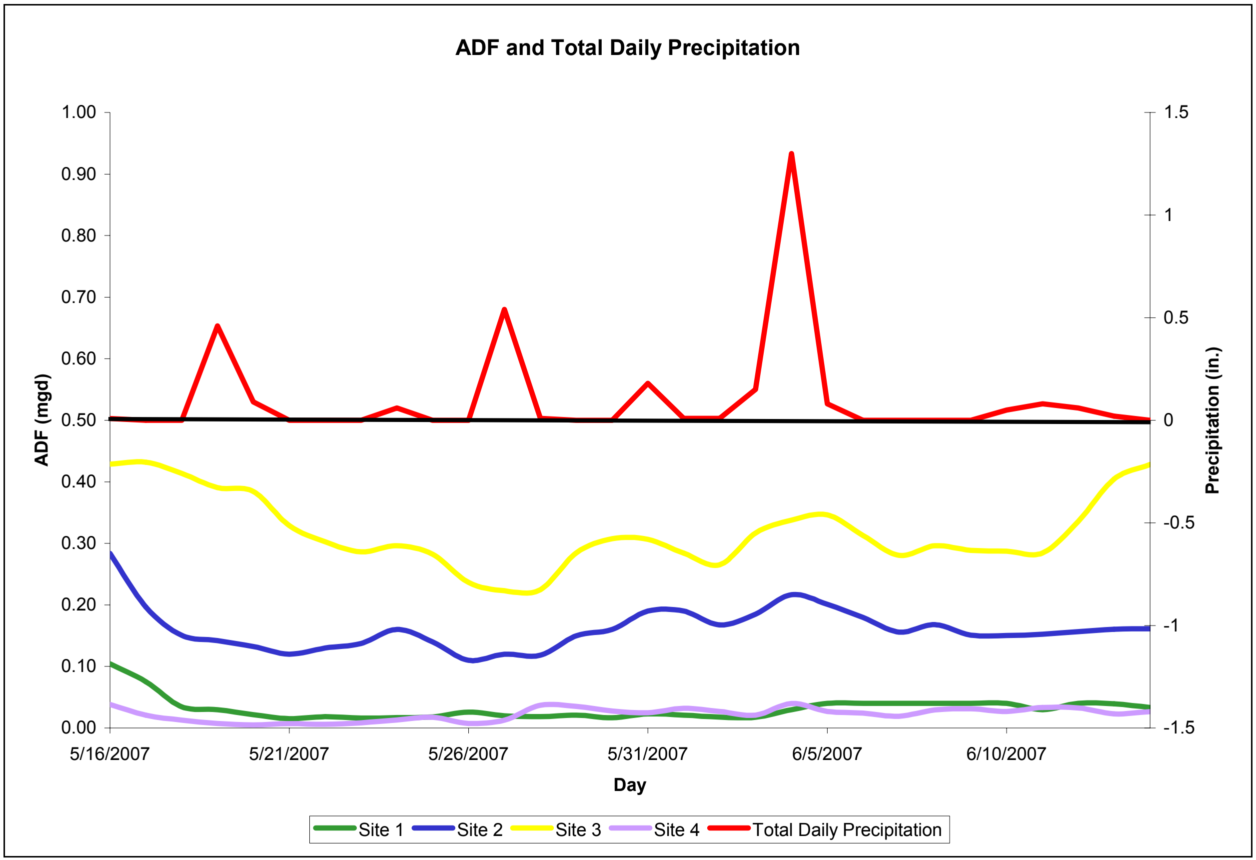


Figure 2-7: ADF and Precipitation

There is no clear visual correlation between flow and precipitation in the above figure. To further understand if there was a correlation, a statistical correlation was calculated between the metered flow and the precipitation events for each site.

This was done by graphing ADF and precipitation in a scatter plot for each location. A linear trend line was graphed on each scatter plot. The correlation coefficient (C) was calculated by taking the square root of the average variance of each data point from the trend line. The correlation coefficient is a number between negative one (-1) and positive one (1). If the flows were strongly correlated, then the magnitude of the correlation coefficient would be close to one. If the flows were independent, then the magnitude of the correlation coefficient would be close to zero. See Appendix A for plots and trend lines for each of the flow metering sites.

The correlation coefficients calculated for each of the four flow metering sites are presented in Table 2-2.

Table 2-2: Correlations Between Precipitation and Flow

Location	| C |
1	-0.13
2	0.10
3	0.02
4	0.05

There are no strong correlations between average day flow and precipitation, which indicates that there are not significant sources of infiltration or inflow to the system

2.4.3 Capacity Analysis

The flow metering data was analyzed visually and by using Manning's Equation to draw conclusions about the available flow capacity at the metered locations. As discussed in Section 2.1.1, The Manning Equation relates the water depth ratio (d/D, or the depth of flow divided by pipe diameter) to flow through the pipe.

The flow and water depth ratio (d/D) for all flow metering data for each location were graphed in scatter plots, presented and discussed in the sections below. From the flow and water depth ratio data, a best fit Manning's Curve was derived for the manhole conditions by calculating the hydraulic radius (R) and slope (S) from flow data, and using standard values for the roughness coefficient (n).

Manning's Equation calculates the maximum flow that can be conveyed through a pipe at a given slope, water depth ratio, and roughness coefficient. If the flow through a pipe is less than that predicted by Manning's Equation, other pipe conditions, such as blockages or other upstream or downstream conditions, may be impacting the flow. For this reason, the best fit Manning's Curves that were determined for each flow monitoring location followed the "leading edge" of the flow data, with an allowance for typical variations in flow meter precision.

Below is the analysis of the flow data capacity at each location. In Figures 2-6 through 2-9, the solid line in each graph is the estimated Mannings's Curve based on the best-fit of the flow metering data; the dashed line is the Manning's Curve based on the downstream pipe slope.

2.4.3.1 Location 1

The maximum water depth ratio at Location 1 was 0.28. At this water depth ratio, the flow is at approximately 17% of the full pipe capacity. Figure 2-8 is a graph of the flow metering data for Location 1 with a best-fit Manning's Curve shown to illustrate the flow pattern.

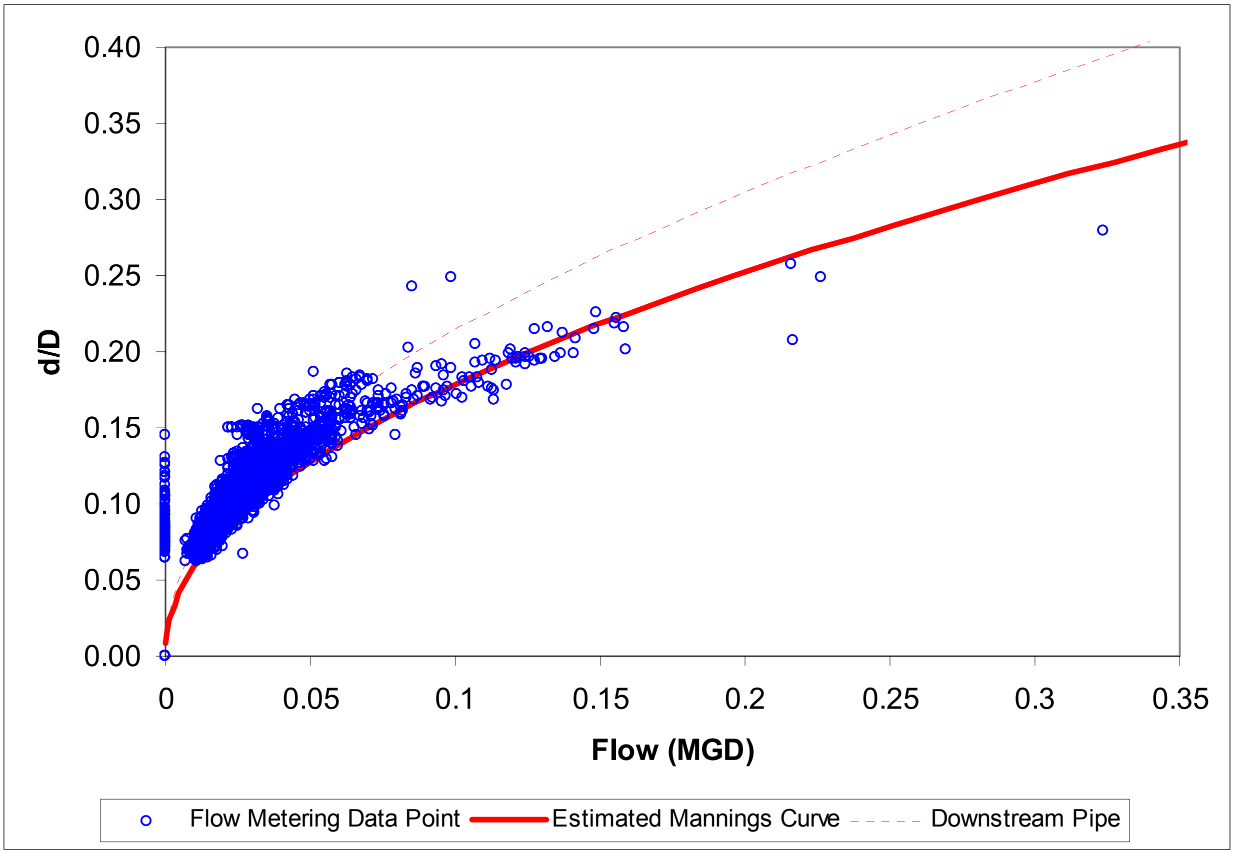


Figure 2-8: Flow vs. Water Depth Ratio at Location 1

Most of the data points lie around or just above the Manning's Curve. This means while the pipe generally conveys flow as predicted by the Manning's Equation, other conditions in the pipe may impede the flow to a certain extent. The data points that lie on the vertical axis (zero flow points) and have a water depth ratio of greater than zero indicate that there was water in the pipe even when there was no flow. This indicates issues such as sags or back-ups from downstream that may have caused water to remain in the pipe even when there was no flow. In the downstream area of Location 1, there was only one pipe inspected as part of the Condition Assessment. This pipe was determined to be in excellent condition. There is therefore the possibility of issues further downstream from the inspected pipe segment.

There were issues discovered in the camera inspection upstream of Location 1, which may also have affected the metered flow, as well as the pipe system capacity. Beginning in the Colonial Quad area, furthest upstream from MH145 (F11_sMH03), some of the issues in this set of pipes which may have affected flow metering results were fine roots, 20% sagging, 5% grease, two cracks/breaks and a pipe blockages of 20-70%. Downstream from Colonial Quad, the Collins Circle contained the following issues which may have affected flow metering results: fine roots, 15-25% sagging, 5% grease, and a pipe which was blocked by 70%. Downstream from Collins Circle, just upstream from MH145 (F11_sMH03), is the North State Quad and Softball Field which contained fine roots, sags of 25-50%, 5% grease, one crack, and pipe blockages of 20-85%.

The maximum recommended flow capacity at Location 1 occurs when the water depth ratio is equal to 90%. Since there is no flow data at a water depth ratio in this range, the maximum flow capacity was estimated using the best fit Manning's Curve derived equation for this location, and is estimated to be approximately 1.5 MGD. The presence of points with higher water depth ratios than predicted for a given flow using the Manning's Curve may mean that the flow capacity is restricted to less than 1.5 MGD.

2.4.3.2 Location 2

The maximum water depth ratio at Location 2 was 0.47. At this water depth ratio, the flow is at approximately 33% of the full pipe capacity. Figure 2-9 is a graph of the flow metering data for Location 2 with a best-fit Manning's Curve shown to illustrate the flow pattern.

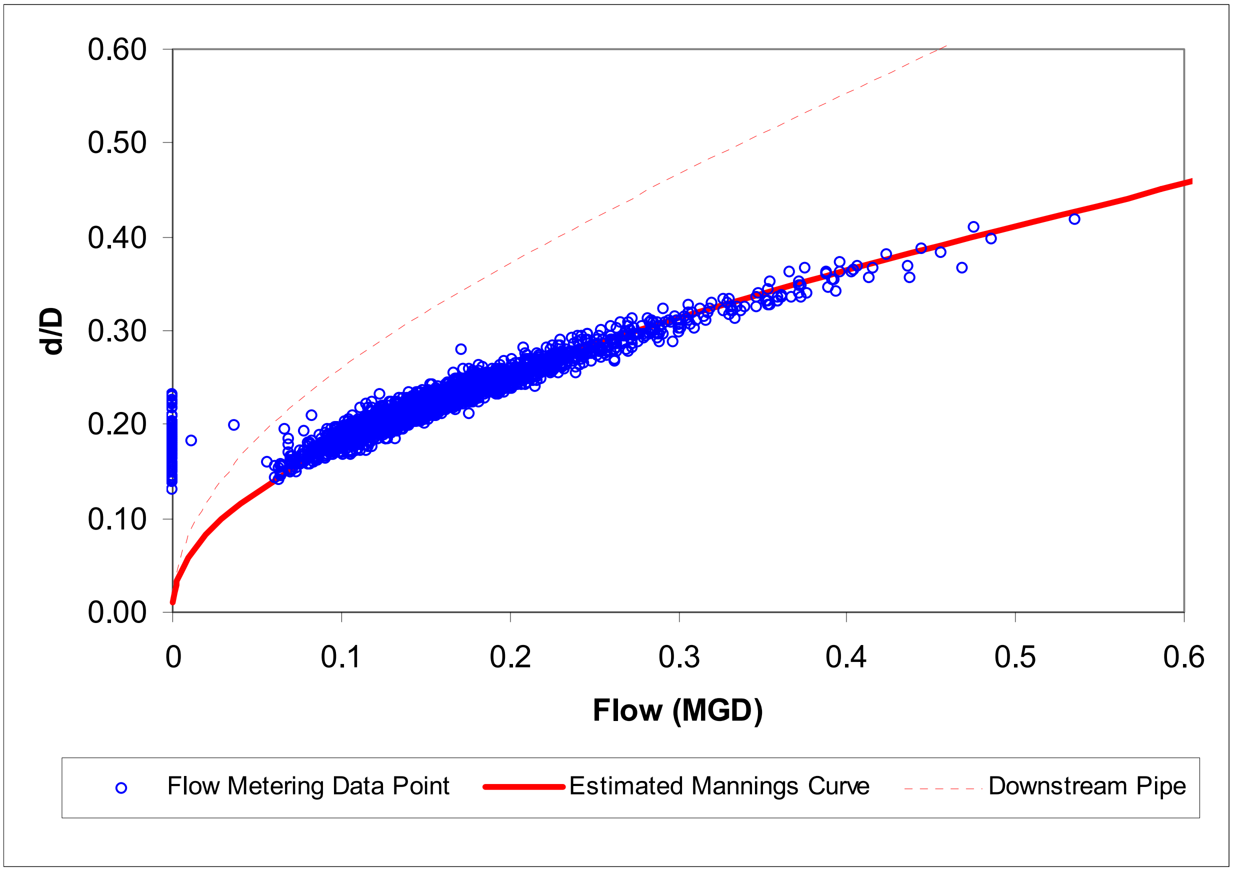


Figure 2-9: Flow vs. Water Depth Ratio at Location 2

Most of the data points fit very closely to the Manning Curve. This means that the upstream and downstream conditions allow for unimpeded flow in the range of flows observed during the metering period. The data points that lie on the vertical axis (zero flow points) and have a water depth ratio of greater than zero indicate that there was water in the pipe even when there was no flow. This indicates issues such as sags or back-ups from downstream that may have caused water to remain in the pipe even when there was no flow. In the downstream area of Location 2, there was only one pipe inspected as part of the Condition Assessment. There was one pipe downstream, east of MH103 (K10_sMH02). This pipe contained fine roots and a 45% sag, which may have been the cause of there being water in the pipe even when there was zero flow.

There were issues discovered in the camera inspection upstream of Location 2, which also may have affected the metered flow as well as the pipe system capacity. The area directly upstream of Location 2 was the Indian Quad and Justice Drive Area which included pipes with fine roots, sags of 20-45%, 20% grease, blockages of 25-90% and a crack. For more information about pipe conditions which may have affected the flow metering, see the Condition Assessment.

The maximum recommended flow capacity at Location 2 occurs when the water depth ratio is equal to 90%. Since there is no flow data at a water depth ratio in this range, the maximum flow capacity was estimated using the best fit Manning's Curve derived equation for this location, and is estimated to be approximately 1.5 MGD. The presence of points with higher water depth ratios than predicted for a given flow using the Manning's Curve may mean that the flow capacity is restricted to less than 1.5 MGD.

2.4.3.3 Location 3

The maximum water depth ratio at Location 3 was 0.75. At this depth ratio, the flow is at approximately 75% of the full pipe capacity. Figure 2-10 is a graph of the flow metering data for Location 3 with a best-fit Manning's Curve shown to illustrate the flow pattern.

