CITY of NOVI CITY COUNCIL

Agenda Item G
December 22, 2014

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SUBJECT: Approval to award an engineering services agreement in the amount of $41,500 to Orchard, Hiltz, and McCliment (OHM) for professional surveying services related to the Nine Mile Road gravity relief sewer project to address capacity issues at the Park Place Pump Station.

SUBMITTING DEPARTMENT: Department of Public Services, Water and Sewer Division

CITY MANAGER APPROVAL:

EXPENDITURE REQUIRED	**$ 41,500**
AMOUNT BUDGETED	**$ 754,000**
APPROPRIATION REQUIRED	**$ 0**
LINE ITEM NUMBER	**592-592.00-974.098**

BACKGROUND INFORMATION: The Park Place Pump Station was identified in the recently completed 2014 Collection System Capacity Study as having sanitary sewage flows, both now and at full build-out, that exceed the station's capacity. A copy of the Capacity Study, which contains a map of the project area and study findings, is provided as an attachment (less appendices). As part of the Capacity Study, several alternatives were evaluated to eliminate the capacity restrictions at the pump station. A review of the rough project costs for the different alternatives to correct the capacity issues indicated construction costs of approximately $500,000 for a parallel force main and approximately $1,500,000 for a gravity relief sewer. Therefore, the Capacity Study concluded that a parallel force main from the pump station to the gravity outlet at Nine Mile and Kensington Road was the most cost effective alternative that would provide the best system redundancy.

Upon completion of the Capacity Study, the Department of Public Service's Water and Sewer Division reviewed the cost analysis of the alternatives with local underground contractors who routinely construct both force main and gravity sewers and found that the cost of providing a gravity relief sewer for the Park Place Pump Station may have been overstated. These local underground contractors indicated that a gravity sewer could potentially be constructed at $100 per foot of pipe, while the parallel force main could be constructed at approximately $70 per foot of pipe. This cost differential indicates that the City could install a gravity relief sewer at a smaller premium than originally anticipated, and given the fact that pump station upgrades would still be required at the Park Place Pump Station with the parallel force main option, the construction costs of the two alternatives could be of similar magnitude. Furthermore, the gravity relief sewer provides more redundancy for providing service to this area of the City as it does not rely on a mechanical pumping system (and a perpetually high cost of maintenance) to move the flows through the system. A gravity relief sewer would also provide more sewer system access for the users along Nine Mile Road because it would have more capacity than the existing force main.

Given the estimated costs and benefits of the gravity relief sewer alternative to service the Park Place Pump Station, the Water and Sewer Division proposes to move forward with efforts to complete a topographic survey of the route along Nine Mile Road for the proposed gravity sewer. The topographic survey will be used by the Water and Sewer Division to complete an in-house design of the proposed 7,500 foot, 12-inch relief sewer, which is expected to save the City at least $50,000 in design fees that otherwise would be paid to a consultant.

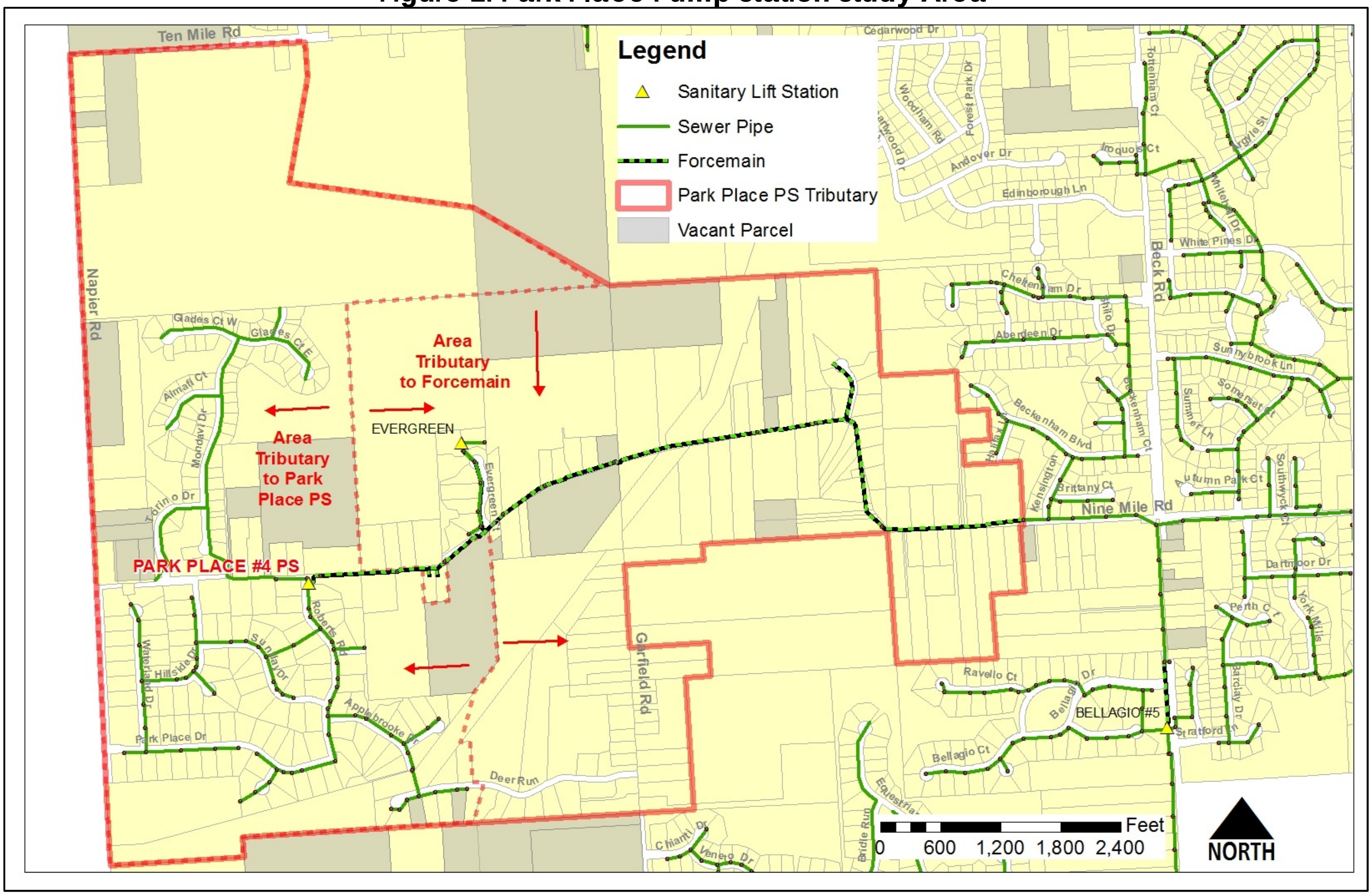
The proposed survey costs are in-line with the contract unit costs that the City currently has with the three qualified consulting engineering firms that provide civil engineering services to the City (one of which is OHM).

RECOMMENDED ACTION: Approval to award engineering services agreement in the amount of $41,500 to Orchard, Hiltz, and McCliment (OHM) for professional surveying services related to the Nine Mile Road gravity relief sewer project to address capacity issues at the Park Place Pump Station.