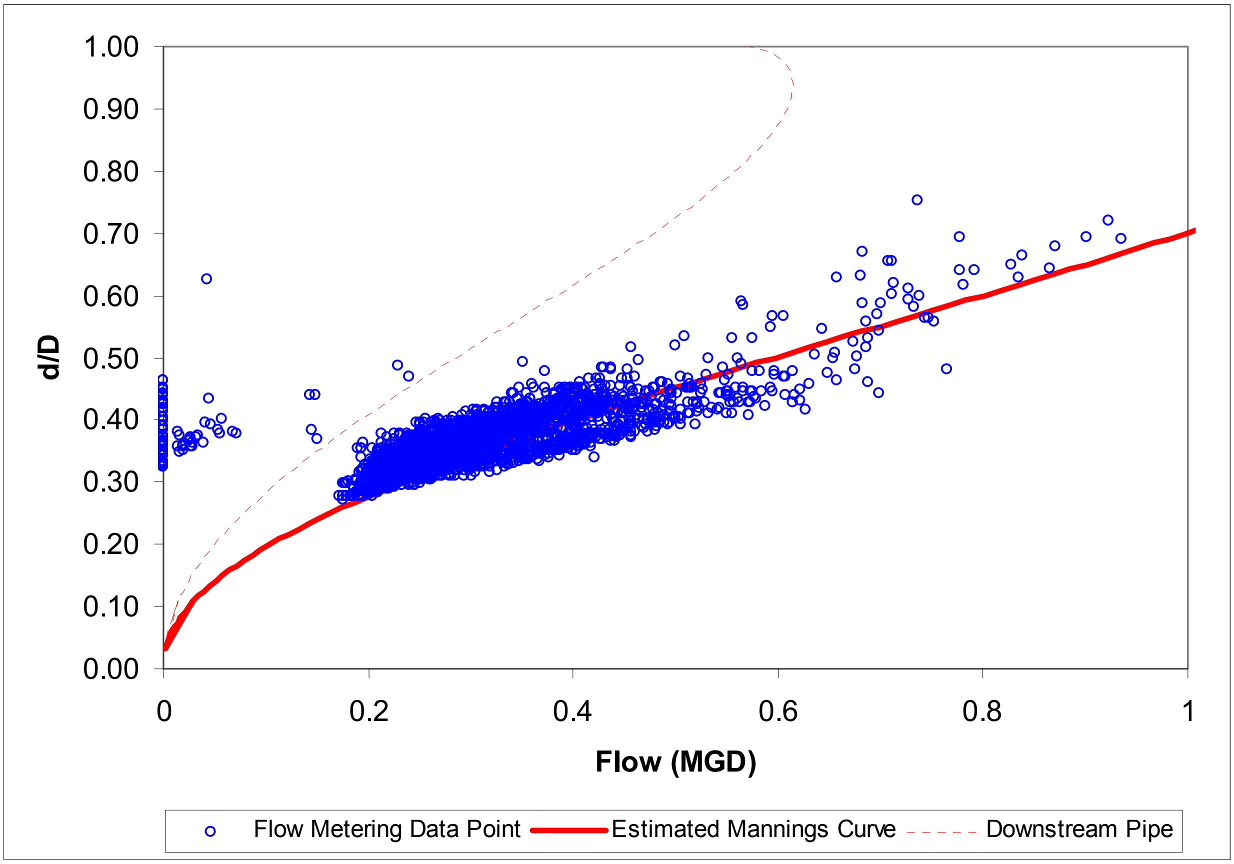


Figure 2-10: Flow vs. Water Depth Ratio at Location 3

Most of the data points are distributed around the Manning Curve. This means while the pipe generally conveys flow as predicted by the Manning's Equation, other conditions in the pipe may impede the flow to a certain extent. The data points that lie on the vertical axis (zero flow points) and have a water depth ratio of greater than zero indicate that there was water in the pipe even when there was no flow. This indicates issues such as sags or back-ups from downstream that may have caused water to remain in the pipe even when there was no flow. There is also a group of data points with flows between 0 and 0.1 which have water depth ratios of approximately 0.35-0.4 instead of 0.0-1.5 which is suggested by the Manning curve. This group of points also indicates issues such as sags or back-ups from downstream which may have caused water to remain in the pipe even when there was no flow. Downstream of Location 3 was the Indian Quad and Justice Drive Area which included pipes with fine roots, sags of 20-45%, 20% grease, blockages of 25-90% and a crack.

There were also issues discovered in the camera inspection upstream of Location 3, which may have affected the metered flow as well as the pipe system capacity. The area upstream of Location 3 is the University Field area were deemed to be in poor condition for up to 40% blockages, 35% grease, up to 60% sags, and fine roots. For more information on pipe conditions, see the Condition Assessment.

The maximum recommended flow capacity at Location 3 occurs when the water depth ratio is equal to 90%. Since there is no flow data at a water depth ratio in this range, the maximum flow capacity was estimated using the best fit Manning's Curve derived equation for this location, and is estimated to be approximately 1.3 MGD. The presence of points with higher water depth ratios than predicted for a given flow using the Manning's Curve may mean that the flow capacity is restricted to less than 1.3 MGD.

2.4.3.4 Location 4

The maximum water depth ratio at Location 2 was 0.19. At this water depth ratio, the flow is at approximately 8% of the full pipe capacity. Figure 2-11 is a graph of the flow metering data for Location 4 with a best-fit Manning's curve shown to illustrate the flow pattern.

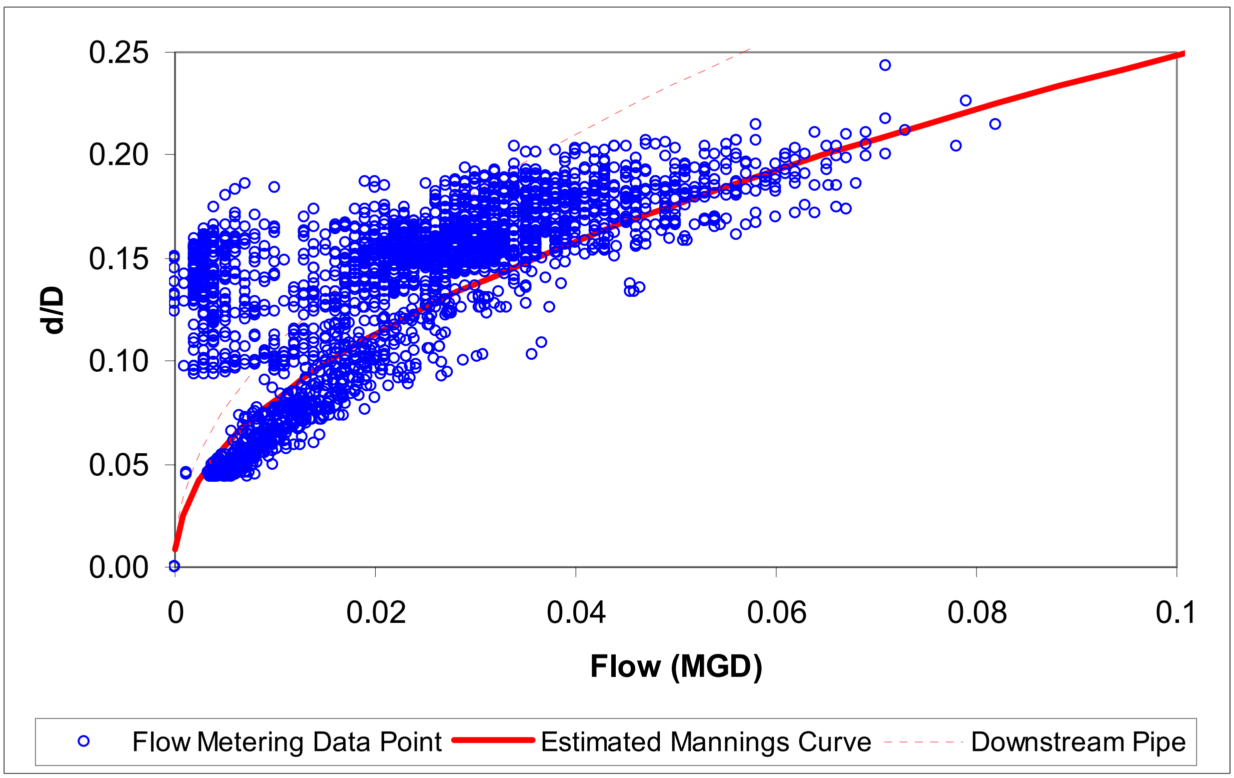


Figure 2-11: Flow vs. Water Depth Ratio at Location 4

Most of the data points are distributed mostly above and to the left of the Manning Curve, with a cluster of data points with low flow and high water depth ratio. The data points that lie on the vertical axis (zero flow points) and have a water depth ratio of greater than zero indicate that there was water in the pipe even when there was no flow. This indicates issues such as sags or back-ups from downstream that may have caused water to remain in the pipe even when there was no flow. There is also a group of data points with flows between 0 and 0.1 MGD that have water depth ratios of 0.07-0.15 instead of the 0.0-0.06 that is predicted by the Manning's Equation. This indicates issues such as sags or back-ups from downstream which may have caused low flow despite the amount of water in the pipe. There were issues discovered in the camera inspection downstream of Location 4, which may have affected the metered flow as well as the pipe system capacity. Beginning in the furthest upstream area, the Dutch Quad area had pipes with fine roots, sags of 15-35%, 10% grease, and three cracks/breaks. There were no pipes inspected upstream of MH46 (G06_sMH06), so issues upstream which may have also impacted the flow metering results are unknown. For more information on the condition of the pipes, see the Condition Assessment.

The maximum recommended flow capacity at Location 4 occurs when the water depth ratio is equal to 90%. Since there is no flow data at a water depth ratio in this range, the maximum flow capacity was estimated using the best fit Manning's Curve derived equation for this location, and is estimated to be approximately 1.4 MGD. The presence of points with higher water depth ratios than predicted for a given flow using the Manning's Curve may mean that the flow capacity is restricted to less than 1.4 MGD.

2.5 CONCLUSIONS

2.5.1 Northern Interceptor

2.5.1.1 Sector I

The estimated maximum flow capacity of Sector I based on the capacity model is 0.47 MGD. The flow from this area is not constrained by the downstream sector. Based on the condition assessment, the pipes in this area are in poor condition due to cracks, blockages up to 70%, sags up to 25% and grease. As a result, the estimated current flow capacity of Sector I is less than 0.47 MGD.

2.5.1.2 Sector II

The estimated maximum flow capacity of Sector II based on the capacity model is approximately 1.32 MGD. The flow from this area is not constrained by the downstream sector. Based on the condition assessment, the pipes in this area are in poor condition due to blockages up to 60%, sags up to 50% and grease. As a result, the estimated current flow capacity of Sector II is less than 1.32 MGD.

2.5.1.3 Sector III

The estimated flow capacity of Sector III based on the capacity model is approximately 1.62 MGD. Based on the condition assessment, the pipes in this area are in poor condition due to cracks, blockages up to 85%, and sags up to 30%. The condition would most likely reduce the flow capacity of all of the pipes.

Flow metering data was taken at a downstream location of this sector. The flow metering data indicated that the maximum instantaneous flow over the flow metering period was 0.082 MGD, which is significantly less than the Sector III modeled maximum flow capacity of 1.62 MGD, as well as the upstream sectors. The maximum water depth ratio in this pipe was less than 20% indicating that this pipe has capacity available in its current condition.

As a result, the flow capacity of Sector III is currently less than 1.62 MGD. Since maximum observed flow was 0.082 MGD, there would be approximately 1.54 MGD of capacity remaining in Sector III if the pipes were rehabilitated to be in good condition.

2.5.1.4 Overall Northern Interceptor

The northern interceptor has additional flow capacity available. Based on flow metering data, the maximum observed instantaneous flow rate through the interceptor with the current poor condition of pipes was 0.082 MGD and the average daily flow was 0.03 MGD, which are both significantly less than modeled flow capacities for Sectors I-III (0.47 – 1.62 MGD.) The capacity model assumed that the pipes were in good condition. Since the pipes in the northern interceptor were in poor condition, the actual current flow capacity is most likely less than the modeled capacity.

2.5.2 Southern Interceptor

2.5.2.1 Sector IV

The estimated maximum flow capacity of Sector II based on the capacity model is approximately 0.38 MGD. The flow from this area is not constrained by the downstream sector. Based on the Condition Assessment, the pipes in this area are on average in fair condition. Certain pipes are in good condition, and certain pipes are in poor condition due to blockages up to 90% and sags up to 75%. Therefore, the flow capacity of Sector IV is currently less than 0.38 MGD.

2.5.2.2 Sector V

The estimated maximum flow capacity of Sector V based on the capacity model is approximately 1.17 MGD, based on the limiting downstream segment in Sector VI. The Condition Assessment activities determined that the pipes in this area are on average in fair condition. Certain pipes are in good condition and certain pipes are in poor condition due to cracks, sags up to 30% and grease. Therefore, the flow capacity of Sector V is currently less than 1.17 MGD.

Flow metering data was taken for the lateral coming into Sector V. The data indicated that the maximum instantaneous flow for the system was 0.32 MGD. This flow is less than the maximum estimated flow capacity of the section of interceptor the lateral ties into. This flow in addition to the maximum upstream flow capacity (0.38 MGD from Sector IV) is still less than the estimated flow capacity of Sector V, 1.17 MGD. Therefore, there is additional capacity in this sector of pipe.

2.5.2.3 Sector VI

The estimated maximum flow capacity of Sector VI based on the capacity model is approximately 1.30 MGD. Based on the Condition Assessment, the pipes in this area are on average in fair condition. Certain pipes are in good condition, and certain pipes are in poor condition due to cracks, blockages up to 90%, sags up to 60%, and grease blockages of up to 35%. As a result, the flow capacity of Sector VI is currently less than 1.30 MGD.

Flow metering data was taken at two locations in Sector VI, one location south of Indian Quad, and one at the downstream end of the southern interceptor. The location south of Indian Quad had the largest observed flows. The maximum instantaneous flow was 0.94 MGD. This flow is less than the estimated maximum flow capacity of this area, 1.30 MGD. The maximum observed water depth ratio was 0.75. The maximum recommended water depth ratio is 0.90, so the current capacity is approaching the maximum recommended value.

The flow metering location at the end of the southern interceptor, Location 2, had a maximum instantaneous flow of 0.54 MGD. This number is less than half of the estimated flow capacity of Sector VI, 1.30 MGD. The maximum observed instantaneous flow measured upstream at Location 3 was 0.94 MGD, which is also less than the estimated flow capacity of Sector VI. The maximum water depth ratio at Location 2 was 0.42, which is less than the maximum recommended value of 0.90. The maximum water depth ratio of the southern interceptor was 0.75 at Location 3, which is close to the maximum recommended value of 0.90. This section of Sector VI has some additional capacity in its current condition.

2.5.2.4 Overall Southern Interceptor

The southern interceptor has limited additional available flow capacity. The maximum instantaneous water depth ratio for the southern interceptor, measured at Location 3 south of Indian Quad, was 0.75, which is approaching the maximum recommended value of 0.90.

Flow metering data indicated that there were lower flows at flow metering Location 2, at the end of the southern interceptor, than at the upstream Location 3. The maximum observed instantaneous flow at Location 3 was 0.94 MGD whereas the downstream observed maximum instantaneous flow at Location 2 was 0.54 MGD. Flow meter calibration and installation data was checked to verify that flow meters were functioning correctly. The Condition Assessment indicated that there were issues such as root blockages and sags between the two locations. However, this would not account for the difference in flows that were metered between the two locations. We recommend that flow metering continues as a regular part of inspection and maintenance work to clarify this issue.

There is no additional estimated flow capacity in the Southern Interceptor because of pipe blockages of up to 90% in Sectors IV and VI.

2.5.3 Capacity Impacts of Proposed Expansion Projects

A list of planned campus construction projects was obtained by the University at Albany Office of Campus Planning. These projects were grouped into years by their estimated start date. The list is summarized in Table 2-3.

Table 2-3: Planned Campus Construction Projects

Year	Project
Year 1 – 2008/2009	Pond Enhancement
Year 2 – 2009/2010	State Quad Renovation
	State Quad Parking
	Renovate Health Center
	Service Building A Renovation
	Grounds Building
Year 3 – 2010/2011	School of Business
	Campus Center Addition
	Campus Center Renovation
	Dutch Quad Renovation
	Water Tower and Foundation Renovation
	Student Housing
	Connector Road
	Multi-use Stadium
Year 4 – 2011/2012	Relocate Data Center
	Library Renovation
	Science Surge Building
Year 5 – 2012/2013	Parking Structure
	Fine Arts Studio
	Purple Path Phase 2
	Northern Landscape Improvement
	Southern Landscape Improvement
	Construct Storage Structure

Since there is no additional estimated flow capacity in the southern interceptor, capacity should be increased before the new expansion projects on this interceptor are completed. Expansion projects on the southern interceptor include the new Data Center, Stadium, Science Surge, Student Housing and Fine Arts Studio.

There is additional estimated flow capacity in the Northern Interceptor. The Northern Interceptor should be able to accommodate the additional flows of the new Business Building. The other projects above are not expected to add any wastewater flow.

2.5.4 Additional Discussion

During analysis, we found a correlation between the pipe slopes and the condition of pipes. The pipes with zero slope in the capacity model were all in poor condition. Pipes with very small slopes were generally also in fair or poor condition. Most of these pipes had large sags in them. If pipes in poor condition are rehabilitated, we recommend that they are designed with a minimum flow velocity of 2 ft/s, in accordance with Ten State Standards for Wastewater, 2004, and NYSDEC Design Standards for Wastewater Treatment Works, 1988.

3. STORMWATER COLLECTION SYSTEM

3.1 METHOD

The capacity of the campus stormwater collection system was assessed by developing a model of the system SewerGEMS®. The model was used to estimate the quantity and hydraulic grade line of the flow throughout the system. The model developed using this software is a simplified representation of the stormwater sewer system at the campus. The model includes areas of the campus, also known as subbasins, where stormwater collects and flows into the stormwater collection system. Additional system components included in the model are catch basins, manholes, conduits, building drains, and outlets. In the model, stormwater flows from subbasins into catch basins, and stormwater from building roofs and courtyards flows into the system through building drains. Building drains tie directly into the system at catch basins and manholes. Given these components, the software estimates the stormwater generated during a rainfall event and the quantity and hydraulic grade line of flow through the system.

3.1.1 Hydrologic Modeling

Given a rainfall event, SewerGEMs® modeling software estimates the quantity of stormwater collected in a subbasin using Soil Conservation Services (SCS) methodology. This methodology assigns a curve number to each subbasin. This curve number can be directly applied to estimate the quantity of stormwater collected in each subbasin, and therefore the quantity of stormwater flowing into the stormwater collection system.

The curve number of a subbasin is dependent primarily upon both the land cover type and the Hydrologic Soil Group, more specifically the soil type, of the subbasin. These two factors determine the volume of stormwater which infiltrates into the soil and the volume of water which collects on the subbasin and flows into the stormwater collection system.

Soil types are classified into four Hydrologic Soil Groups. Soils classified under Hydrologic Group A have a low runoff potential and high infiltration rates, whereas soils classified under Hydrologic Group D have a high runoff potential and very low infiltration rates. As a result, subbasins with soils classified as Hydrologic Group A have a lower curve number than those with soils classified as Hydrologic Group D. In addition, subbasins with pervious land covers have a lower curve number than those with impervious land covers. As a result, for subbasins with a larger curve number, a greater amount of stormwater flows from the subbasin into the catch basins.

The Hydrologic Soil Groups of the soils at the campus were assigned using Natural Resources Conservation Service (NRCS) mapping. For areas of the campus not classified as Urban Land or Water, the soils are comprised primarily of loamy fine sands. It is estimated that approximately 86% of the soil at the campus is classified as Hydrologic Soil Group A, 8% as Hydrologic Soil Group B, 5% as Hydrologic Soil Group C, and 1% as Hydrologic Soil Group D.

For simplicity, areas of the campus were classified under three land cover types: pervious, impervious, and forested. Areas of the campus classified as pervious included athletic fields and lawns. Areas classified as impervious included paved and gravel parking areas, roads, sidewalks, tennis courts, and building roofs. Areas classified as forested included those containing shrubbery and woods.

Curve numbers were obtained from the SCS Technical Report 55 (TR-55). For pervious areas, curve numbers for open-space in fair condition (areas with grass cover between 50 to 75 percent) were assumed. Curve numbers with this type of land cover range from 49 to 84 depending on the Hydrologic Soil Group of the soils. For impervious areas, curve numbers of 98 were assumed. Curve numbers for impervious areas are not dependent upon the Hydrologic Soil Group of the soils. For forested areas, the average of the curve numbers for woods in fair condition and brush in fair condition were assumed. Depending upon the Hydrologic Soil Group of the soils, these values ranged from 36 to 78.

For simplicity, curve numbers for pervious and forested areas were assumed independent of the Hydrologic Soil Group due to the small variation amongst the soil across the campus. Instead, weighted curve numbers were calculated using the total areas of each of the four Hydrologic Soil Groups present at the campus. For pervious areas, a curve number of 53 was assumed, and for forested areas, a curve number of 40 was assumed. Since 86% of the soil is classified as Hydrologic Soil Group A, the weighted curve numbers are at the lower end of the ranges previously discussed. The curve number of each subbasin was estimated by calculating the total areas of the pervious, impervious, and forested land types. Using these areas, a weighted curve number was calculated and applied to the subbasin.

As discussed previously, the curve number is applied to estimate the quantity of stormwater collected in a subbasin for a rainfall event. The quantity of stormwater flowing from a subbasin into a catch basin as a function of time is defined as a hydrograph. As a result, a hydrograph is dependent upon the rainfall event distribution, total rainfall depth during the event, and the time of concentration. The hydrograph can be directly applied to determine the peak volumetric rate of stormwater flowing from a subbasin into a catch basin.

An SCS synthetic rainfall distribution was used to model the rainfall events. For Albany, New York, rainfall events follow a Type II distribution. The rainfall events used to access the capacity of the stormwater collection system were the 2-, 5-, 10-, and 25-year 24-hour rainfall events. For Albany, New York, total rainfall for the 2-, 5-, 10-, and 25-year 24-hour rainfall events are estimated to be 2.8, 3.7, 4.2, and 5.0 inches respectively. These rainfall depths were obtained from SCS TR-55.

The time of concentration of a subbasin is the time at which all points in the subbasin contribute runoff to the catch basin. The time of concentration of each subbasin was determined by estimating the flow path for the point hydraulically furthest from the outlet, or the flow path with the greatest time of travel. Flow paths were estimated using topography and surface conditions obtained from a fly-over survey of the campus. The SCS TR-55 sheet flow and shallow concentrated flow models for time of concentration were then applied to determine the values for times of concentration of each subbasin. For the first 100 feet, sheet flow was assumed. Thereafter, shallow concentrated flow was assumed for the remainder of the flow path.

Following SCS TR-55 methodology, the minimum time of concentration that can be used is 0.1 hour. To be conservative, this value was assumed for areas draining from buildings, such as building roofs, since flow paths could not be determined.

3.1.2 Hydraulic Modeling

SewerGEMs modeling software estimates the quantity of flow through conduits using Manning's equation. Manning's equation can be directly applied to free-surface flows. For pressure flows, however, the Preissmann slot method was applied thus allowing Manning's equation to be applied to pressure flow. For simplification, the Manning's roughness coefficient "n" value for concrete conduits, 0.013, was assumed for all conduits in the model.

Energy losses through manholes and catch basins were modeled using the energy equation. For simplification, a loss coefficient of 0.65 was assumed for all manholes and catch basins in the model. This loss coefficient is applied to junctions, such as manholes and catch basins, with a bend between 45- and 90-degress.

The stormwater collection system consists of 14 subsystems, each subsystem defined by its outlet, or ultimate discharge point. Some subsystems discharge to detention ponds, as others discharge to infrastructure located off campus. The boundary condition, or water surface elevation, at each outlet was assumed in the model. For subsystems discharging into detention ponds, a free outlet was assumed. For subsystems discharging into infrastructure located outside campus boundaries, the outlet was assumed to be flowing full. The subsystems are discussed in further detail in Section 3.1.3.

3.1.3 Subsystems

Each of the fourteen stormwater collection subsystems consists of subbasins, catch basins, manholes, conduits, and outlets unique to each subsystem, and is defined by its outlet, or ultimate discharge point. Some subsystems discharge to detention ponds, as others discharge to infrastructure located off campus on Western Avenue, Fuller Road, or Washington Avenue. Figure 3-1 provides the general locations of these subsystems, and the blue lines in this figure represent conduits. Descriptions of these subsystems are provided in subsequent sections.

Figure 3-1: Locations of Subsystems I – XIV

3.1.3.1 Subsystem I

Subsystem I is located west of Fuller Road and encompasses Freedom Quadrangle and Freedom Gold Lots A and C. Included in this subsystem, are student housing buildings and Jose Marti Drive. The outlet of this subsystem is a wetlands area located south of Freedom Quadrangle. Subsystem I is approximately four acres, half of which is impervious while the other half being pervious or forested. This subsystem consists of 17 subbasins.

3.1.3.2 Subsystem II

The Subsystem II basin is located west of Fuller Road, just north of the Subsystem I basin, and encompasses buildings and parking areas north of Tri-Centennial Drive. The Subsystem II basin is approximately eight acres, five of which are impervious, while the remaining three acres consist of pervious or forested areas. Subsystem II basin is comprised of 16 subbasins. This subsystem discharges to a detention pond located north of Tri-Centennial Drive.

3.1.3.3 Subsystem III

Subsystem III is located on the northwestern part of campus east of Fuller Road. This subsystem is north of Subsystem IV and comprises of the northern part of Empire Commons, including the North and West Gold Lot, Excelsior Drive, and the northern part of Capital Hill. Subsystem III is approximately seven acres, and of these seven acres, roughly five are impervious and two are pervious. Subsystem III is comprised of 16 subbasins, and this subsystem discharges to a detention pond located west of the West Gold Lot.

3.1.3.4 Subsystem IV

Subsystem IV is located on the northwestern part of campus east of Fuller Road, south of the Subsystem III. The southern part of Empire Commons, including the South Gold Lot, Liberty Lane, and the southern part of Capital Hill, is serviced by this subsystem. Subsystem IV consists of 17 subbasins and encompasses approximately seven acres, four of which are impervious while the remaining three acres are pervious or forested. This subsystem discharges to a detention pond located northeast of the intersection of Tri-Centennial Drive and Fuller Road.

3.1.3.5 Subsystem V

Subsystem V is located west of Collins Circle and consists of 50 subbasins. The Northwest Gold Lot, Colonial Quadrangle, and parts of the Colonial Gold and Purple Lots, Academic Podium, and Collins Circle are within this subsystem. This subsystem discharges to a detention pond located north of the Northwest Gold Lot and is estimated to be approximately 37 acres. Of these 37 acres, 19 acres consist of impervious areas, and the remaining 18 acres are pervious or forested.

3.1.3.6 Subsystem VI

Subsystem VI discharges to the lake east of the lacrosse and hockey fields. This subsystem encompasses part of the Academic Podium, the Health and Counseling Building, Colonial Gold Lot – C, the Podium West and Dutch Purple Lots, the Dutch Quadrangle, the northern part of the Dutch Gold Lot, as well as University Field and the Physical Education building. This subsystem consists of 93 subbasins and approximately 55 acres, 27 of which are impervious, while the remaining 28 are pervious or forested.

3.1.3.7 Subsystem VII

Subsystem VII is located on the southwestern part of campus, and includes the Power Plant and surrounding buildings and lots. This subsystem consists of approximately 2.5 acres, 1.5 acres which is impervious while the remaining one acre is pervious. Subsystem VII is composed of five subbasins, and this subsystem discharges to infrastructure on Fuller Road.

3.1.3.8 Subsystem VIII

Subsystem VIII consists of approximately 61 acres, 26 of which are impervious, while the remaining 35 are pervious or forested. This subsystem is located east of Collins Circle, and encompasses the State Quadrangle, State Gold and Purple Lots, State Drive, and parts Collins Circle, the Academic Podium, and Carillon Drive. Subsystem VIII consists of 70 subbasins and discharges to infrastructure on Washington Avenue.

3.1.3.9 Subsystem IX

Subsystem IX is generally located in the center of campus. The Dutch Field, Indian Quadrangle, and parts of the Academic Podium fall within this subsystem. This subsystem consists of approximately 32 acres and 21 subbasins. Of the 32 acres, 23 are impervious. The remaining nine acres are pervious or forested. Subsystem IX discharges to the lake east of the lacrosse and hockey fields.

3.1.3.10 Subsystem X

Subsystem X discharges to the lake on the southeastern part of campus, east of the lacrosse and hockey fields. Subsystem X is located on the southeastern part of campus, and encompasses the SEFCU Arena, the majority of Dutch Gold Lot, as well as the Dutch Tennis Courts. Subsystem X is composed of 61 subbasins and 42 acres, about half this area is impervious, while the remaining half is pervious or forested.

3.1.3.11 Subsystem XI

Subsystem XI is located on the eastern part of campus, and encompasses the Life Sciences building, the Visitors Parking Lot-P2, and parts of the Indian Quadrangle. Subsystem XI consists of ten acres of land, six of which are impervious, while the remaining four are pervious or forested. Subsystem XI composes of 27 subbasins and discharges to the lake on the southeastern part of campus, east of the lacrosse and hockey fields.

3.1.3.12 Subsystem XII

Subsystem XII is located on the eastern part of campus and includes the area surrounding the building east of the University Police building and parts of Justice Drive. This subsystem consists of ten acres, two of which are comprised of impervious areas, and eight of which are pervious or forested. Subsystem XII consists of eight subbasins. This subsystem discharges to the lake on the southeastern part of campus, east of the lacrosse and hockey fields.

3.1.3.13 Subsystem XIII

Subsystem XIII is located on the eastern part of campus and includes the University Police Station building and the Boor Sculpture Studio. This subsystem consists of eight acres of land. Of these eight acres, three acres are impervious and the remaining five are pervious or forested. This subsystem consists of 20 subbasins, and discharges to the lake on the southeastern part of campus, east of the lacrosse and hockey fields.

3.1.3.14 Subsystem XIV

Subsystem XIV is located on the southern end of campus. The SEFCU Arena Gold Lot and the Athletic Practice Fields are within this subsystem. Subsystem XIV consists of 52 acres of land, 9 of which are impervious while the remaining 43 are pervious or forested. This subsystem consists of 61 subbasins and discharges to infrastructure on Western Avenue.

3.1.4 Recent Rainfall Data

Table 3-1 is provided as a reference to summarize the cumulative rainfall amounts of 24-hour rainfall events that Albany, New York has experienced within the past ten years. The table was created based on data obtained through the National Oceanic and Atmospheric Administration (NOAA).

Over the past ten years, there were five 24-hour rainfall events with cumulative rainfall amounts between 2.8 and 3.7 inches. There was one 100-year rainfall event in the past ten years, in September of 1999, when Albany received 6 inches of rain in a 24-hour period.

Table 3-1: 10 Years of Rainfall Data - Albany, New York

Number of 24-Hour Rainfall Events					
Total Rainfall:	**2.8" – 3.7" 2-5 yr Event**	**3.7" - 4.2" 5-10 yr**	**4.2" - 5.0" 10-25 yr**	**5.0" - 5.5" 25-50 yr**	**5.5" - 6.0" 50-100 yr**
thru 6/2008	-	-	-	-	-
2007	**1** (2.78")	-	-	-	-
2006	-	-	-	-	-
2005	-	-	-	-	-
2004	**1** (2.78")	-	-	-	-
2003	-	-	-	-	-
2002	-	-	-	-	-
2001	-	-	-	-	-
2000	**3** (2.91", 3.24", 3.37")	-	-	-	-
1999	-	-	-	-	**1** (6.0")
1998	-	-	-	-	-

3.2 CAPACITY MODEL RESULTS

As discussed previously, the SewerGEMs software estimates the hydraulic grade line elevations of the flow through the stormwater collection system during a simulated rainfall event. The software estimated these elevations assuming the stormwater collection system is free of debris and sediment.

The estimated hydraulic grade line elevation at a catch basin or manhole represents the water surface elevation (WSE) of the flow as it passes through each of these structures. Once the SewerGEMs software has simulated the rainfall event, the software reports the maximum water surface elevation (MWSE) in each catch basin and manhole throughout the duration of the rainfall event. The MWSEs reported as being greater than the rim elevation of the structure may be indicative of overflow, and possible flooding, in the general area of the structure. As a result, catch basins and manholes with estimated MWSE within two feet of the rim elevation are discussed in this section since these elevations may be indicative of overflow and possible flooding. A value of two feet was chosen since each rim elevation was calculated by interpolating between two one-foot contour lines obtained from the topographic mapping of the campus. Moreover, a value of two feet identifies structures which have the potential to overflow during a rainfall event.

Also discussed previously, the rainfall events simulated to evaluate the capacity of the stormwater collection system are the 2-, 5-, 10-, and 25-year, 24-hour rainfall events, and results from simulating each of these events will be discussed. It should be noted that a 2-year, 24-hour rainfall event has a greater likelihood of occurring annually than a 25-year, 24-hour rainfall event. Consequently, if MWSEs within a structure are within two feet of the rim for a 2-

year, 24-hour rainfall event, then this structure has a greater likelihood for overflow. As a result, if the MWSE in a structure is within two feet of the rim for a 2-year, 24-hour rainfall event, it is likely that the MWSE is greater for the 5-, 10-, and 25-year, 24-hour rainfall events.

3.2.1 Subsystem I

As discussed previously, Subsystem I includes Freedom Quadrangle and Freedom Gold Lots A and C. For the 2-, 5-, 10-, and 25-year, 24-hour rainfall events, MWSE in manholes and catch basins of this subsystem are not estimated as being within two feet of the rim elevation. As a result, overflow has not been estimated to occur in the areas serviced by Subsystem I.

3.2.2 Subsystem II

Stormwater from areas west of Fuller Road and north of Tri-Centennial Drive flows into Subsystem II. For the 2-, 5-, 10-, and 25-year, 24-hour rainfall events, MWSE in manholes and catch basins of this subsystem are not estimated to be within two feet of the rim elevation. As a result, overflow has not been estimated to occur in the areas serviced by this subsystem.

3.2.3 Subsystem III

Subsystem III services the northern part of Empire Commons, including the North and West Gold Lot, Excelsior Drive, and the northern part of Capital Hill. For the 2-, 5-, and 10-year, 24-hour rainfall events, MWSE in manholes and catch basins are not estimated as being within two feet of the approximate rim elevation.

For the 25-year, 24-hour rainfall event, the MWSE in a catch basin located in the North Gold Lot is estimated to be within two feet of the rim elevation. This catch basin is identified in red in Figure 3-2. Should the MWSE exceed the rim elevation, based on the topography it appears that overflow from this catch basin should flow to a sump surrounding this catch basin.

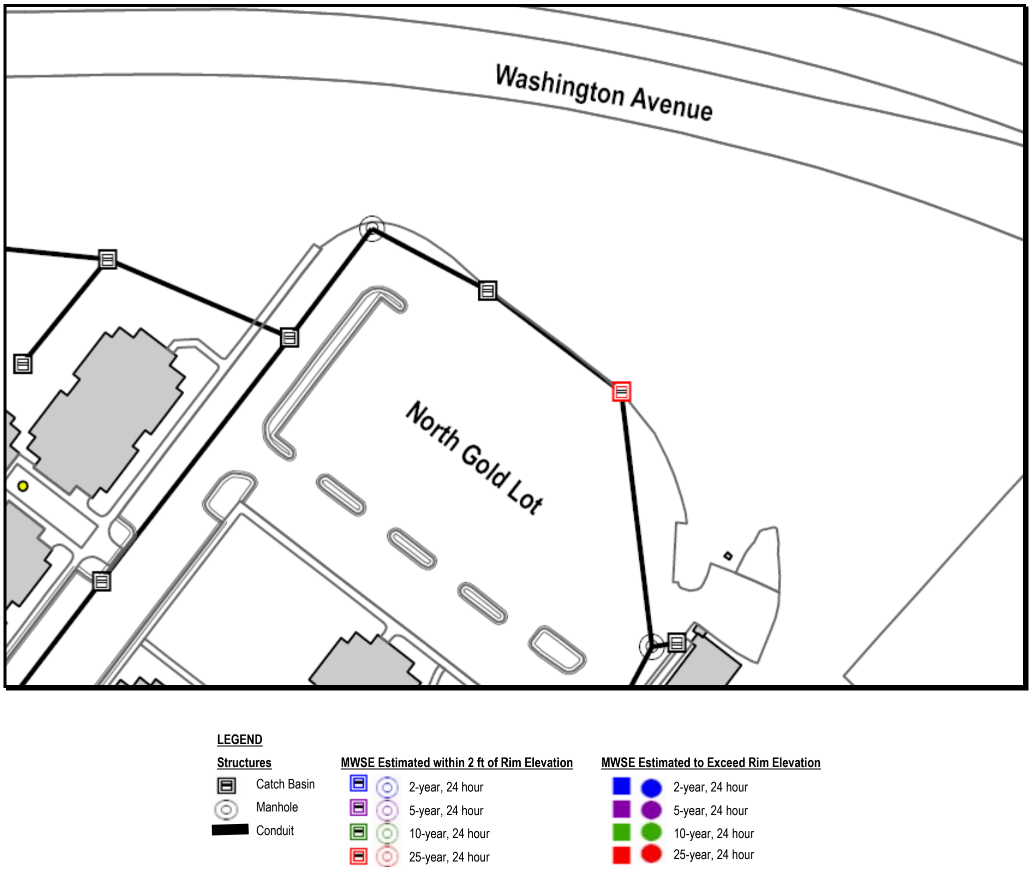


Figure 3-2: Subsystem III – North Gold Lot

3.2.4 Subsystem IV

The southern part of Empire Commons, including the South Gold Lot, Liberty Lane, and the southern part of Capital Hill, is serviced by Subsystem IV. For a portion of this subsystem, stormwater is conveyed through perforated high-density polyethylene (HDPE) conduits. Perforated conduits allow stormwater to discharge from the conduit into the surrounding soil. If the water table is above the inverts of the conduits, stormwater may not discharge from the conduits and groundwater may flow into the perforated conduits. SewerGEMs software does not have the capabilities to model perforated conduits, and as a result, conduits in this subsystem were modeled as non-perforated. Assuming the conduits to be non-perforated implies groundwater does not flow from the surrounding soil into the conduit and also stormwater does not discharge from the conduit into the surrounding soil.