	1	2	Y	N
Mayor Gatt				
Mayor Pro Tem Staudt				
Council Member Casey				
Council Member Markham				

	1	2	Y	N
Council Member Mutch				
Council Member Poupard				
Council Member Wrobel				

Figure 1: Park Place Pump Station Study Area



Engineering Advisors

City of Novi

RVSDS Tributary Area Sanitary Sewer Capacity Assessment

November 7, 2014

cityofnovi.org

OHM 34000 Plymouth Road Livonia, Michigan 48150 P 734.522.6711 www.ohm-advisors.com

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I. Introduction

The City of Novi has implemented a Capacity, Management, Operation, and Maintenance (CMOM) program to manage the capacity in its sanitary sewer system. The first phase of the CMOM program was completed in 2005 and focused on growth projections, a capacity assessment, and implementation of a sewer tracking system required by the Michigan Department of Environmental Quality (MDEQ) under Part 41 Act 451 of 1994. Due to the limited outlet capacity to the Rouge Valley Sewage Disposal System (RVSDS), the effective control of peak inflow and infiltration (I/I) flows was a key objective identified for the City in the 2005 CMOM program Phase 1 Report (2005 Study). Rather than allowing the limited outlet capacity to be used for system I/I, the City established the goal of managing and controlling wet weather flows, thereby reducing the potential for sewer overflows and basement backups, while also preserving available outlet capacity for future growth.

Since 2005, the City has implemented additional Phases of the CMOM program. Phase II of the CMOM was completed in 2007 and focused on performing a sewer system evaluation survey (SSES) in the high priority areas and in evaluating improved management tools. Phase III of the CMOM was completed in 2008 and focused on the implementation of rehabilitation in a pilot area. Subsequent phases focused on different sub-areas to continue the most cost effective methods of managing and improving the system. Concurrent to CMOM implementation, some planned developments within the City were built out and are now contributing additional flows to the sanitary collection system. Due to the continuing CMOM activities and continued population growth, the City desired to update the capacity analysis of the sanitary collection system.

The key objectives of this study include:

- An evaluation of existing and future system flows including:
    - An assessment of existing flows based on flow metering data
    - An estimate of dry weather flows based on anticipated growth
    - An evaluation of design wet weather flows based on the MDEQ Sanitary Sewer Overflow (SSO) policy
- A hydraulic evaluation of the performance of the system for existing and future flow conditions.

This report details the tasks associated with these key objectives and summarizes findings.

The existing City GIS information was used to build the City hydraulic model, and used in this study. The extent of the model is identified in Figure 1 and includes select trunk sewers in the City sanitary sewer system as outlined in the initial Request for Proposal issued by the City. The hydraulic model was built in the EPASWMM 5.0 modeling platform.

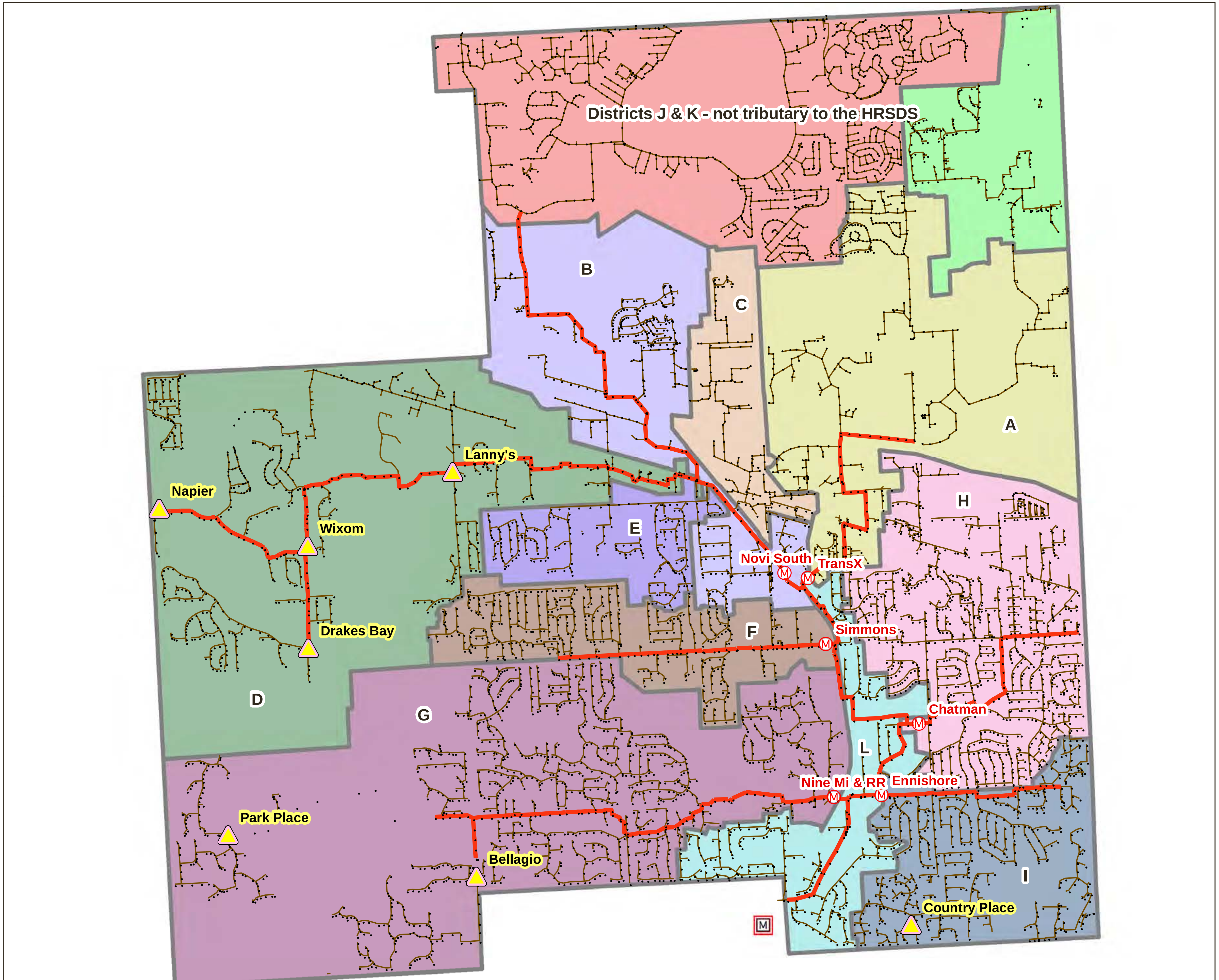
Flow metering data for both the temporary meters as well as the select pumping stations were provided by City staff.

Figure 1

Sanitary Sewer System

Flow Meters, Pump Stations &
Modeled Sewers

CITY OF NOVI
10.21.2014



Legend

Sewer Districts

- A - Hudson
- B - Husky
- C - West Oaks
- D - Lannys Road
- E - Eleven Mile
- F - Simmons
- G - Nine Mile
- H - Willow Brook
- I - Turtle Creek
- J - Walled Lake
- K - Commerce Twp
- L - Interceptor
- Outlet Meter
- Temporary Meter
- Modeled Sewers
- Select Pumping Station

0 0.45 0.9 1.35 1.8 Miles

Source: Data provided by Oakland County and the City of Novi. Orchard, Hiltz and McCliment does not warrant the accuracy of the data and/or the map. This document is intended to depict the approximate spatial location of the mapped features within the Community and all use is strictly at the user's own risk.

Coordinate System: NAD 1983 StatePlane Michigan South FIPS 2113 IntlFeet

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A total of six temporary flow meters were used as part of this study. Their locations are shown in Figure 1. In addition, flow metering data from the downstream meter on 8 Mile Road has been collected as well as data on the seven pumping stations also shown in Figure 1. Finally, several rain gages were used as part of this study, including the following:

- City of Novi Park Place
- City of Novi Department of Public Services (DPS)
- Michigan Automated Weather Network (MAWN) Commerce Township
- Oakland County #843
- Wayne County #R11
- Detroit Metropolitan Airport (DTW)

All of these rain gages, with the exception of the DTW gage, were used in identifying spatially uniform rain events, which can be used in the development of the hydrologic antecedent moisture model. For the outlet in particular, the DPS, MAWN, and #843 rain gauges were used in identifying spatially uniform rain events. The recorded rainfall from these three gauges was averaged and the standard deviation calculated on a daily basis. For a storm event to be selected,

- the average rain of the three gauges had to be great than 0.6 inches of rain and
- the ratio of the standard deviation over the average had to be less than 35%.

Some exceptions were made to these criteria due to storms lasting over multiple days and storm volumes much larger than 0.6 inches. Once the spatially uniform events were identified, which are detailed in Appendix G, the DPS rain gage became the primary source for rainfall data used for the calibration process. Any missing historic DPS data was supplemented with the MAWN rain gage data. Once the antecedent moisture model was developed and calibrated, approximately 64 years of DTW data was used to develop a frequency plot for peak flow rates.

II. Hydrology

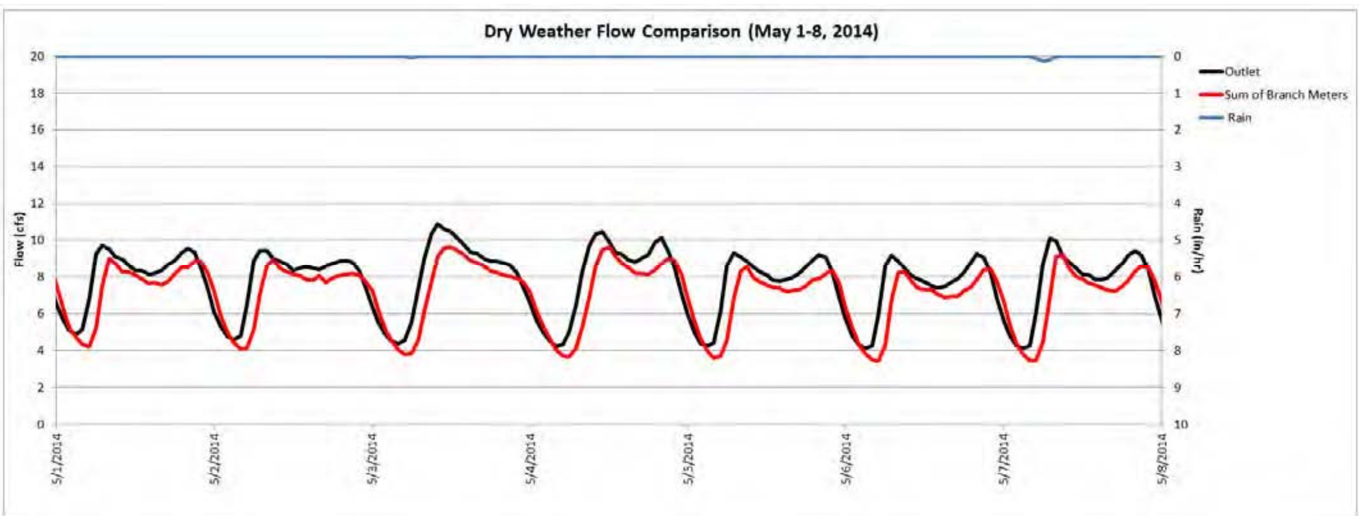
Temporary flow metering data was collected by the City at six locations near the downstream end of each major trunk sewer. In addition, flow data was collected at select pumping stations. This data was used to develop hydrologic models for each sub-area.

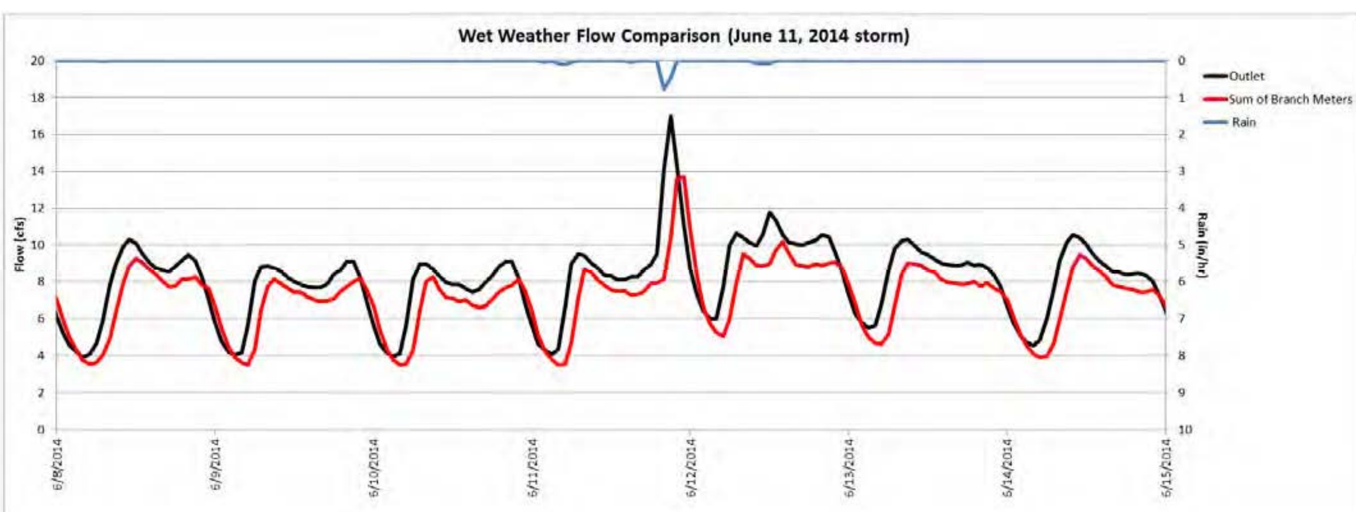
A. Mass Flow Balance

Initially, a mass flow balance was performed with these temporary meters. Flows from the temporary meters upstream were summed up and compared against the flow metering data collected at the outlet of the system at 8 Mile Road. The outlet meter on 8 Mile Road is a Palmer-Bowlus type flume with a sonic down-looker that measures depth in the flume throat. Subsequently, flow is computed from a table that relates the depth in the flume to a flow rate. This meter has continuous data from many years and is calibrated regularly. There is high confidence in the flow data at this meter. Figure 2 on the following page shows the mass flow balance comparison between the temporary meters and the outlet meter for dry and wet weather flows.