Assuming the perforated HDPE conduits function properly and the water table is below the conduits, MWSE in manholes and catch basins of this subsystem are not estimated to be within two feet of the rim elevation for the 2-, 5, 10-, and 25-year, 24-hour rainfall events. As a result, overflow has not been estimated to occur in the areas serviced by this subsystem.

3.2.5 Subsystem V

Stormwater from the Northwest Gold Lot, Colonial Quadrangle, and parts of the Colonial Gold and Purple Lots, Academic Podium, and Collins Circle flows into Subsystem V, and Figure 3-3 identifies catch basins and manholes of this subsystem which MWSE are within two feet of the rim elevations for the 2-, 5-, 10-, and 25-year, 24 hour rainfall events. Discussion of these structures follows this figure.

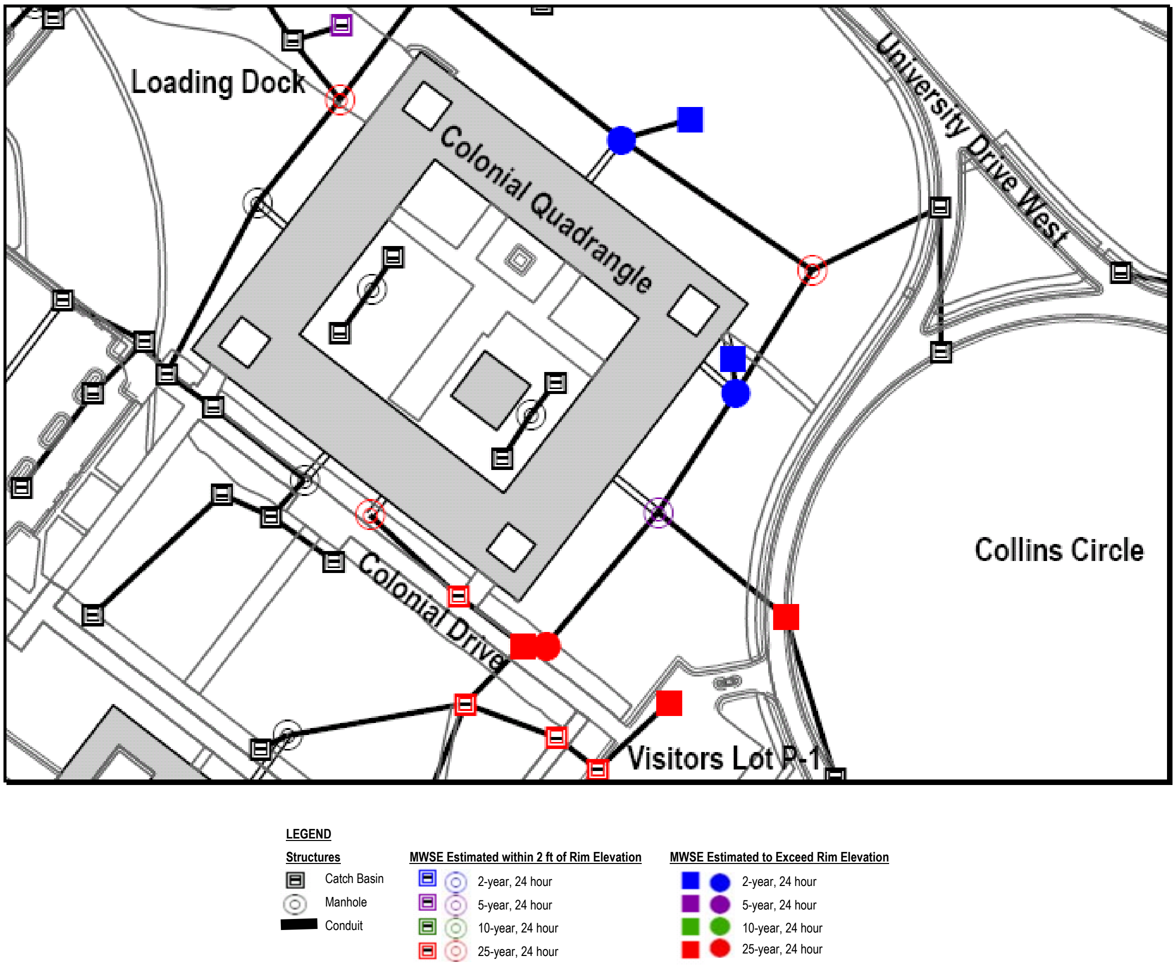


Figure 3-3: Subsystem V – Colonial Quadrangle

Maximum water surface elevations in the catch basin and manholes located in the grassy, open area north of Colonial Quadrangle are estimated as being greater than the rim elevation for the 2-year, 24-hour rainfall event. These structures are identified as solid blue in Figure 3-3. Based on the topography, overflow from these structures should flow to the low point of the grassy, open area north of Colonial Quadrangle.

For the 5-year, 24-hour rainfall event, the estimated MWSE in the catch basin located in the loading dock area of Colonial Quadrangle, identified as purple in Figure 3-3, is less than two feet from the rim. For the 25-year, 24-hour rainfall event, the estimated MWSE for the manhole also located in this area is less than two feet from the rim. Although overflow is not estimated in this area for the 25-year, 24-hour rainfall event, overflow should flow to a sump surrounding the catch basin should there be any overflow.

For the 25-year, 24-hour rainfall event, catch basins and manholes in the Colonial Drive and Visitors Lot P-1 area have MWSE within two feet of or exceeding the rim elevations of these structures. These structures are identified as red in Figure 3-3. It appears that the MWSE in these structures, are exacerbated as a result of flow from the building drains from the Academic Podium and Colonial Quadrangle into this portion of the subsystem. For areas discharging into the stormwater collection system via building drains, the time of concentrations for these areas were approximated as being 0.1 hours. This time of concentration value is conservative, and it appears that this approximation exacerbates the MWSE in these structures. Nonetheless, based on the topography in these areas, overflow from the catch basins located on Colonial Drive and in Visitors Lot P-1 should be confined to sumps surrounding these catch basins. Overflow from the catch basin located next to Collins Circle should flow along the curb into the catch basin north of this catch basin.

3.2.6 Subsystem VI

For the 2- and 5-year, 24-hour rainfall events, MWSE in manholes and catch basins of this subsystem are estimated to be greater than two feet within the rim elevation. As discussed previously, this subsystem encompasses part of the Academic Podium, the Health and Counseling Building, Colonial Gold Lot – C, the Podium West and Dutch Purple Lots, the Dutch Quadrangle, the northern part of the Dutch Gold Lot, as well as University Field and the Physical Education building. As a result, overflow is not estimated in these areas for the 2- and 5-year, 24-hour rainfall events.

For the 10-year, 24-hour rainfall event, some catch basins in the Dutch Purple Lot and the northern part of the Dutch Gold Lot, and on University Drive West just south of the intersection of Tri-Centennial and University Drive West are estimated as having MWSE within two feet of the rim elevations of these structures. These structures are identified as green in Figure 3-4. For the 25-year, 24-hour rainfall event, additional structures in these areas are estimated as having MWSE within two feet of the rim of these catch basins, and three catch basins located in the northern part of the Dutch Gold Lot are estimated as having MWSE extend above the rim elevation. These three catch basins are identified as a solid red in Figure 3-4.

It appears that the MWSE in the area identified in Figure 3-4 are the result of a conduit constricting the volumetric rate of flow in this portion of the subsystem.

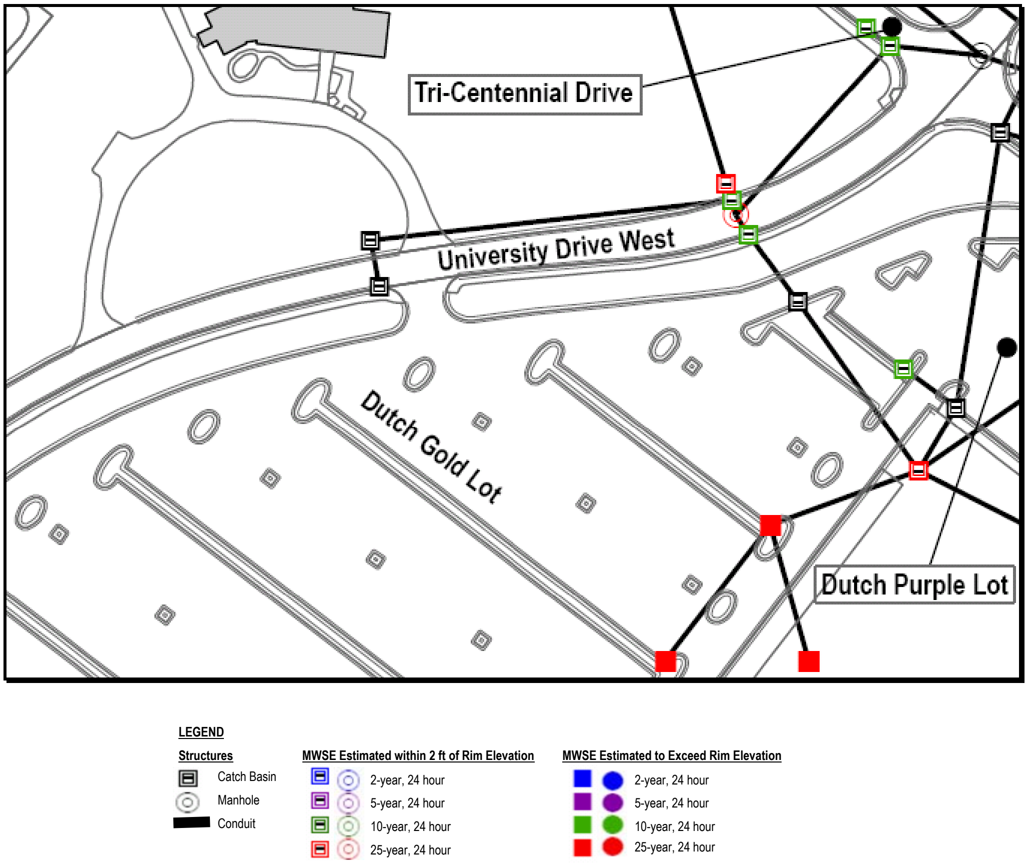


Figure 3-4: Subsystem VI – Dutch Lots

For the 25-year, 24-hour rainfall event, catch basins located in the western loading dock area of the Academic Podium, and areas east of this loading dock, are estimated as having MWSE within two feet of the rim elevation. These structures are identified as red in Figure 3-5. Estimated MWSE for these catch basins do not extend above the rim elevation for the 25-year, 24-hour rainfall event. Should overflow from these catch basins occur, it appears from the topography that overflow should flow to a low point at the entrance of this loading dock.

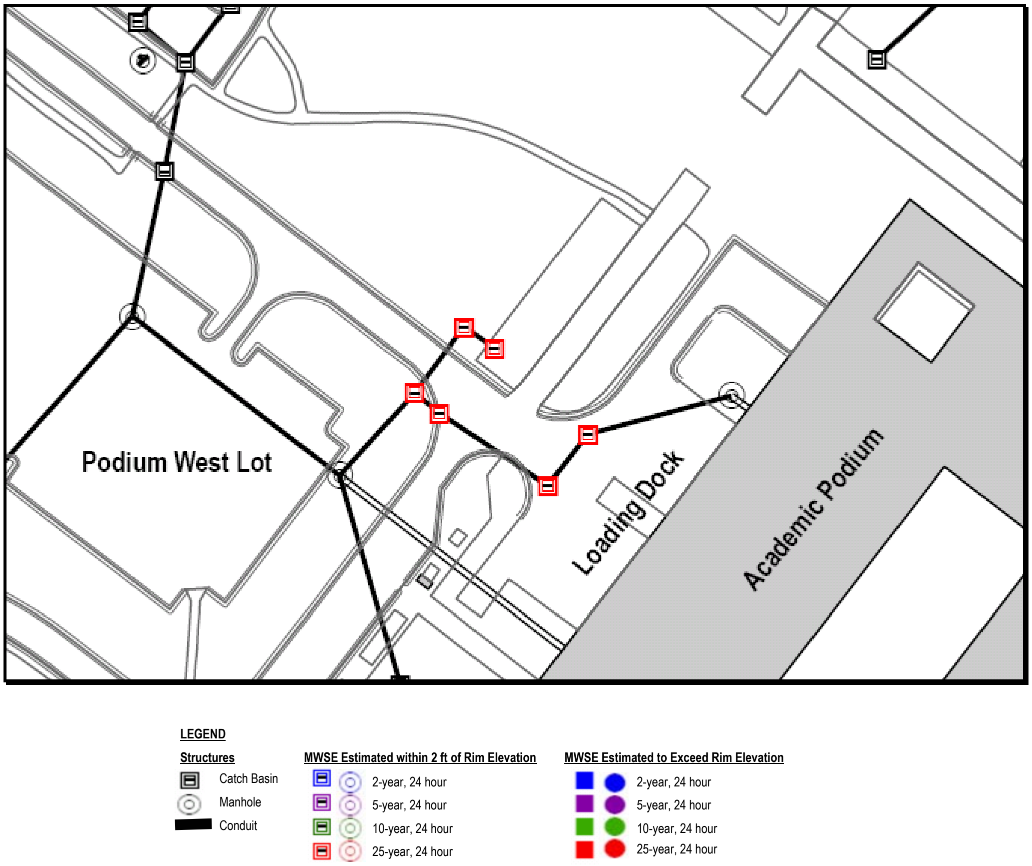


Figure 3-5: Subsystem VI – Academic Podium Western Loading Dock

3.2.7 Subsystem VII

For the 2-, 5-, 10-, and 25-year, 24-hour rainfall events, MWSE in manholes and catch basins of this subsystem are not estimated to be within two feet of the rim elevation. This subsystem is located on the southwestern part of campus, and collects stormwater from the Power Plant and surrounding buildings and lots.

3.2.8 Subsystem VIII

Subsystem VIII is located east of Collins Circle, and encompasses the State Quadrangle, State Gold and Purple Lots, State Drive, and parts Collins Circle, the Academic Podium, and Carillon Drive. For the 2-year, 24-hour rainfall events, maximum surface elevations in manholes and catch basins of this subsystem are not estimated to be within two feet of the rim elevation.

For the 5-year, 24-hour rainfall event, a catch basin located in the open area northeast of the State Quadrangle and is estimated to have MWSE within two feet of the rim elevation. This catch basin is identified as purple in Figure 3-6, and it is estimated that MWSE in this catch basin does not exceed the rim elevations for the 25-year, 24-hour rainfall event. However, should the MWSE be greater than the rim elevation, overflow should be confined to a grassy sump surrounding this catch basin.

For the 10-year, 24-hour rainfall event, estimated MWSE in catch basins and a manhole located in the proximately of the northeast entrance to the State Gold Lot are within two feet of the rim elevation of these structures. These structures are identified as green in Figure 3-6. MWSE in these structures are not estimated to be greater than the rim elevation for the 25-year, 24-hour storm. However, should the MWSE exceed the rim elevation of the two catch basins, if appears from the topography that overflow should be confined to a sump surrounding these catch basins. Overflow from the manhole should flow to a low point in the grassy, open area surrounding this manhole.

Additional catch basins located at the northeast entrance of the State Gold Lot and the State Quadrangle loading dock are estimated as having MWSE within two feet of the rim elevation for the 25-year, 24-hour rainfall event. These structures are identified in red in Figure 3-6. MWSE in these structures are not estimated to be greater than the rim elevation for the 25-year, 24-hour storm. However, should MWSE exceed rim elevations for the catch basins located at the north entrance of the State Gold Lot, it appears from the topography that overflow should flow to the catch basin on University Drive East west of the entrance to the State Gold Lot. Moreover, overflow from the two catch basins located in the loading dock area should flow to the sump surrounding the catch basin located in the driveway of the loading dock. Based on the topography, it appears that the overflow should not obstruct the entrance to the building at this location.

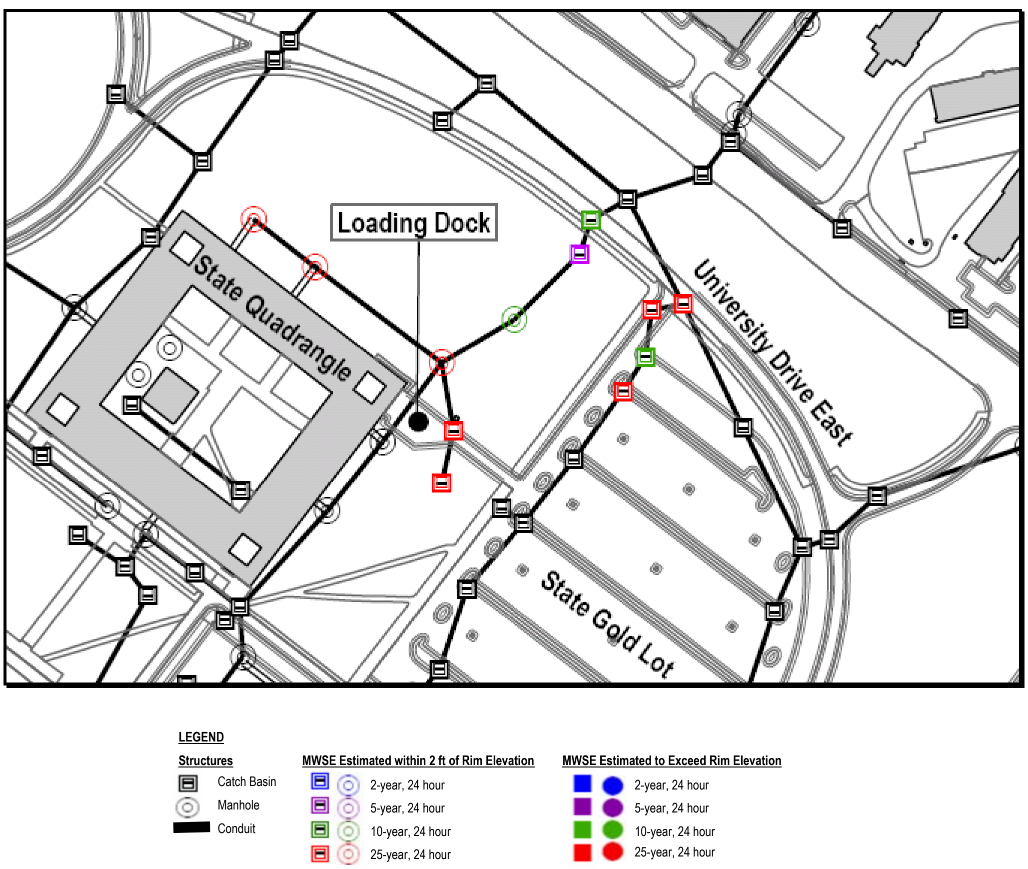


Figure 3-6: Subsystem VIII – State Quadrangle & State Gold Lot

For the 25-year, 24-hour rainfall event, catch basins located in the northern portion of Collins Circle are estimated to have MWSE slightly lower than the rim elevation. It appears from the topography that overflow from these catch basins would be confined to the grassy sump surrounding these two catch basins. Moreover, in catch basins located at the entrance north of Collins Circle, it is estimated that MWSE are less than two feet from the rim elevation. It appears from the topography that overflow from these catch basins should flow to a low point in the road at this entrance. These catch basins are identified in Figure 3-7.