Figure 2 suggests that, during the monitoring period, the sum of the upstream temporary meters agree with the downstream outlet meter flows to within approximately 10% of the flow rate. There is a small area, the Interceptor district L, that is not included in the summation of upstream meters. Although it likely contributes less than 10% of the total system flow, the overall flow balance is considered reasonable. There are some larger deviations between the sum of the temporary meters and the outlet meter during two large storm events, one in May and the other in June. This is partly due to temporary meter drop out during these periods. Finally, there appears to be an approximate one hour time shift between the sum of the temporary meter flows and the outlet meter flow. It is likely due to a shift in time sequence (i.e. eastern standard versus daylight savings time) between the setting of the temporary flow meters and the outlet meter. Appendix A contains raw (i.e. unedited flow signals as recorded by the temporary flow meters) flow rate and velocity signals from the temporary meters.

Figure 2: Mass Flow Balance Comparison





B. Antecedent Moisture Model Development

The Antecedent Moisture Model (AMM) allows for development of a continuous hydrologic model of a system, accounting for the variation in antecedent moisture conditions. Recent rainfall and soil moisture conditions significantly affect the system response to wet weather events, and the AMM accounts for these variations.

After the development of AMM models, an accuracy of fit analysis was performed. Such an analysis includes an evaluation of model errors and quantifies model performance to determine if the model is calibrated sufficiently. The accuracy of fit compares the peak flows and volumes between the actual observed values in the system to the model predictions for several large storm events. Net average error is the average of all the errors from several storms and allows positive and negative values to offset each other. The net average error is a measure of the model bias and should be close to zero. Total average error is the average of the absolute value of the errors from several storms and is a measure of the model's ability to predict volumes and flows for individual storm events.

Flow data from the outlet, trunk sewers, and select pumping stations were used in developing antecedent moisture models. The Bellagio and Napier pumping stations had insufficient flow data to develop antecedent moisture models. For the Napier pumping station, available flow metering data was used to estimate the volume of rainfall-induced sanitary sewer flows for each significant storm event. The same was done for the Novi South meter, which is downstream of the Napier pumping station. A volumetric scaling factor was determined between the pumping station and the Novi South meter. This was applied to the design event hydrograph for the Novi South meter to approximate the design event hydrograph for the Napier pumping station. The Bellagio station flow data only consisted of peak flow rate recordings of select few storm events. For the events that Bellagio had a peak flow recording, the peak flow recordings of the temporary branch meter downstream of this station, i.e. Nine Mile & RR, were noted. A peak flow scaling factor was approximated with this information. This factor was applied to the Nine Mile & RR meter design event hydrograph in order to estimate the design event hydrograph for the Bellagio pumping station.

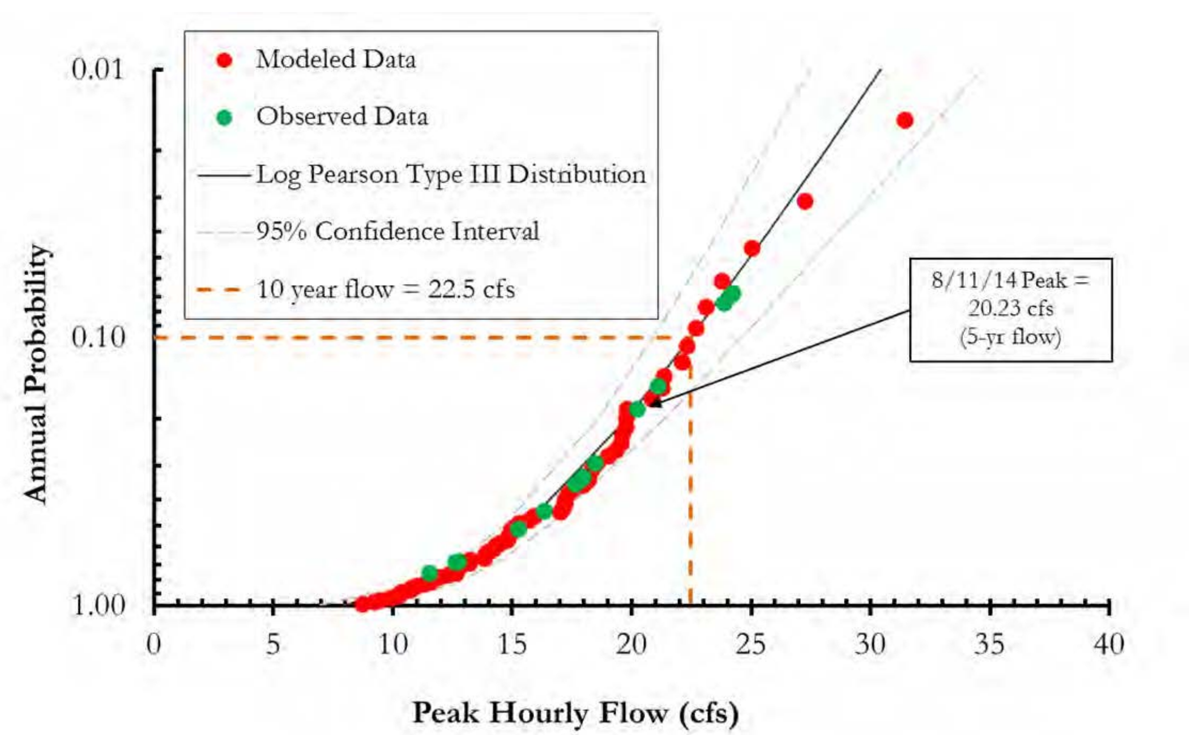
Table 1 summarizes the accuracy of fit analysis and indicates that the hydrologic models are accurately simulating wet weather peak flows and volumes within an average net error (model bias) of approximately 5% (within 1% for the outlet and branch meters) and total average errors (predictive accuracy) of 26% or less (within 19% for the outlet and branch meters). The accuracy of fit tables and plots of each storm used in model calibration can be found in Appendix B. Appendix B also contains the outlet meter statistics, including capture coefficients, for the storm events used in the outlet meter calibration process.

Recognizing the fact that the outlet is a key design point in the system, the accuracy of the outlet model was evaluated in more detail to help build confidence in the results. For example, the 10-year frequency peak design flow was estimated using two different methods to make sure both methods produced similar results. The first method was based on 15 years of actual metered flows at the outlet. The second method was based on a long-term simulation using the outlet model with 64 years of local rainfall and temperature data as input. The 10-year flow based on actual meter data was within the 95% confidence interval of the corresponding value based on the model, which provides support for the accuracy of the outlet model and can be seen in Figure 3. Also, it was noted that peak flow and volume errors of the outlet model were highest for relatively small storms and that the predictive accuracy of the model was best for relatively large storms.

Table 1: Antecedent Moisture Model (AMM) Accuracy of Fit Results Summary

Location	Peak Flow		Volume	
	Net Avg. Error	Total Avg. Error	Net Avg. Error	Total Avg. Error
Outlet	-0.9%	18.3%	0.3%	11.6%
Ennishore	0.3%	18.6%	-0.7%	9.9%
Chattman	0.4%	7.2%	0.2%	4.2%
9 Mile and Railroad Tracks	-0.7%	10%	0.7%	5.2%
Novi South	-0.2%	26%	-0.3%	9.8%
Trans X	-0.3%	15.8%	-0.4%	2.4%
Simmons	-0.5%	13.4%	0.4%	7.5%
Park Place Pump Station	-3.7%	24.4%	4.4%	15.7%
Country Place Pump Station	0.3%	14.8%	0.2%	7.2%
Drakes Bay Pump Station	-5.1%	17.8%	4.7%	15.1%
Wixom Pump Station	0.1%	17%	4.4%	9.0%
Lanny's Pump Station	-0.9%	22.1%	2.4%	18.1%
Napier Pump Station	N/A	N/A	N/A	N/A
Bellagio Pump Station	N/A	N/A	N/A	N/A

Figure 3: Outlet Meter Frequency Analysis



C. Frequency Analysis

A frequency analysis was performed by routing 64 years of historic rainfall through the calibrated AMMs. Because the process uses the continuous AMM and the historic rainfall to generate a long-term flow record, the resulting output provides information on the likelihood of various flows occurring. It also accounts for variations in rainfall amounts, rainfall pattern and various wetness conditions. This results in 64 years of predicted flow that can be used in a statistical analysis of that flow to develop a plot of the peak flow rate versus the annual probability of that flow occurring. The annual peak flow rates that occurred during the growth season (defined from April through October) were used to determine recurrence interval for flows in that sewershed using a Log-Pearson Type III Distribution. The recurrence interval estimates the likeliness that a given flow rate will occur. The average recurrence interval can be related to frequency of occurrence. For example, over a long period of time, the 10-year flow can be expected to occur with an average interval of ten years. This means there is a 10% probability of that flow being exceeded in a given year. Table 2 below provides a summary of the frequency analysis-based design event peak flow rates, i.e. flow rates for a 10% exceedance probability. In this table, base flow refers to the ground water infiltration component of the flow signal. Average diurnal, on the other hand, refers to the average of the sanitary sewer flow, which excludes base flow or rain dependent inflow and infiltration (RDII). Therefore, the total peak flow rate is the sum of the RDII and base flow and average diurnal flow components of the flow signal. Details of the frequency plots can be found in Appendix C. Table 2 also includes approximated capacities of the sections of sanitary sewers the temporary meters were directly located in as well as pump station estimated firm capacities (as obtained by the City) and outlet flow contract capacity (not physical sewer pipe capacity). As can be seen, the outlet contract capacity, Chattman Manning sewer flow capacity, and Country Place pumping station firm capacity are exceeded by the design event peak flow rate estimate.

It is important to note that even though the Chattman sewer Manning flow capacity is exceeded, the peak depth of the sewer is contained within the diameter of the sewer pipe. This can partly be explained with the fact that the Chattman sewer (which is 21 inches in diameter) discharges into a 42" downstream interceptor sewer, which has much more capacity. This sewer pipe combination allows the Chattman sewer where the temporary meter is located at, to carry more flow than it otherwise could (Manning flow capacity).

Table 2: Design Event Peak Flow Rates Summary

Location	10-year Frequency Peak Flow Total (cfs)	10-year Frequency RDII (cfs)	Baseflow (cfs)	Average Diurnal Flow (cfs)	Capacity* (cfs)
Outlet**	22.5	15.67	3.77	3.10	20.5
Ennishore	4.3	3.47	0.39	0.43	10.8
Chattman**	7.9	6.33	1.00	0.55	6.9
9 Mile and Railroad Tracks	4.9	3.92	0.42	0.52	8.5
Novi South	4.5	2.90	0.72	0.89	54.5
Trans X	2.2	1.10	0.70	0.43	21.9
Simmons	1.6	1.32	0.04	0.28	2.2

Park Place Pump Station**	0.19	0.14	0.02	0.03	0.16
Country Place Station**	1.28	1.04	0.10	0.14	1.18
Drakes Bay Pump Station	0.69	0.56	0.05	0.08	0.89
Wixom Pump Station	2.80	2.36	0.15	0.30	3.12
Lanny's Pump Station	3.00	2.48	0.20	0.33	7.92***
Napier Pump Station	0.71	0.51	0.05	0.15	2.93
Bellagio Pump Station	0.12	0.08	0.02	0.02	1.12

* Pipe, Pump, or Outlet Capacity
** 10-yr Frequency Peak Flow exceeds capacity
*** Includes estimated relief sewer capacity

D. Historic to Current Data Comparison

The City had conducted a CMOM study in 2005 for the purposes of understanding available capacity and potential future capacity needs in their sanitary sewer system. Some simple comparisons were made of the information from this and the previous study. The population of the City in the 2005 study, for example, was estimated to be 51,970. Currently (as reported by SEMCOG as the City's estimated July 2014 population) the City population is estimated at 60,290, suggesting a 14% increase.

Temporary flow meter data was also available from the 2005 study. In an effort to evaluate change in flows between the previous and current study, meter data average flow rate comparisons were performed. They are summarized in Table 3 for the common meters (same location of temporary meters between the two studies).

As can be seen from this table, the difference in average flow between the branch meters as well as the outlet meter is relatively small (a reduction of approximately 6% at the outlet), suggesting that the average flow response of the system has not changed significantly over time.

Table 3: Temporary Meter Average Flow Rate Comparison

Location	2014 Study (cfs)	2004 Study (cfs)	Difference (cfs)
Outlet	6.87	7.3	-0.43
Ennishore	0.82	0.89	-0.07
Chattman	1.55	1.62	-0.07
9 Mile and Railroad Tracks	0.94	1.11	-0.17
Trans X	1.13	1.00	0.13

Considering that significant development has occurred since 2005, this may be surprising. The CMOM work that the City has done may be potentially responsible for keeping flows consistent. Reductions in per capita usage and variations in climatological conditions could also explain these observations.

E. Comparison to 10 year – 1 hour Dormant & 25 year – 24 hour Growth

The design hydrograph was developed by identifying normal wetness conditions in Bulletin 71 and comparing the monthly rainfall to the Novi DPW monthly totals. The design storm rainfall were found to be 3.9 inches for the 25-yr, 24-hr growth period and 1.8 inches for the 10-yr, 1-hr design dormant period. These design storm rainfalls were placed in the antecedent moisture model to compare to the 10-yr frequency flow. The storm placements and design hydrographs can be found in Appendix H. The resulting hydrograph peaks closely matched the 10-yr frequency, validating our results. The 8/5/2008 design storm hydrograph was then used to scale to match the 10-yr frequency flow and resulted in the final hydrograph.

III. Inflow & Infiltration Analysis

Inflow and infiltration (I/I) metrics were computed to determine if some areas are showing high rates of I/I. The 2014 monitoring period was used in the analysis, which included seven storms with 0.7" or more rainfall accumulation. The I/I metrics included average dry weather flows unitized by population equivalency, wet weather peaking factors (wet weather peak flow divided by average dry weather flow), and capture coefficients (percent of rainfall captured as I/I over a specific drainage area). The flow rates were unitized in this way to eliminate the size of each meter district, which provides a basis to compare I/I rates between all meter districts, regardless of size. For example, a relatively small meter district may discharge a relatively small peak flow compared to a larger meter district, but the smaller meter district may have a much higher rate of I/I on a per person basis, which would be reflected by a relatively high peak per capita flow rate.