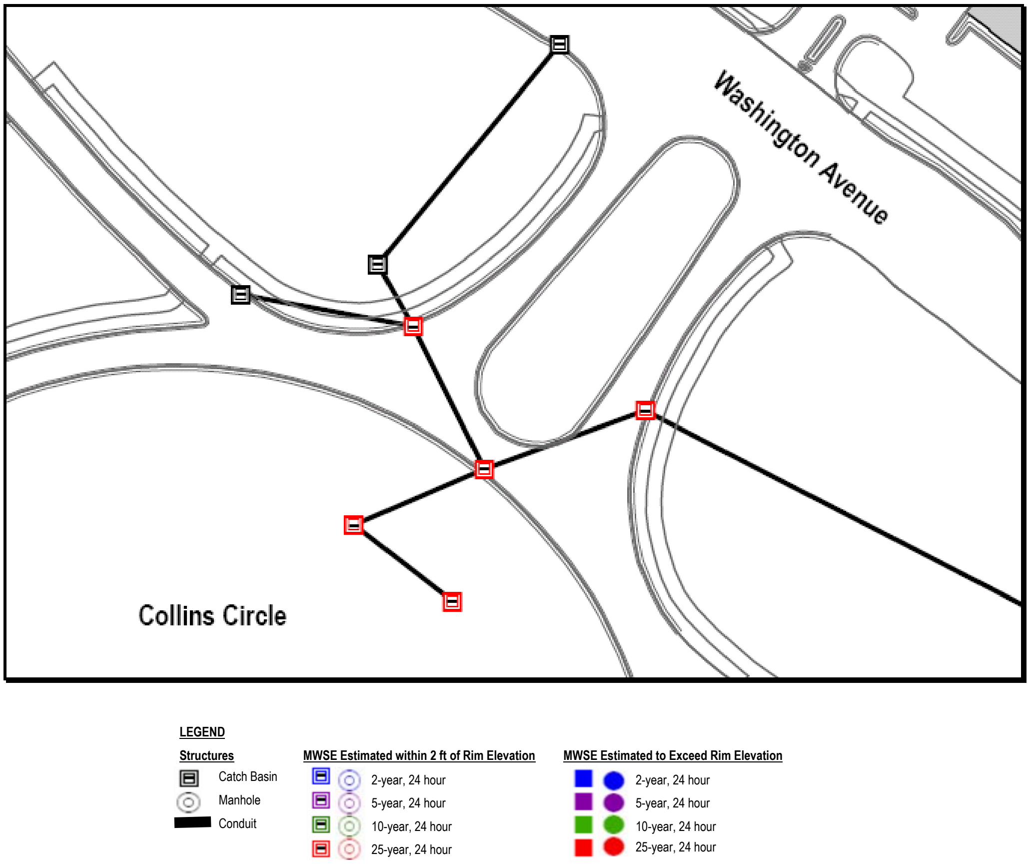


Figure 3-7: Subsystem VIII – Collins Circle

3.2.9 Subsystem IX

Stormwater from the Dutch Field, Indian Quadrangle, and parts of the Academic Podium flows into Subsystem IX. For the 2-, 5-, 10-, and 25-year, 24-hour rainfall events, MWSE in manholes and catch basins of this subsystem are not estimated to be within two feet of the rim elevation. As a result, overflow has not been estimated to occur in the areas serviced by this subsystem.

3.2.10 Subsystem X

Subsystem X is located on the southeastern part of campus, and encompasses the SEFCU Arena, the majority of Dutch Gold Lot, as well as the Dutch Tennis Courts. On the eastern side of the Dutch Gold Lot, MWSE in catch basins are estimated to exceed the rim elevation for the 5- and 25-year, 24-hour rainfall event. As discussed previously, structures with MWSE exceeding the rim elevation for the 5-year, 24-hour rainfall event also have MWSE greater than the rim elevation for the 10- and 25-year rainfall event. It appears from the topography that overflow should be confined to a sump surrounding these catch basins. For other catch basins in this lot, MWSE are not estimated to exceed rim elevations for the 25-year, 24-hour rainfall event but have MWSE within two feet of the rim elevation.

It appears that MWSE in catch basins located in the Dutch Gold Lot are a result of large volumes of stormwater flowing from subbasins into the subsystem and the conduits conveying this stormwater. These subbasins generate large volumes of stormwater due to the imperviousness and size of these subbasins. It appears that the 15-inch conduits, which convey these large volumes of stormwater, are limiting the flow rates through this portion of the subsystem resulting in MWSE being within two feet of, or exceeding, rim elevations.

In addition to the catch basins located in the Dutch Gold Lot, a catch basin located west of the northwestern corner of the Dutch Tennis Court, is estimated as having a MWSE within two feet of the rim of this catch basin for the 5-year, 24-hour rainfall event. For the 25-year, 24-hour rainfall event, the MWSE in this catch basin is not estimated as exceeding the rim elevation. If the MWSE should exceed that of the rim, overflow from this catch basin should be confined to a grassy sump surrounding this catch basin.

Structures discussed in this section are identified in Figure 3-8.

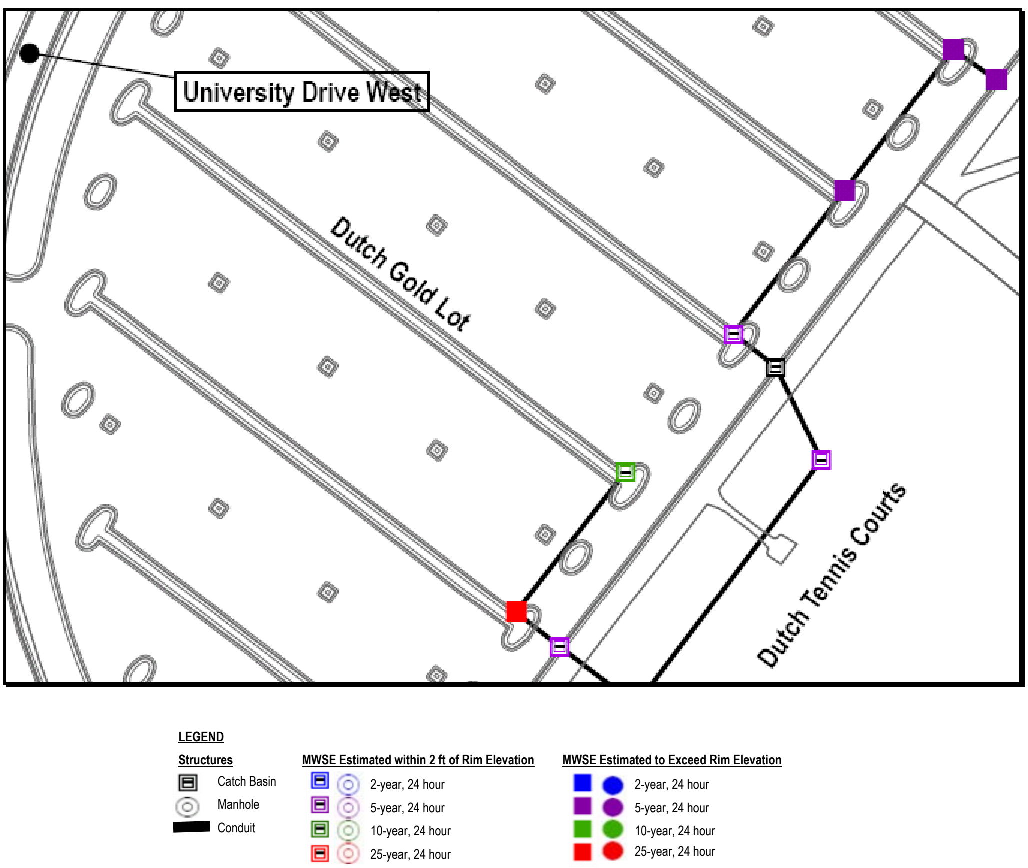


Figure 3-8: Subsystem X – Dutch Gold Lot

3.2.11 Subsystem XI

Subsystem XI collects stormwater from areas on the eastern part of campus including the Life Sciences building, the Visitors Parking Lot-P2, and parts of the Indian Quadrangle. For the 2- and 5-year, 24-hour rainfall events, estimated

MWSE in catch basins and manholes are not estimated as being within two feet of the rim elevation with the exception of a catch basin located in the Loading Dock area. This catch basin is identified as purple in Figure 3-9, and this area is discussed in detail further in this section.

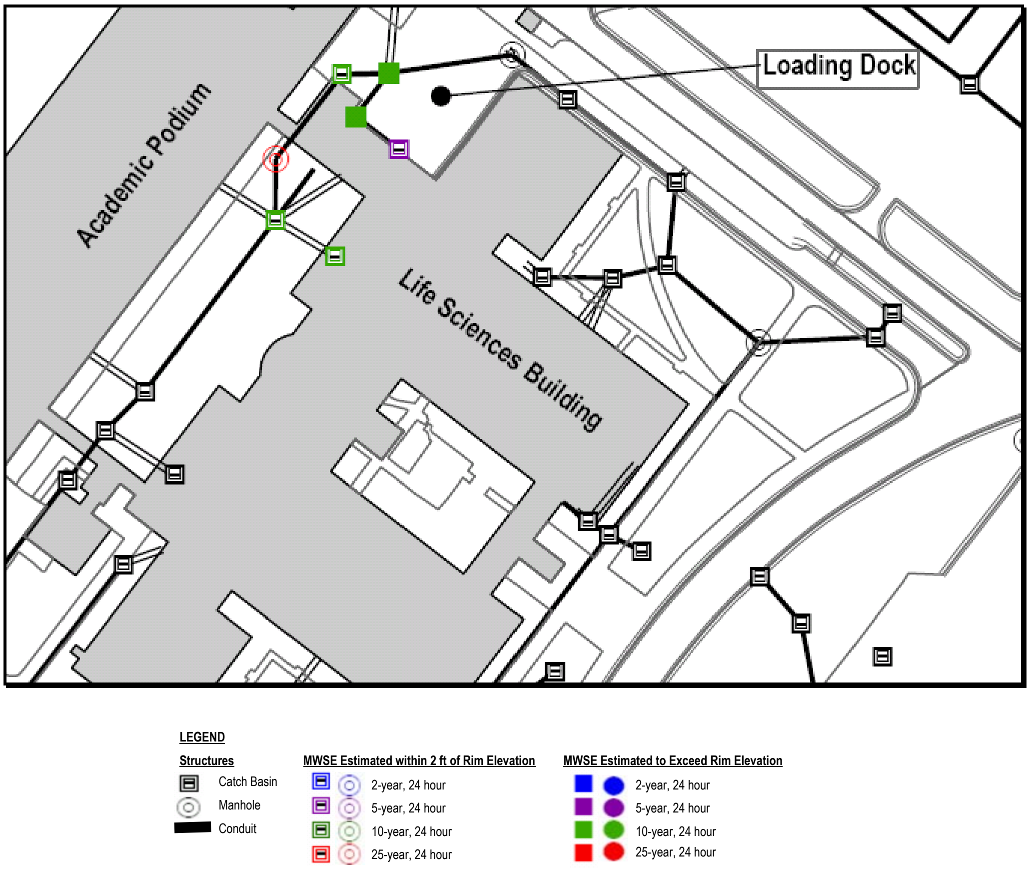


Figure 3-9: Subsystem XI – Life Sciences Building

For the 10- and 25-year, 24-hour rainfall events, it is estimated that MWSE in additional structures in the loading dock area of the Life Sciences building are within two feet of the rim elevation, and for some catch basins, the estimated MWSE exceeds the rim elevation for the 10-year, 24-hour rainfall event. These structures are identified as green in Figure 3-9. It appears that the MWSE in the structures in this area are the result of flow from building drains from the Academic Podium and Life Sciences building. Since the flow paths for stormwater flowing from building areas could not be determined, it was approximated that the time of concentrations for these areas were 0.1 hours. This value is conservative provided the size of the building areas discharging into each structure, and it appears that the MWSE are exacerbated as a result of this approximation.

For the 25-year, 24-hour rainfall event, the MWSE in the catch basin on Indian Drive located approximately 150 feet from the entrance of this road reaches less than two feet of the rim elevation. If the MWSE exceeds the rim elevation of this catch basin, it appears from the topography that the overflow from this catch basin should pond on Indian Drive, then discharge to a catch basin on Justice Drive. The catch basin on Justice Drive is part of Subsystem XII.

3.2.12 Subsystem XII

Subsystem XII is located on the eastern part of campus and includes the area surrounding the building east of the University Police building and parts of Justice Drive. For this subsystem, catch basins and manholes are not estimated as being within two feet of rim elevations during the 2-, 5-, 10-, and 25-year, 24-hour rainfall events.

3.2.13 Subsystem XIII

During the 2-, 5-, and 10-year, 24-hour rainfall events, MWSE in structures in Subsystem XIII are not estimated to be within two feet of the rim elevations. This subsystem collects stormwater eastern part of campus and includes the University Police Station building and the Boor Sculpture Studio.

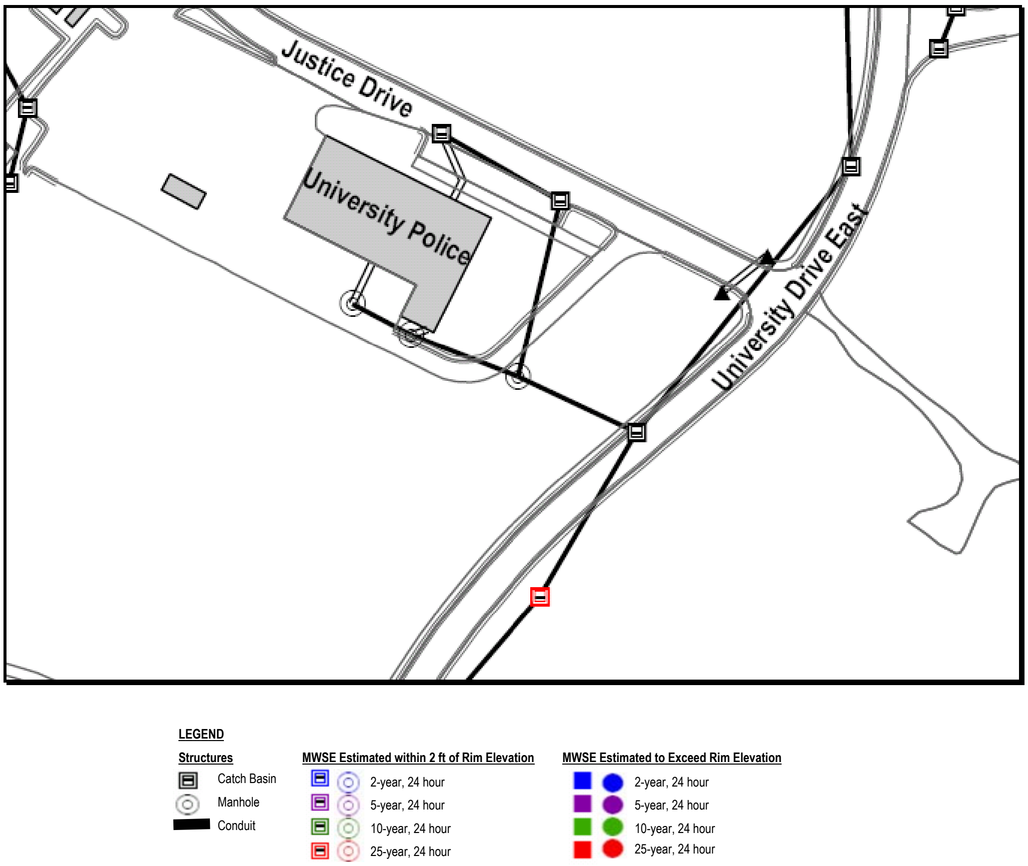


Figure 3-10: Subsystem XIII – University Drive East

Only for the 25-year, 24-hour rainfall event does a catch basin located east of the University Police building on University Drive East have an estimated MWSE within two feet below of the rim elevation. This catch basin is identified as red in Figure 3-10. If the MWSE should be greater than the rim elevation in this catch basin, it appears from the topography that overflow should flow to a low point in a forested area.

3.2.14 Subsystem XIV

Subsystem XIV collects stormwater from portions of the southern end of campus, including the SEFCU Arena Gold Lot and the Athletic Practice Fields. For the 2-year, 24-hour rainfall event, estimated MWSE are not within two feet of the rim of the structures. For the 5-, 10-, and 25-year, 24-hour rainfall events, Figure 3-11 identifies structures within this area that have MWSE within two feet or exceeding the rim elevation.

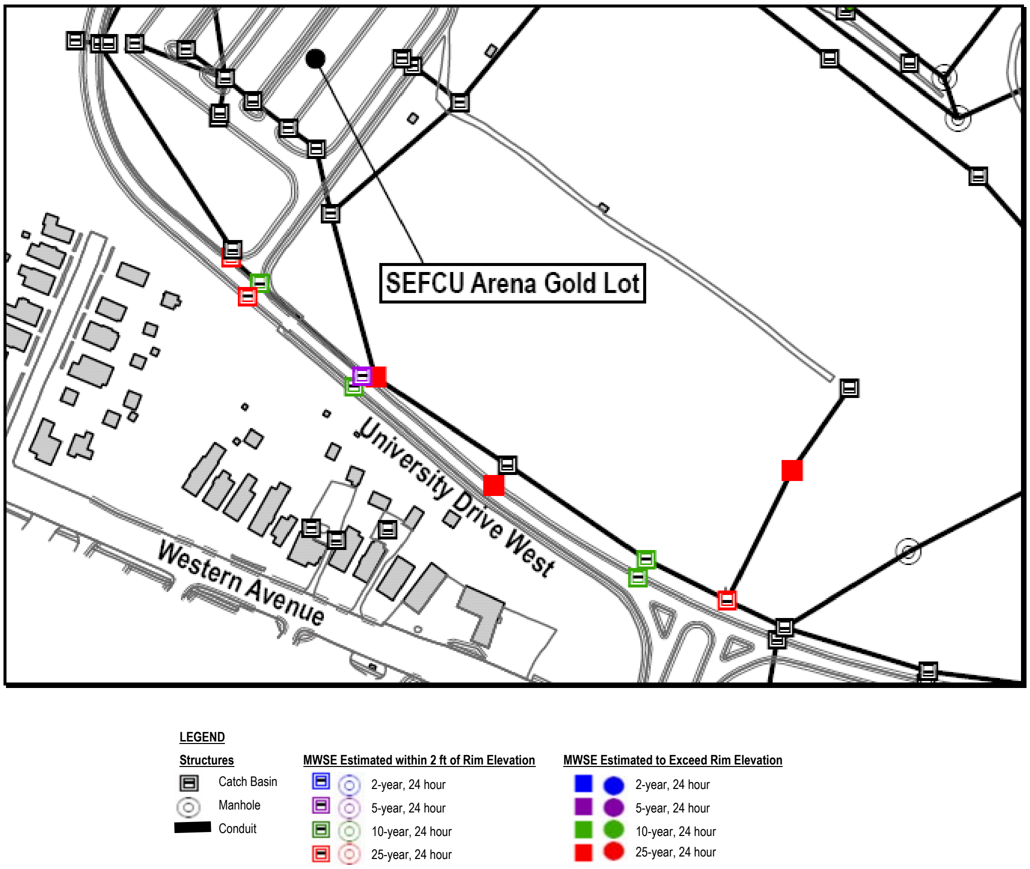


Figure 3-11: Subsystem XIV – University Drive West

Catch basins along University Drive West and catch basins just north of this road are estimated as having MWSE within two feet of, and not exceeding, the rim elevation for the 5- and 10-year, 24-hour rainfall events. This stretch of University Drive West commences at the entrance from Western Avenue and ends at the SEFCU Arena Gold Lot entrance. Only for the 25-year, 24-hour rainfall event do catch basins along this stretch have estimated MWSE that exceed the rim elevations. As discussed previously, overflow estimated during the 25-year, 24-hour rainfall event is least likely to occur at the campus when compared to the 2-, 5-, and 10-year, 24-hour rainfall events.

It appears that the MWSE in the catch basins in this area of Subsystem XIV are a result of the large volumes of stormwater flowing from the large, grassy subbasins, north of University Drive West, into this subsystem. Based on the topography, overflow from catch basins located in the grassy area directly north of University Drive West should flow through a swale to a low point located in this grassy, open area. It does not appear that stormwater accumulation in this area should have negative impacts to the campus.

3.2.15 Water Quality Volume in Retention Pond

From the Pond Assessment Report prepared by Woodard & Curran, last revised September 24, 2007, the volume of the retention pond located on the southeastern part of campus, east of the lacrosse and hockey fields, will be approximately 9.1 million gallons once the pond dredging project currently in progress is completed. This value assumes that the sediment and muck has been removed from the pond as recommended in the Pond Assessment Report.

Currently, Subsystems VI, IX, X, XI, XII, and XIII and the area surrounding the retention pond discharge stormwater runoff into the pond. Runoff discharged into the retention pond is captured and treated by the pond, and the volume of runoff which is treated is known as the water quality volume. To determine the potential water quality volume that can be provided by the retention pond for future development, the current water quality volume provided by the retention pond must be calculated.

Per the New York State Department of Environmental Conservation Stormwater Management Design Manual dated April 2008, 90% of the average annual stormwater runoff volume of a site must be captured and treated to remove pollutants. The water quality volume is directly related to the amount of impervious cover at a site, the site area, and the 90% rainfall event number of the site. In order to determine the current water quality volume provided by the retention pond, the amount of impervious cover of each subsystem and the area surrounding the retention pond was calculated. Table 3-2 summarizes the amount of impervious cover and total area of the subsystems and area discharging into the retention pond.

Table 3-2: Subsystems and Areas Discharging to Retention Pond

Subsystem or Area	Impervious Area (ft^2)	Total Area (ft^2)
VI	1,180,484	2,408,157
IX	844,789	1,231,680
X	951,021	1,907,088
XI	586,578	716,930
XII	89,379	446,045
XIII	135,172	358,639
Surrounding Area	63,526	882,372
Total	3,850,949	7,950,911

From the values in Table 3-2 and assuming a 90% rainfall event number of 0.95 which was obtained from the Design Manual, Table 3-3 summarizes the water quality volume requiring treatment by the retention pond.

Table 3-3: Water Quality Volume

Subsystem or Area	Water Quality Volume (gallons)
VI	700,489
IX	486,732
X	563,351
XI	333,867
XII	60,845
XIII	82,664
Surrounding Area	4,319
Total	2,232,268

The current volume of stormwater runoff captured and treated by the retention pond is approximately 2.2 million gallons as provided in Table 3-3. With a retention pond volume of approximately 9.1 million gallons under the assumption the sediment is removed per the Pond Assessment Report, the available water quality volume which can be provided by the retention pond for future development is approximately 6.9 million gallons.

Per the Design Manual, stormwater runoff discharging into a retention pond from future development must be pretreated. Pretreatment can be provided by a sediment forebay or equivalent upstream pretreatment. Typical examples include earthen dams, concrete weirs, and gabion baskets. However, if the stormwater runoff from an inlet to the retention pond exceeds 10% or more of the total design storm flow to the retention pond, a sediment forebay must be provided as pretreatment. The forebay shall be sized to contain 10% of the water quality volume, and the water quality volume provided by the forebay counts toward the total water quality requirement. In either scenario, the New York State Department of Environmental Conservation should be contacted as to which pretreatment measures should be implemented for future development as the actual measured required will be specific to the proposed development.

3.3 CONCLUSIONS

Overall, it appears the capacity of 9 of the 14 subsystems are adequate for up to the 25-year, 24-hour rainfall event assuming that all structures installed as identified on the plans supplied to Woodard & Curran and that the structures and conduits are free of debris and sediment. Structures within these nine subsystems may have been identified in the previous section as having MWSE within two feet of the rim elevation. However, it appears these predicted occurrences are isolated incidents and may not necessarily imply limited capacity of the subsystem. As a result, these nine subsystems are not discussed further in this section.

The remaining five subsystems have a significant number of structures with estimated MWSE within two feet of the rim elevations. Given the significant number of structures with elevated MWSE values, it does not appear that these predicted occurrences would be isolated incidents. The structures with elevated MWSE values are generally clustered together and as a result indicate that there may be limited capacity in these areas. Thse five subsystems are discussed further below.

The following is a list of the subsystems and respective areas which appear, from the model results, to have limited capacity. Potential impacts from stormwater overflow in these areas, as they appear from the topography in these locations, are described, and the list is ordered from subsystems having greatest potential impact from overflow to least potential impact from overflow.

1. Subsystem VIII - Areas in the vicinity of the State Quadrangle, State Gold Lot, and Collins Circle (Figure 3-6 and Figure 3-7):
    - Stormwater overflow from structures located north of Collins Circle may pond in this area and impact traffic through the main entrance to the campus from Washington Avenue and the part of University Drive north of Collins Circle.
    - Stormwater overflow from structures located in the State Gold Lot may pond in this parking area and limit the amount of available parking.
    - Stormwater overflow from structures located north of State Quadrangle should flow to an open, grassy area. It appears that stormwater ponding in this area is of limited concern.
    - Expansion projects potentially impacted: School of Business, State Quad Renovations, State Quad Parking Expansion
2. Subsystem XIV - University Drive West closest to the entrance of Western Avenue (Figure 3-11):
    - Stormwater overflow from structures located in this portion of University Drive West may impact traffic through the entrance to the campus from Western Avenue and the part of University Drive West adjacent to this entrance.
    - Expansion projects potentially impacted: Stadium
3. Subsystem V - Areas adjacent to Colonial Quadrangle (Figure 3-3):
    - Stormwater overflow from structures located south of Colonial Quadrangle, such as structures in Visitors Lot P-1 and in the Colonial Drive parking area, may pond in these areas and limit the amount of available parking.
    - Stormwater overflow from structures located north of Colonial Quadrangle should flow to an open, grassy area. It appears that stormwater ponding in this area is of limited concern.
    - Expansion projects potentially impacted: None
4. Subsystems VI and Subsystem X - Dutch Gold Lot (Figure 3-4 and
5. Figure **3-8**):
    - Stormwater overflow from structures in the Dutch Gold Lot may pond in this parking area and limit the amount of available parking.
    - Expansion projects potentially impacted: Dutch Quad Renovations

It is recommended that the above areas be investigated further to evaluate the necessity and measures required to improve the capacity of the subsystems in these areas.

Assuming the pond sediment is removed from the retention pond, the water quality volume which can be potentially provided by the retention pond is approximately 6.9 million gallons, and stormwater runoff discharging from future development into the retention pond must be pretreated. Discussions should be held with the New York State Department of Environmental Conservation regarding which pretreatment measures should be implemented.

4. IRRIGATION SYSTEM

4.1 METHOD

The area that can be irrigated is a function of the type of plant material being irrigated, rainfall conditions, evapotranspiration potential, irrigation water supply capacity, and efficiency of the irrigation system. The area that can be irrigated was calculated based on turfgrass vegetation; an irrigation water supply capacity of 1,200 gallons per minute, which is the capacity of the irrigation pump station; and an irrigation system efficiency of 75%. This irrigation demand was compared to the capacity of the stormwater pond used as the source for irrigation water supply.

4.2 IRRIGATION SUPPLY CALCULATION RESULTS

Based on evapotranspiration potential, the month of July is the limiting month for irrigation, with an evapotranspiration potential of 6.82 inches. June and August are the next most limiting months with evapotranspiration potentials of 6.30 inches and 5.89 inches, respectively.

During drought conditions (zero rainfall) in the month of July, on a pumping schedule of seven hours per day, seven days per week, an area of approximately 75 acres of turfgrass could be irrigated based on the pump station's capacity. With average rainfall and a mix of plant material and turf, an area of approximately 100 acres could be irrigated. The calculations in support of this determination can be found in Appendix B.

The volume of the stormwater pond used as the irrigation water source was estimated to have an existing volume of 6.6 million gallons. Dredging sediment from this pond is expected to provide an additional 2.5 million gallons (38%) of storage capacity. If the pond dredging takes place at the proposed magnitude, approximately 9.1 million gallons of storage is available.

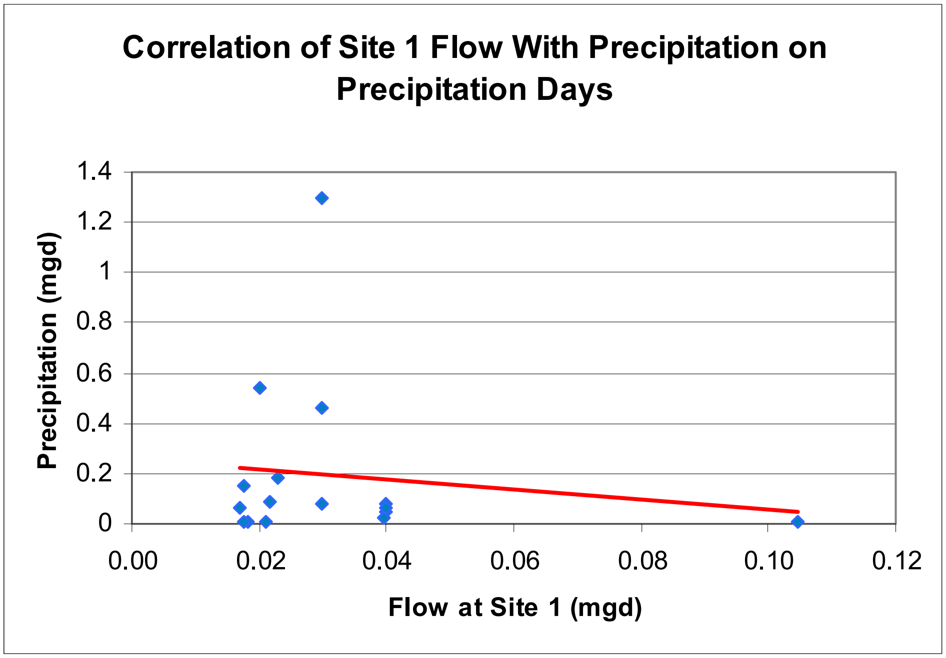
During drought conditions in the month of July, if the maximum potential area of turf is irrigated (75 acres), the stormwater pond would be able to provide fewer than 19 days of irrigation capacity. During similar conditions in June and August, the pond could provide fewer than 20 and 22 days of irrigation water supply, respectively. The actual number of days of irrigation supply available will be dependent on the usable water from the retention pond including factors such as intake elevation and turbidity.

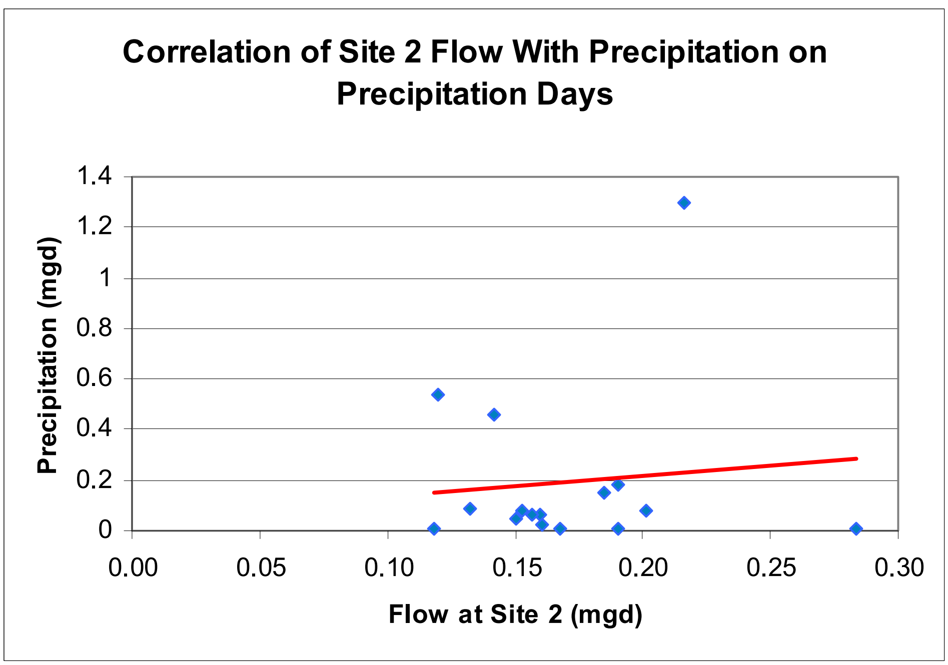
For every inch of rainfall on the campus, approximately 1,600,000 gallons of water is directed to the stormwater pond. This historic average rainfall for Albany in the month of July is approximately 4.4 inches, translating into a pond recharge of over 7,000,000 gallons.

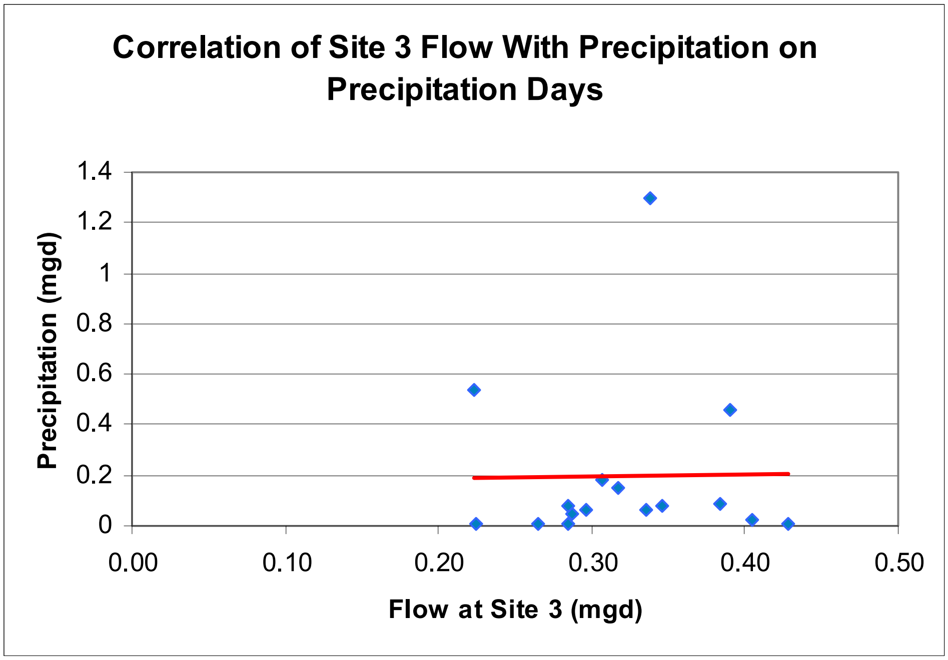
4.3 CONCLUSIONS

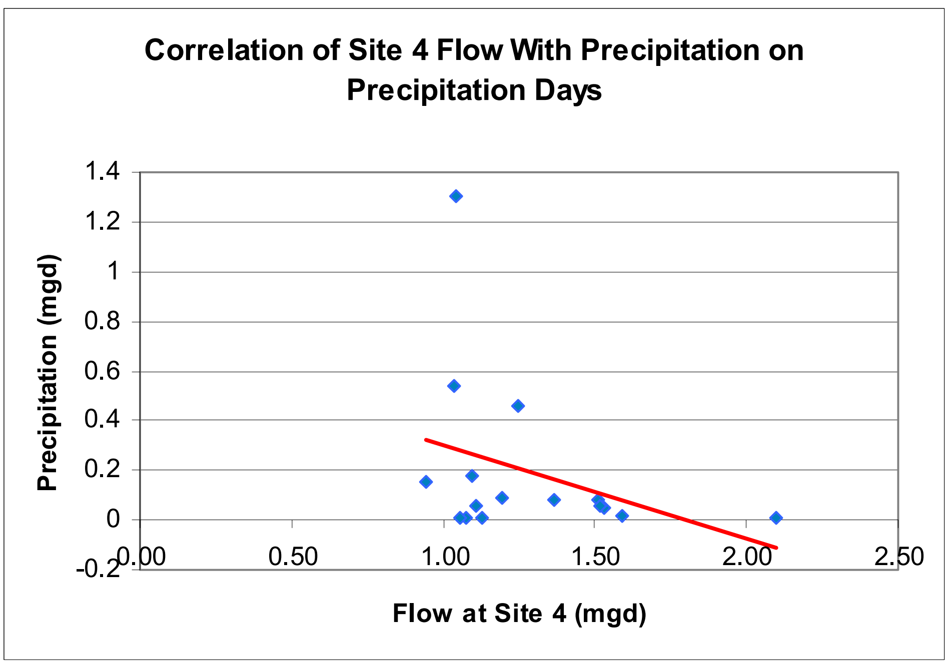
Based on irrigation system mapping, approximately 45 acres of the campus are outfitted for irrigation. This is less than the calculated maximum amount of area that can be irrigated using the existing stormwater pond and irrigation pumping station. Additional capacity exists for the expansion of the irrigated area, if desired by the University. This irrigation capacity analysis assumes that the irrigation system is in good repair, without major leaks that would reduce the system's capacity.