I/I rates were compared to Environmental Protection Agency (EPA) benchmarks for non-excessive I/I rates. The EPA benchmarks are per capita flow rate thresholds, below which I/I are considered non-excessive. If a sub-district is above the EPA benchmarks, it does not necessarily mean that the I/I are excessive, it means a cost-effectiveness analysis is needed to determine if the I/I is excessive or not. A cost effectiveness analysis involves weighing the transport and treatment costs versus I/I removal costs. If the transport and treatment costs outweigh the I/I removal costs, the I/I is considered excessive.

Table 4 summarizes the results of the I/I analysis. As can be seen from this table, only two districts (Trans X metering district collecting flows from the Hudson sewer district and the Interceptor district) showed relatively high groundwater infiltration (>120 gpcd), based on the EPA benchmark associated with the 7-14 day average high groundwater flow. However, the flow rates for these two districts were only marginally larger than the EPA guideline (larger by approximately 10%), which suggests that all of the sub-districts showed reasonably low infiltration rates. The analysis suggested that no district indicated excessive inflow response (>275 gpcd total daily average storm flow), even though the system experienced rain events larger than 1.5" during the 2014 flow monitoring period. For reference, a 25-year, 24-hour design event would have a rain volume of 3.9 inches.

Table 4 indicates that all of the sub-districts had similar, reasonably-low wet weather peaking factors (near 2.0), which supports the findings based on the EPA benchmarks. For the seven storms analyzed, the average of the peaking factors ranged from 1.8 to 2.2 for all sub-districts, with the exception of the Chattman branch, which had relatively low peaking factors (1.4 on average).

Table 4 indicates that all of the sub-districts had reasonably-low capture coefficients (0.23% to 0.75%), which also support the findings based on the EPA benchmarks. The highest capture coefficients were observed in the Chattman branch (0.75% on average) and the TransX branch had the lowest captures (0.23% on average).

This analysis indicates that the City is doing a good job of managing the sanitary sewer system through the CMOM program.

Table 4: Wet Weather Flow Analysis Summary

Inflow & Infiltration Summary Statistics ~ April 1, 2014 through July 15, 2014 Monitoring Period

Sewer District	Acreage	Population Equivalency	Average Dry Weather Flow (May 1-7, 2014)	
			(cfs)	(gallons/capita/day)
Branch #1 - Novi South of Railroad	2,570 acres	10,638 people	1.70 cfs	103 gpcd
Branch #2 - Chattman	1,710 acres	11,520 people	1.79 cfs	101 gpcd
Branch #3 - 9-Mile and Railroad	3,000 acres	9,280 people	1.04 cfs	72 gpcd
Branch #4 - TransX	1,620 acres	5,820 people	1.20 cfs	133 gpcd*
Branch #5 - Ennishore	660 acres	6,570 people	0.89 cfs	88 gpcd
Branch #6 - Simmons	880 acres	4,075 people	0.45 cfs	71 gpcd
Incremental (Outlet minus sum of branch meters)	852 acres	3,586 people	0.71 cfs	128 gpcd*
Outlet	11,292 acres	51,489 people	7.78 cfs	98 gpcd

*7-14 day high groundwater wastewater flow less than 120 gpcd indicates non-excessive infiltation based on EPA Handbook: Sewer System Infrastructure Analysis and Rehabilitation (EPA 625/6-91/030, 1991). Above 120 gpcd, a cost-effectiveness analysis is needed to determine whether or not infiltration is excessive.

Rain Gauge	Rain Events 0.7" or Greater							Average Rain Event (in.)
	4/29/2014	5/12/2014	5/28/2014	6/2/2014	6/11/2014	6/18/2014	7/8/2014	
Novi DPW	0.76"	1.66"	0.88"	0.70"	1.62"	1.75"	0.82"	1.17"
Novi Park Place Pump Station	0.69"	1.07"	0.76"	0.86"	1.00"	1.99"	0.75"	1.02"

Sewer District	Peak Wet Weather Flow (cfs)							Average Peak Flow (cfs)
	4/29/2014	5/12/2014	5/28/2014	6/2/2014	6/11/2014	6/18/2014	7/8/2014	
Branch #1 - Novi South of Railroad	3.01	4.69	2.47	3.09	3.09	2.67	3.26	3.18
Branch #2 - Chattman	2.53	3.25	3.08	3.13	4.02	3.96	3.09	3.29
Branch #3 - 9-Mile and Railroad	1.74	2.13	1.55	2.37	2.89	3.00	1.82	2.21
Branch #4 - TransX	1.37	2.37	1.57	1.47	1.63	1.49	1.71	1.66
Branch #5 - Ennishore	1.37	1.61	1.42	1.82	1.86	2.55	1.57	1.74
Branch #6 - Simmons	0.85	1.29	0.60	0.90	1.02	1.25	0.68	0.94
Incremental (Outlet minus sum of branch meters)	0.97	2.15	1.06	1.16	2.48	1.62	0.58	1.43
Outlet	11.85	17.49	11.73	13.94	16.98	16.52	12.72	14.46

Sewer District	Wet Weather Flow Peaking Factor (Peak Wet Weather Flow / Average Dry Weather Flow)							Avg WWF Peaking Factor
	4/29/2014	5/12/2014	5/28/2014	6/2/2014	6/11/2014	6/18/2014	7/8/2014	
Branch #1 - Novi South of Railroad	1.8	2.8	1.5	1.8	1.8	1.6	1.9	1.9
Branch #2 - Chattman	1.4	1.8	1.7	1.7	2.2	2.2	1.7	1.8
Branch #3 - 9-Mile and Railroad	1.7	2.0	1.5	2.3	2.8	2.9	1.8	2.1
Branch #4 - TransX	1.1	2.0	1.3	1.2	1.4	1.2	1.4	1.4
Branch #5 - Ennishore	1.5	1.8	1.6	2.0	2.1	2.8	1.8	1.9
Branch #6 - Simmons	1.9	2.9	1.3	2.0	2.3	2.8	1.5	2.1
Incremental (Outlet minus sum of branch meters)	1.4	3.0	1.5	1.6	3.5	2.3	0.8	2.0
Outlet	1.5	2.2	1.5	1.8	2.2	2.1	1.6	1.9

Sewer District	RDII Volume (1000's of cubic feet)							Avg RDII Vol.
	4/29/2014	5/12/2014	5/28/2014	6/2/2014	6/11/2014	6/18/2014	7/8/2014	
Branch #1 - Novi South of Railroad	28	NA	NA	16	28	46	NA	30
Branch #2 - Chattman	17	NA	51	29	110	72	NA	56
Branch #3 - 9-Mile and Railroad	49	NA	28	24	77	NA	11	38
Branch #4 - TransX	NA	46	3	0	49	19	6	21
Branch #5 - Ennishore	NA	NA	15	12	15	NA	9	13
Branch #6 - Simmons	8	26	NA	4	18	22	NA	16
Incremental (Outlet minus sum of branch meters)	NA	NA	NA	NA	NA	NA	NA	NA
Outlet	263	534	135	143	430	600	132	319

Sewer District	Capture Coefficient (RDII Volume / Rainfall Volume)							Avg Capture Coefficient
	4/29/2014	5/12/2014	5/28/2014	6/2/2014	6/11/2014	6/18/2014	7/8/2014	
Branch #1 - Novi South of Railroad	0.40%	NA	NA	0.25%	0.19%	0.28%	NA	0.28%
Branch #2 - Chattman	0.36%	NA	0.94%	0.68%	1.10%	0.66%	NA	0.75%
Branch #3 - 9-Mile and Railroad	0.60%	NA	0.29%	0.31%	0.43%	NA	0.12%	0.35%
Branch #4 - TransX	NA	0.47%	0.06%	0.00%	0.52%	0.19%	0.13%	0.23%
Branch #5 - Ennishore	NA	NA	0.70%	0.73%	0.38%	NA	0.47%	0.57%
Branch #6 - Simmons	0.33%	0.50%	NA	0.18%	0.36%	0.39%	NA	0.35%
Incremental (Outlet minus sum of branch meters)	NA	NA	NA	NA	NA	NA	NA	NA
Outlet	0.84%	0.78%	0.38%	0.50%	0.65%	0.84%	0.39%	0.63%

Sewer District	Total Daily Average Storm Flow (cfs)							Avg (cfs)
	4/29/2014	5/12/2014	5/28/2014	6/2/2014	6/11/2014	6/18/2014	7/8/2014	
Branch #1 - Novi South of Railroad	1.90	NA	1.82	1.77	1.82	1.85	1.93	1.85
Branch #2 - Chattman	1.87	1.96	2.07	1.86	1.92	2.45	1.83	2.00
Branch #3 - 9-Mile and Railroad	1.12	1.16	1.10	1.13	1.15	1.46	1.10	1.18
Branch #4 - TransX	1.17	1.31	1.18	1.13	1.15	1.25	1.19	1.20
Branch #5 - Ennishore	0.92	0.91	1.01	0.92	0.93	NA	1.07	0.96
Branch #6 - Simmons	0.49	0.55	0.37	0.48	0.43	0.54	0.36	0.46
Incremental (Outlet minus sum of branch meters)	0.70	3.52	0.83	0.79	1.27	2.89	0.62	1.52
Outlet	8.17	9.42	8.38	8.10	8.67	10.44	8.12	8.76

Sewer District	Total Daily Average Storm Flow (gpcd)**							Avg (gpcd)
	4/29/2014	5/12/2014	5/28/2014	6/2/2014	6/11/2014	6/18/2014	7/8/2014	
Branch #1 - Novi South of Railroad	116	NA	111	108	110	112	118	112
Branch #2 - Chattman	105	110	116	105	108	137	103	112
Branch #3 - 9-Mile and Railroad	78	81	76	79	80	102	77	82
Branch #4 - TransX	130	145	131	126	128	138	132	133
Branch #5 - Ennishore	91	90	99	91	92	NA	106	95
Branch #6 - Simmons	78	88	59	77	68	86	57	73
Incremental (Outlet minus sum of branch meters)	126	NA	150	142	229	NA	112	152
Outlet	103	118	105	102	109	131	102	110

**Total daily average storm flow < 275 gpcd indicates non-excessive inflow based on EPA Handbook: Sewer System Infrastructure Analysis and Rehabilitation (EPA 625/6-91/030, 1991). Above 275 gpcd, a cost-effectiveness analysis is needed to determine whether or not inflow is excessive.

IV. Future Flow Projections

SEMCOG projects no significant change in the City's population over the next 25 years. However, the City has been experiencing new developments, particularly in areas with large parcels of vacant land. Therefore, future flow projections were based on anticipated addition of REUs provided by the City for the western areas and summarized in Appendix D. In addition, the Hudson district future flow projections were performed using the REU projections made in 2005. Other areas within the City were consolidated because they are not tributary to portions of the collection system with capacity restrictions. These consolidated future projection values were obtained with the assistance of City staff and were based on vacant parcels with an assumed density of 0.8 REU per acre.

The REU values were converted to average flow rate by use of per capita flow values and persons per household (REU). Finally, all the future REU values were peaked by a peaking factor equation used in the 10-State Standards. The purpose of peaking the average flow estimates is to account for reasonable amount of I/I as well as account for a maximum day type flow generation. It was assumed that one REU equated to 2.47 capita (current SEMCOG estimate). Table 5 below summarizes the anticipated future REUs as well as approximated future flow estimates.

Table 5: Anticipated Future Growth

Nine Mile District	**Future REU**	**Capita Estimate**	**gpcd**	**gpd**	**cfs**	**Peaking Factor**	**Peak Flow(cfs)**
Park Place station + force main	254	627	102	63,993	0.099	3.9	0.39
Bellagio Station	400	988	102	100,776	0.156	3.8	0.59
Lannys Road District	**Future REU**	**Capita Estimate**	**gpcd**	**gpd**	**cfs**	**Peaking Factor**	**Peak Flow(cfs)**
Drakes Bay Station	851	2,101	114	239,568	0.371	3.6	1.32
Lanny incremental p.s. area	273	673	114	76,759	0.119	3.9	0.46
Wixom incremental p.s. area	255	630	114	71,803	0.111	3.9	0.44
Knightsbridge station (Napier)	179	442	114	50,403	0.078	4	0.31
Hudson District	**Future REU**	**Capita Estimate**	**gpcd**	**gpd**	**cfs**	**Peaking Factor**	**Peak Flow(cfs)**
Hudson	720	1,778	154	273,873	0.424	3.6	1.54
Simmons District	**Future REU**	**Capita Estimate**	**gpcd**	**gpd**	**cfs**	**Peaking Factor**	**Peak Flow(cfs)**
Simmons	38	94	114	10,700	0.017	4.3	0.07
Other Districts	**Future REU**	**Capita Estimate**	**gpcd**	**gpd**	**cfs**	**Peaking Factor**	**Peak Flow (cfs)**
Other Districts	800	1,976	103	203,528	0.315	3.6	1.13

This table suggests that an additional, approximate 3,770 REUs (or approximately 9,312 equivalent population) are anticipated in the future.

V. Hydraulic Modeling

The SWMM hydraulic model was used to assess the capacity of the City sanitary trunk sewers as well as the select pumping stations.

A. Select Pumping Stations

Figure 1 identified the pumping stations used in the hydraulic capacity assessment as part of this study. Table 6 lists these stations, their associated firm capacities (as obtained by the City), future flow projections (if applicable), and design event frequency peak flows. Note that the Park Place station includes anticipated future flows tributary to it (upstream) as well as the grinder pump connections to the force main the station discharges into (downstream).