APPENDIX A: FLOW AND PRECIPITATION STATISTICAL ANALYSIS


Correlation of Site 1 Flow With Precipitation on Precipitation Days
Precipitation (mgd)
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0.00
0.02
0.04
0.06
0.08
0.10
0.12
Flow at Site 1 (mgd)


Correlation of Site 2 Flow With Precipitation on Precipitation Days
Precipitation (mgd)
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0.00
0.05
0.10
0.15
0.20
0.25
0.30
Flow at Site 2 (mgd)


Correlation of Site 3 Flow With Precipitation on Precipitation Days
Precipitation (mgd)
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0.00
0.10
0.20
0.30
0.40
0.50
Flow at Site 3 (mgd)


Correlation of Site 4 Flow With Precipitation on Precipitation Days
Precipitation (mgd)
1.4
1.2
1
0.8
0.6
0.4
0.2
0
-0.2
0.00
0.50
1.00
1.50
2.00
2.50
Flow at Site 4 (mgd)

APPENDIX B: IRRIGATION CAPACITY CALCULATIONS

NORTHERN DESIGNS LLC
IRRIGATION CONSULTANTS & DESIGNERS
2089 Hartford Turnpike
North Haven, Connecticut 06473
Phone (203) 239-2710 Fax (203) 239-2715
email: mastram@northerndesigns.com

May 1, 2008

SUNY ALBANY

Albany, New York

Estimated Annual Water Use

	APRIL	MAY	JUNE	JULY	AUG	SEPT	OCT
MONTHLY ET: [Historical - Albany Area]	3.30"	5.27"	6.30"	6.82"	5.89"	3.89"	2.48"
MONTHLY RAINFALL: [1971-2000 Average - Albany)	3.25"	3.67"	3.74"	3.50"	3.68"	3.31"	3.23"

PLANT MATERIAL: COOL SEASON TURFGRASS

TOTAL AREA IN SQ. FT:		3,267,000
TOTAL AREA IN ACRES:		75.00
IRRIGATION SYSTEM EFFICIENCY (DU):	Sprinklers	75%
SPECIES FACTOR (Ks):	Average	0.80
DENSITY FACTOR (Kd):	High	1.00
MICROCLIMATE FACTOR (Kmc):	High	1.00
LANDSCAPE COEFFICIENT (Ks x Kd x Kmc):		80%

	APRIL	MAY	JUNE	JULY	AUGUST	SEPT	OCT
% EFFECTIVE RAINFALL Drought Conditions	**0%**						
Gallons per Month	7,168,656	11,448,126	13,685,616	14,815,222	12,794,965	8,450,325	5,387,354
Gallons per Week	1,792,164	2,862,032	3,421,404	3,703,806	3,198,741	2,112,581	1,346,838
Gallons per Day	238,955	369,294	456,187	477,910	412,741	281,677	173,786
Gallons per Year	73,750,264						
% EFFECTIVE RAINFALL Average Conditions	**40%**						
Gallons per Month	4,344,640	8,259,161	10,435,825	11,773,974	9,597,310	5,574,173	2,580,716
Gallons per Week	1,086,160	2,064,790	2,608,956	2,943,494	2,399,327	1,393,543	645,179
Gallons per Day	144,821	266,425	347,861	379,806	309,591	185,806	83,249
Gallons per Year	52,565,799						

	APRIL	MAY	JUNE	JULY	AUGUST	SEPT	OCT
TOTALS							
% Effective Rainfall	0%						
Gallons per Month	7,168,656	11,448,126	13,685,616	14,815,222	12,794,965	8,450,325	5,387,354
Gallons per Week	1,792,164	2,862,032	3,421,404	3,703,806	3,198,741	2,112,581	1,346,838
Gallons per Day	231,247	369,294	456,187	477,910	412,741	281,677	173,786
Gallons per Year	73,750,264						
TOTALS							
% Effective Rainfall	40%						
Gallons per Month	4,344,640	8,259,161	10,435,825	11,773,974	9,597,310	5,574,173	2,580,716
Gallons per Week	1,086,160	2,064,790	2,608,956	2,943,494	2,399,327	1,393,543	645,179
Gallons per Day	140,150	266,425	347,861	379,806	309,591	185,806	83,249
Gallons per Year	52,565,799						
TOTALS							
Cost Per 100 Cubic ft.	$4.35						
Cost per Month	$25,266.29	$48,031.21	$60,689.63	$68,471.64	$55,813.23	$32,416.65	$15,008.18
Cost per Year	$305,696.83						

IRRIGATION DAYS PER WEEK 7

IRRIGATION HOURS PER DAY — **GALLONS PER MINUTE REQUIRED**

6	711	1,136	1,358	1,470	1,269	838	534
7	610	973	1,164	1,260	1,088	719	458
8	533	852	1,018	1,102	952	629	401
9	474	757	905	980	846	559	356
10	427	681	815	882	762	503	321

IRRIGATION DAYS PER WEEK 5

IRRIGATION HOURS PER DAY — **GALLONS PER MINUTE REQUIRED**

6	996	1,590	1,901	2,058	1,777	1,174	748
7	853	1,363	1,629	1,764	1,523	1,006	641
8	747	1,193	1,426	1,543	1,333	880	561
9	664	1,060	1,267	1,372	1,185	782	499
10	597	954	1,140	1,235	1,066	704	449

IRRIGATION DAYS PER WEEK 3

IRRIGATION HOURS PER DAY — **GALLONS PER MINUTE REQUIRED**

6	1,659	2,650	3,168	3,429	2,962	1,956	1,247
7	1,422	2,271	2,715	2,940	2,539	1,677	1,069
8	1,245	1,988	2,376	2,572	2,221	1,467	935
9	1,106	1,767	2,112	2,286	1,975	1,304	831
10	996	1,590	1,901	2,058	1,777	1,174	748

APPENDIX C: WATER MODEL FIGURES

Figure C-1: Hydraulic Grade Elevation and Pipe Flow With Current Peak Demand

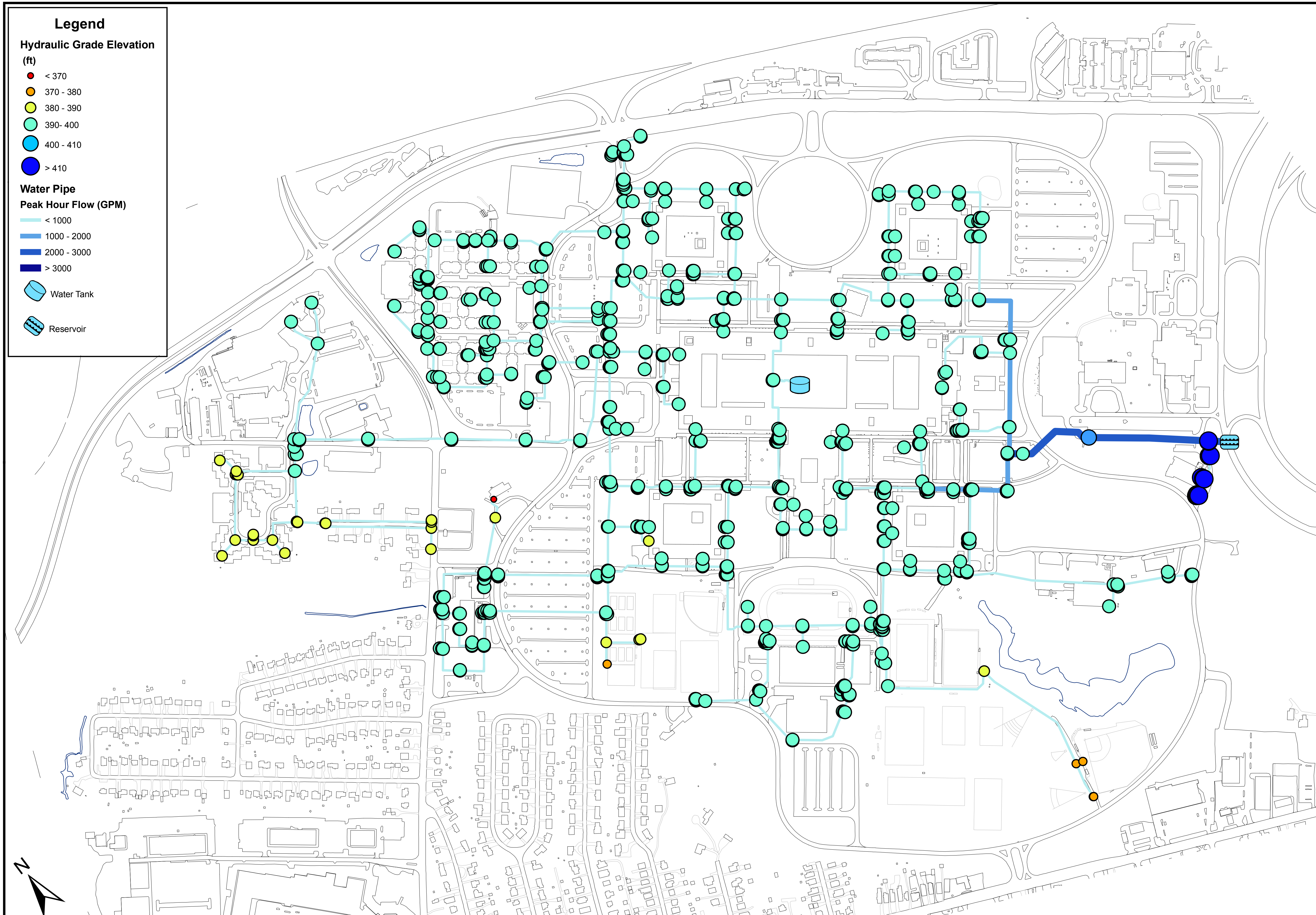
Figure C-2: Available Fire Flow and Pipe Diameter With Current Peak Demand

Figure C-3: Hydraulic Grade Elevation and Pipe Flow With Expanded Peak Demand

Figure C-4: Available Fire Flow and Pipe Diameter With Expanded Peak Demand

Figure C-5: Hydraulic Grade Elevation and Pipe Flow With Secondary Source

Figure C-6: Available Fire Flow and Pipe Diameter With Secondary Source



Hydraulic Grade Elevation and Pipe Flow With Current Peak Demand

Drawn By: AF

Checked By: SCR

\\Whiteplains\Projects\213534 SUCF - SUNY Albany - Asset Management\wip\GIS\WaterCapacityAssessment\PDF\Figure1.pdf

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Commitment & Integrity Drive Results

University at Albany Capactiy Assessment

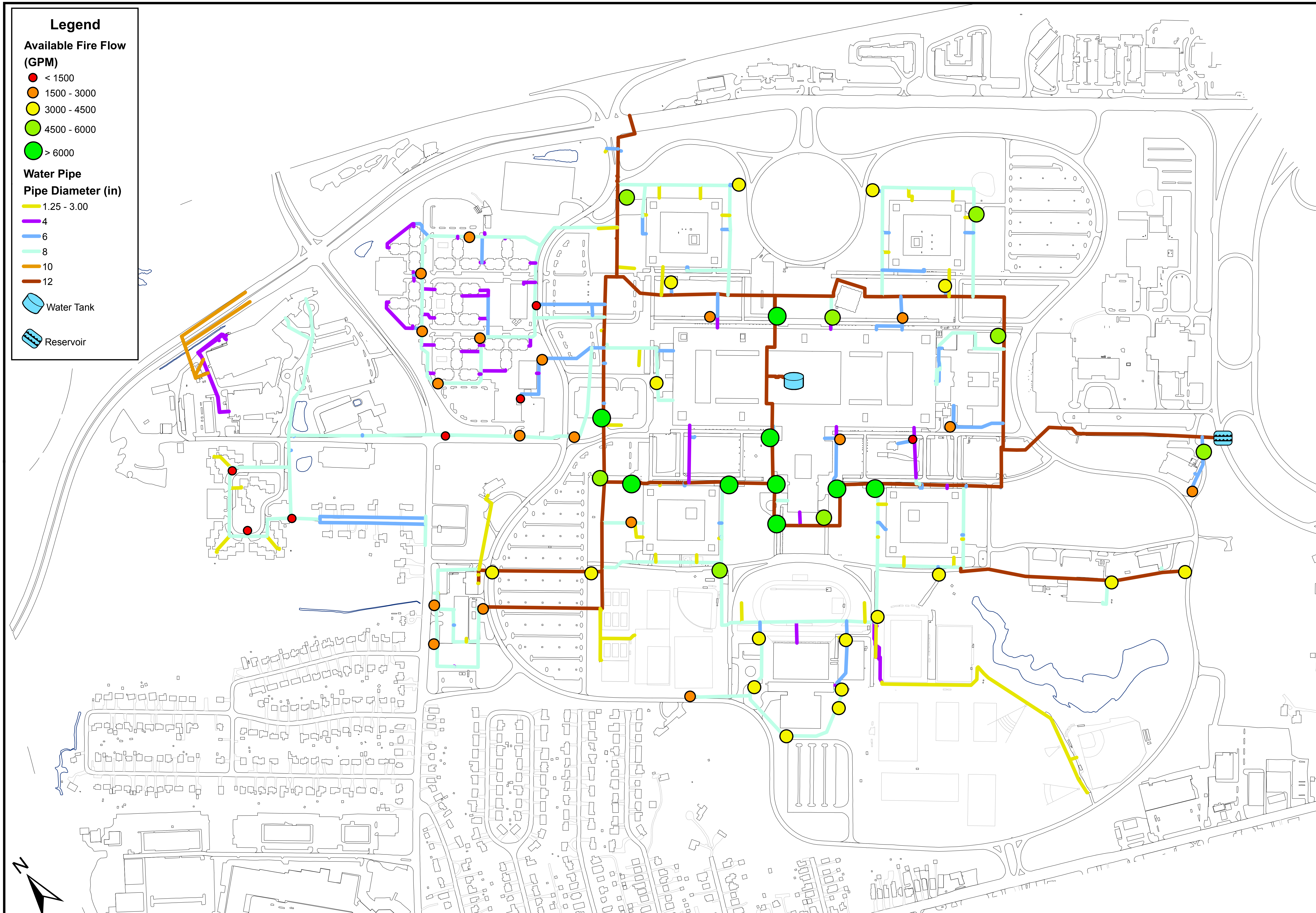
Feet: 0 125 250 500 750 1,000

Date: 10/03/08

Job #213564

Scale: 1" = 500'

Figure: C-1



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Commitment & Integrity Drive Results

Available Fire Flow and Pipe Diameter With Current Peak Demand

Drawn By: AF

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..\Whiteplains\Projects\213534 SUCF - SUNY Albany - Asset Management\wp\GIS\WaterCapacityAssessment\PDF\Figure2.pdf

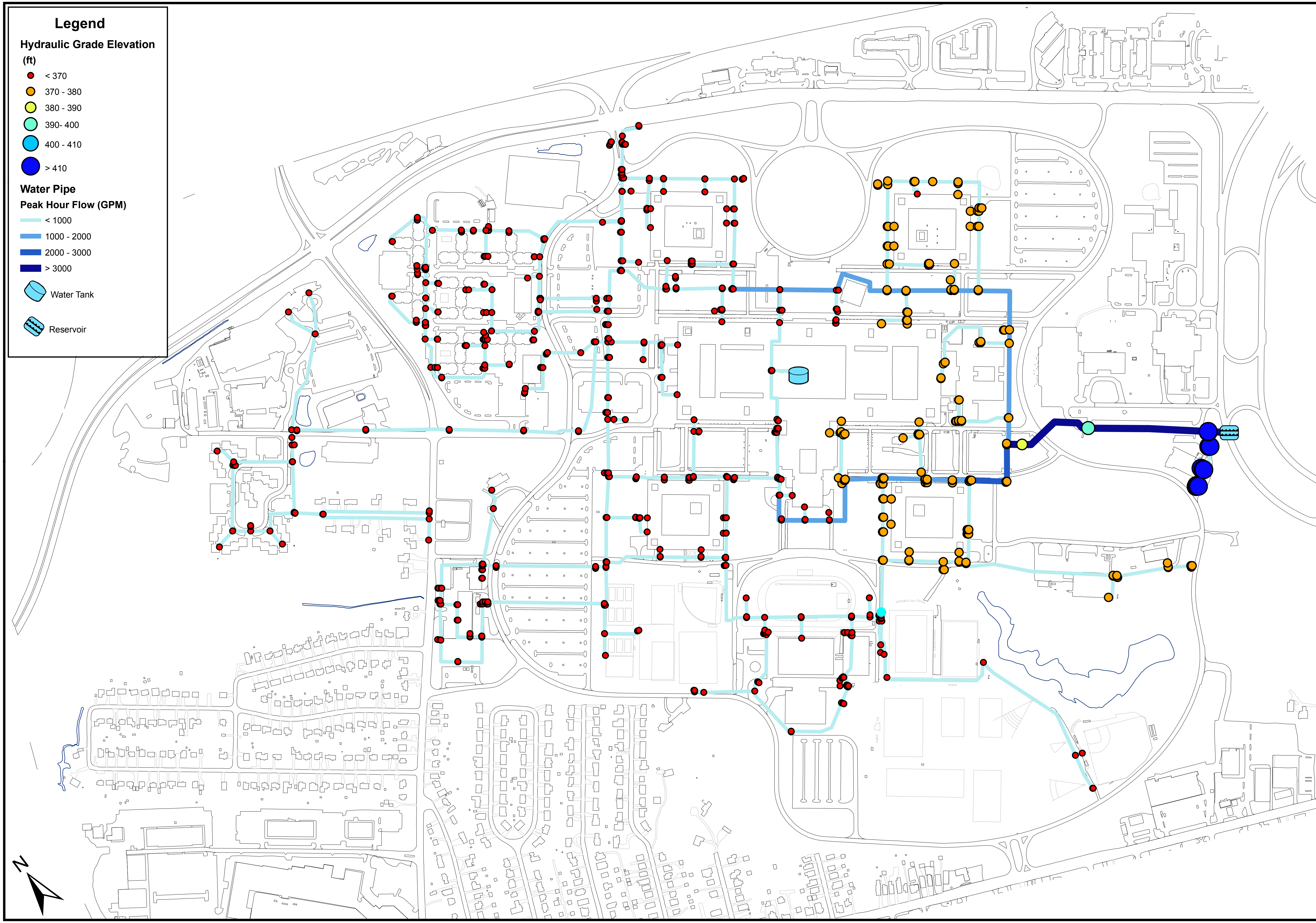
University at Albany Capacity Assessment

Date: 10/03/08

Job #213564

Scale: 1" = 500'

Figure: C - 2



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Hydraulic Grade Elevation and Pipe Flow With Expanded Peak Demand

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..\\Whiteplains\Projects\213534 SUCF - SUNY Albany - Asset Management\wip\GIS\WaterCapacityAssessment\PDF\Figure3.pdf

University at Albany Capactiy Assessment

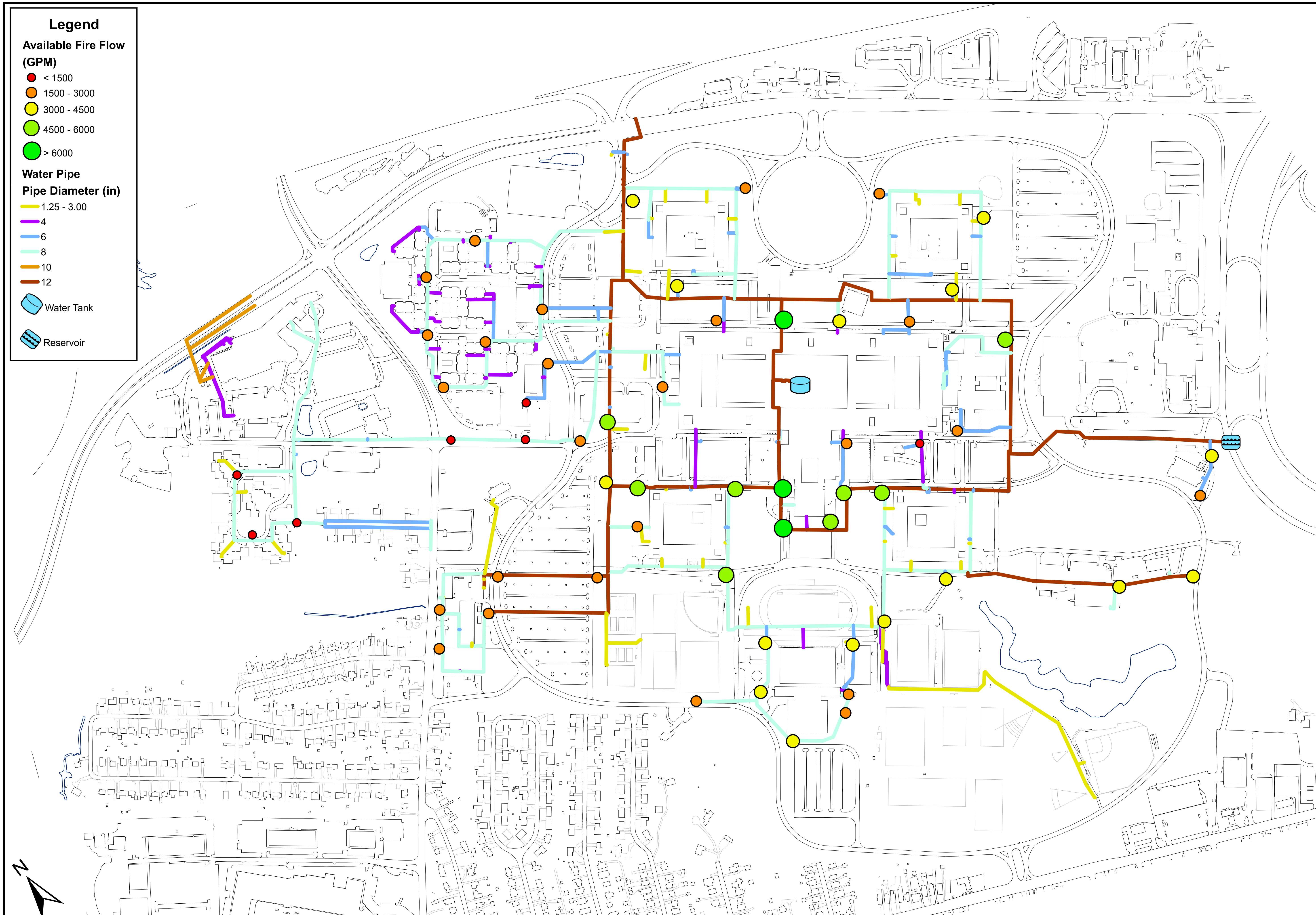
Feet
0 125 250 500 750 1,000

Date: 10/03/08

Job #213564

Scale: 1" = 500'

Figure: C-3



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Commitment & Integrity Drive Results

Available Fire Flow and Pipe Diameter With Expanded Peak Demand

Drawn By: AF

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..\Whiteplains\Projects\213534 SUCF - SUNY Albany - Asset Management\wp\GIS\WaterCapacityAssessment\PDF\Figure6.pdf

University at Albany Capacity Assessment

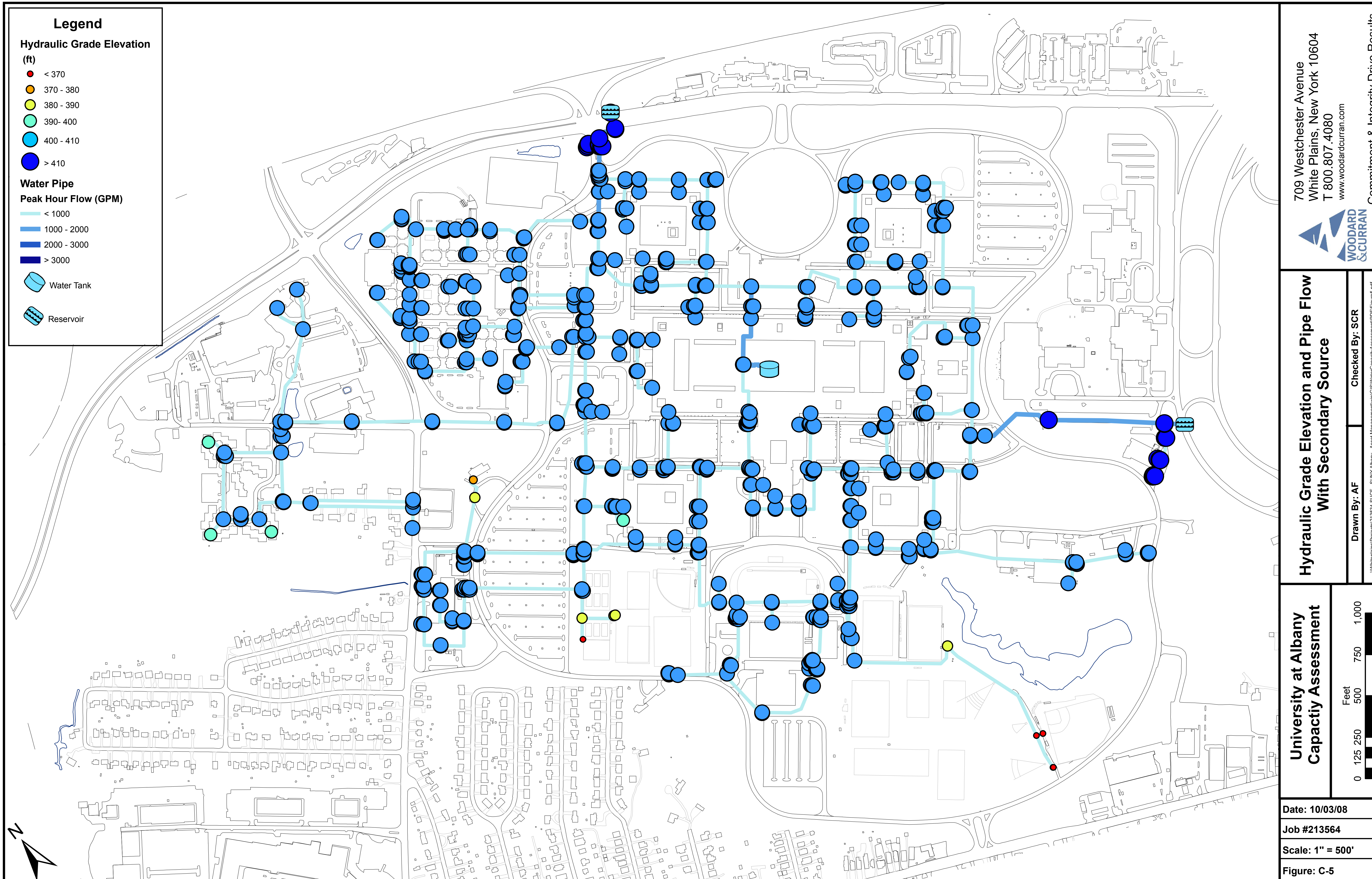
Feet: 0, 125, 250, 500, 750, 1,000

Date: 10/03/08

Job #213564

Scale: 1" = 500'

Figure: C-4



Legend
Hydraulic Grade Elevation (ft)
< 370
370 - 380
380 - 390
390- 400
400 - 410
> 410
Water Pipe
Peak Hour Flow (GPM)
< 1000
1000 - 2000
2000 - 3000
> 3000
Water Tank
Reservoir
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Commitment & Integrity Drive Results
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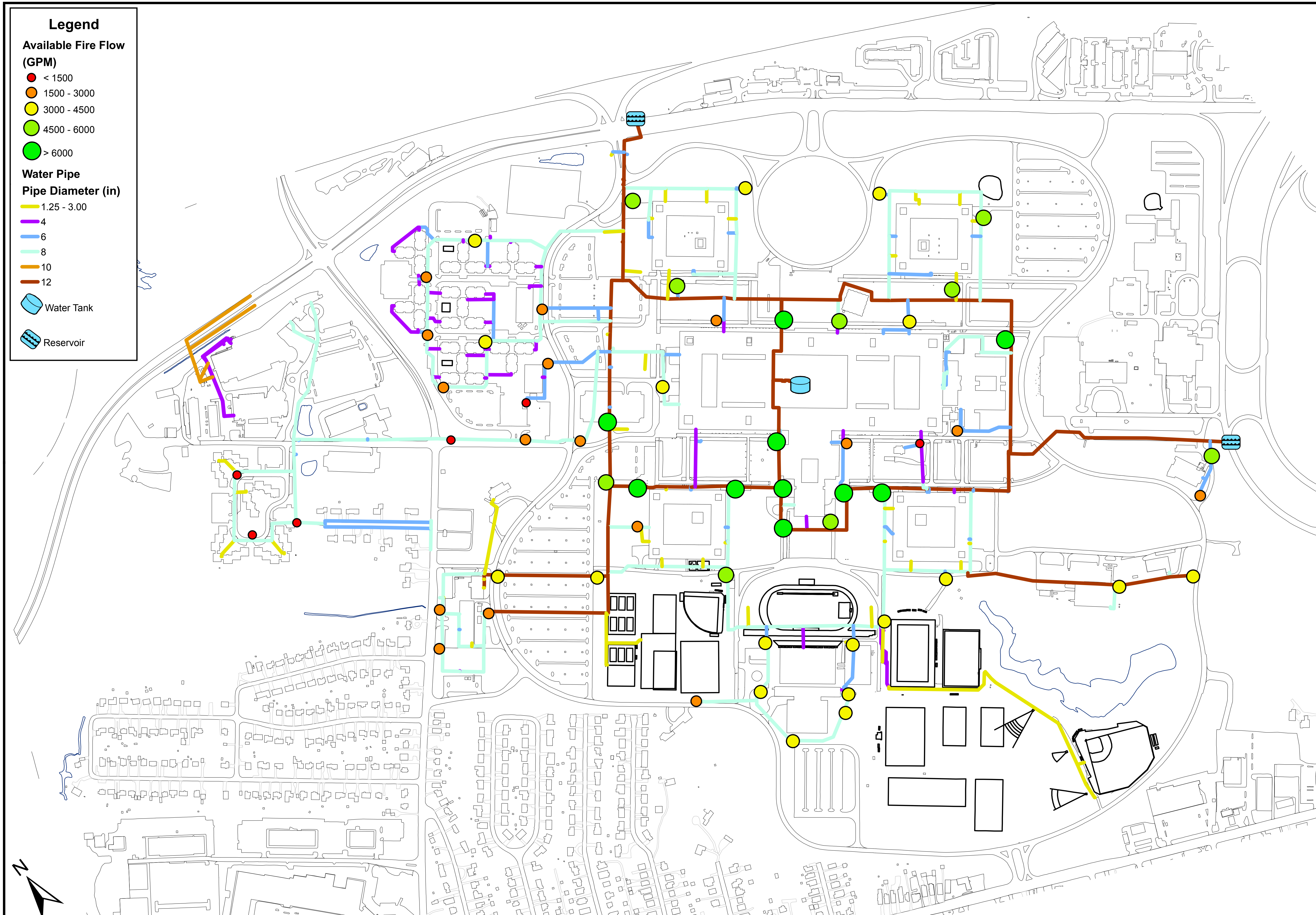
Hydraulic Grade Elevation and Pipe Flow With Secondary Source
Drawn By: AF
Checked By: SCR
University at Albany Capactiy Assessment
Feet
0 125 250 500 750 1,000
Date: 10/03/08
Job #213564
Scale: 1" = 500'
Figure: C-5



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Available Fire Flow and Pipe Diameter With Secondary Source

Drawn By: AF

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University at Albany Capacity Assessment

Feet
0 125 250 500 750 1,000

Date: 10/03/08

Job #213564

Scale: 1" = 500'

C - 6

APPENDIX D: WATER MODEL CRITICAL NODES

Hydrant Node	Hydrant Number	Area	Current Demand	Expanded Demand	Expanded Demand, Second Source
J0005	3182	Freedom Quad/Tri-Centennial	**J0008**	**J0008**	**J0008**
J0011	3184	Freedom Quad/Tri-Centennial	**J0008**	**J0008**	**J0008**
J0015	2771	Freedom Quad/Tri-Centennial	**J0008**	**J0008**	**J0008**
J0038	wHY1	Support Bldg	J0038	J0038	J0038
J0057	wHY2	Support Bldg	J0057	**J0111**	J0057
J0086	wHY3	Support Bldg	J0086	J0086	J0086
J0105	wHY4	Support Bldg	**J0111**	**J0111**	**J0111**
J0112	2446	Freedom Quad/Tri-Centennial	**J0008**	**J0008**	**J0008**
J0120	wHY225	Empire Commons	J0120	J0120	J0120
J0126	wHY5	Dutch Quad	**J0111**	**J0111**	**J0111**
J0133	1043	University Field Area	J0133	J0133	J0133
J0143	2483	Empire Commons	J0143	J0143	J0143
J0154	24	Freedom Quad/Tri-Centennial	**J0008**	**J0008**	**J0008**
J0177	wHY223	Empire Commons	J0177	J0177	J0177
J0185	wHY33	Empire Commons	J0185	J0185	J0185
J0193	wHY226	Empire Commons	J0193	J0193	J0193
J0197	wHY207	University Field Area	J0197	J0197	J0197
J0205	wHY206	University Field Area	J0205	J0205	J0205
J0207	--	Dutch Quad	J0207	J0207	J0207
J0212	wHY210	Dutch Quad	J0212	J0212	J0212
J0224	2442	Freedom Quad/Tri-Centennial	**J0008**	**J0008**	**J0008**
J0236	wHY6	Dutch Quad	J0236	**J0111**	J0236
J0241	wHY34	Empire Commons	J0241	J0241	J0241
J0249	wHY30	University Field Area	J0249	J0249	J0249
J0264	wHY9	Dutch Quad - Pod. W Lot	J0264	**J0111**	J0264
J0269	wHY7	Dutch Quad	J0269	J0269	J0269
J0270	2559	Empire Commons	**J0008**	J0270	J0270
J0293	wHY208	University Field Area	J0293	J0293	J0293
J0294	wHY215	Empire Commons	J0294	J0294	J0294

Hydrant Node	Hydrant Number	Area	Current Demand	Expanded Demand	Expanded Demand, Second Source
J0318	wHY209	University Field Area	J0318	J0318	J0318
J0368	wHY8	Dutch Quad	J0368	**J0111**	J0368
J0369	wHY31	University Field Area	J0369	J0369	J0369
J0379			J0379	J0379	J0379
J0395	wHY204	Campus Center/Sci Library	J0395	**J0111**	J0395
J0428	wHY205	Campus Center/Sci Library	J0428	**J0111**	J0428
J0431	wHY250	University Field Area	J0431	J0431	J0431
J0439	wHY202	Campus Center/Sci Library	J0439	J0439	J0439
J0444	wHY12	Campus Center/Sci Library	J0444	J0444	J0444
J0467	wHY28	Colonial Quad	J0467	J0467	J0467
J0469	wHY201	Campus Center/Sci Library	J0469	J0469	J0469
J0475	wHY26	West Pod. - Bus. Bldg.	J0475	J0475	J0475
J0484	wHY27	Colonial Quad	J0484	J0484	J0484
J0536	wHY16	Indian Quad	J0536	J0536	J0536
J0546	wHY15	Indian Quad	J0546	J0546	J0546
J0549	wHY13	Campus Center/Sci Library	J0549	J0549	J0549
J0574	wHY25	Arts and Sciences	J0574	J0574	J0574
J0609	wHY14	Chemistry	J0618	J0618	J0618
J0619	wHY24	NE Pod. - Fine Arts	J0619	J0619	J0619
J0626	wHY29	Colonial Quad	J0626	J0626	J0626
J0631	wHy221	Life Sciences	J0631	J0631	J0631
J0657	wHY23	NE Pod. - Earth Sci & Math	J0657	J0657	J0657
J0661	wHY220	Justice Dr. - Grounds Bldg.	J0661	J0661	J0661
J0691	wHY22	State Quad	J0691	J0691	J0691
J0692	wHY20	State Quad	J0692	J0692	J0692
J0699	wHY222	Life Sciences	J0699	J0699	J0699
J0705	wHY219	Justice Dr. - Police Bldg.	J0705	J0705	J0705
J0726	wHY21	State Quad	J0726	J0726	J0726
J0731	2613	Bohr Studio	**J0008**	J0731	J0731
J0736	2620	Bohr Studio	J0736	J0736	J0736
J0903	--	Indian Quad	J0627	**J0111**	J0627