Table 6: Pump Station Capacity Comparison

Location (Pump Station)	Firm Capacity (cfs)	Existing Design Peak Flow (cfs)	Future Growth Peak Flow (cfs)	Total Needed Capacity (cfs)	%Δ between needed and firm capacities
Park Place & force main	0.16	0.19	0.39	0.59	269 %
Country Place	1.18	1.28	n/a	1.30	10 %
Drakes Bay	0.89	0.69	1.32	2.02	127 %
Wixom	3.12	2.80	1.32(Drakes)+0.31(Napier)+0.44	4.87	56 %
Lanny's	7.92	3.00	2.07(Wixom)+0.46	5.53	-30 %
Napier	2.93	0.71	0.31	1.01	-66 %
Bellagio	1.12	0.12	0.59	0.69	-38 %

It should be noted that some pump stations flow into others, which have been accounted for in the table above. For example, both the Napier and Drakes Bay station flows converge at Wixom, which is upstream of the Lanny's pumping station. The percent difference column in Table 6 suggests that Park Place and Drakes Bay stations are anticipated to need capacity upgrades in the future. The Lanny's station operation is unique in that any flow above the station's firm capacity of 4.72 cfs discharges into a gravity overflow sewer, which has a minimum Manning (i.e. normal flow conditions) flow capacity of 3.2 cfs. Therefore, the total station capacity would be approximately 4.72+3.2 = 7.92 cfs, which results in it having a higher capacity than anticipated future growth. Therefore, in summary, it is anticipated that the following stations will need capacity upgrades in the future:

- Park Place
- Drakes Bay
- Country Place
- Wixom

B. RVSDS Outlet Capacity

As suggested earlier in this report, the anticipated existing conditions design event peak flow rate at the RVSDS outlet is 22.5 cfs. Also outlined earlier is a projection of future peak flows on a sub-division basis, which sum up to 5.12 cfs. These sub-division level flows were peaked by the 10-State Standards peaking factor formula. However, as these flows travel all the way from the sub-divisions to the outlet, attenuation is expected to occur. According to the 10-State Standards, a peaking factor for the outlet flows, considering the entire RVSDS as the tributary, would be approximately equal to two. The sum of the anticipated average future flows is approximately equal to 1.7 cfs. Therefore, the approximate peak outlet future growth would be on the order of 3.4 cfs. This would result in an anticipated future design event peak flow rate at the outlet of 22.5+3.4 = 25.9 cfs.

Another approach was taken in order to approximate the anticipated future growth induced peak design event flow rate at the outlet. This approach was based on population growth. According to SEMCOG projections at the time of this study, the City's July 2014 population was estimated at 60,290. The City's REU projections suggest an increase of approximate 9,312 equivalent population. This population increase corresponds to an approximately 15% increase from the July 2014 population estimate. An approximate 15% increase in the existing conditions peak design event flow rate of 22.5 cfs results in an anticipated future peak design event flow rate of 25.9 cfs. Both methods provide similar results and it is recommended to use 25.9 cfs as the planned future peak flow for design conditions.

Considering that the current City contract capacity to the RVSDS is 20.48 cfs, it was approximated, using the AMM design event hydrograph at the outlet, that a storage amount of approximately 0.45 million gallons would be needed. This approximation was performed in two ways:

1. A 25-year, 24-hour SCS type II rain distribution was placed on an average antecedent moisture condition month in the AMM model and the resulting hydrograph was increased by the anticipated future flow conditions to the expected peak design event. The volume above the contract capacity was then calculated, which was approximately equal to 0.45 million gallons.
2. The 64 years AMM model results were increased by the anticipated future flows, all the volumes above the current contract outlet capacity was tallied and statistically ranked. The approximate 6^{th} highest value from the top was selected, corresponding to an approximate 10% return frequency (which is the design event return frequency event) and this volume resulted in approximately 0.45 million gallons.

Details of these approximations can be found in Appendix I. It should be noted that these estimates of volume assume that the inflow and infiltration levels of future growth will remain consistent with existing inflow and infiltration levels. Therefore, the City should remain vigilant in their CMOM program to ensure this assumption remains valid.

Table 7 summarizes existing and future flows for the outlet, the temporary meters, and the pumping stations. N/A in Table 7 indicates that no future flows are anticipated in the next, approximately 20 year planning period.

Table 7: Existing and Anticipated Future Flows

Location	Average Diurnal Flow (cfs)	Base flow (cfs)	10-year Frequency RDII (cfs)	10-year Frequency Peak Flow Total (cfs)	Future Average Diurnal Flow (cfs)	Future Peak Flow (cfs)	Future Peak 10-year Frequency Flow Total (cfs)	Capacity (cfs)*
Outlet**	3.10	3.77	15.67	22.5	1.7	3.4	25.90	20.5
Ennishore	0.43	0.39	3.47	4.3	n/a	n/a	4.30	10.8
Chattman**	0.55	1.00	6.33	7.9	n/a	n/a	7.90	6.9
9 Mile and Railroad Tracks	0.52	0.42	3.92	4.9	0.099 (Park Place)+0.156 (Bellago)=0.255	0.39 (Park Place)+0.59 (Bellagio)=0.98	4.90+0.98 =5.88	8.5
Novi South	0.89	0.72	2.90	4.5	0.679 (Lannys)	2.53 (Lannys)	4.50+2.53=7.03	54.5
Trans X	0.43	0.70	1.10	2.2	0.424	1.54	3.74	21.9
Simmons	0.28	0.04	1.32	1.6	0.017	0.07	1.67	2.2
Park Place Pump Station & force main**	0.03	0.02	0.14	0.19	0.099	0.39	0.58	0.16
Country Place Station**	0.14	0.10	1.04	1.28	n/a	n/a	1.28	1.18
Drakes Bay Pump Station**	0.08	0.05	0.56	0.69	0.371	1.32	2.01	0.89
Wixom Pump Station**	0.30	0.15	2.36	2.80	0.560	2.07	4.87	3.12
Lanny's Pump Station	0.33	0.20	2.48	3.00	0.679	2.53	5.53	7.92***
Napier Pump Station	0.15	0.05	0.51	0.71	0.078	0.31	1.02	2.93
Bellagio Pump Station	0.02	0.02	0.08	0.12	0.156	0.59	0.71	1.12

* Pipe, Pump (firm capacity), or Outlet Capacity

** 10-yr Frequency Peak **Future** Flow exceeds capacity (Chattman is an exception because flow capacity is higher than Manning capacity – see report for more detail)

*** Includes estimated relief sewer capacity

C. Temporary Metering Data Hydraulic Indicators

Flow metering data can be viewed with a graphing process referred to as scatter plots. These plots put flow velocity data on one axis and flow depth data on another axis. Then, the metered data is compared against theoretical values of what the data should look like under ideal conditions (e.g. no sedimentation, blockages) using theoretical pipe friction (assuming uniform pipe slope). In addition, the metering data can be compared with a line separating flow regimes. This is important because super-critical flow regimes cannot only be difficult to model with existing EPASWMM modeling platforms but would also indicate the possibility of turbulent regime changes referred to as hydraulic jumps. Such jumps would take up capacity from the pipe, which could otherwise be used to carry sanitary sewer flow coming from the upstream stretches.

Appendix E contains scatter plots of the six temporary meters. Some of the plots also include historic data from the CMOM study for the cases in which the meter locations were the same in the 2014 study. The following observations can be made with the help of these scatter plots:

- All meter locations, with the exception of the Simmons location, suggest some level of blockage downstream, possibly due to joint misalignments, sedimentation build-up or some other obstructions. A small amount of sedimentation is not unusual in a sanitary sewer system but the data can help identify areas that need attention.
- The Novi South and Simmons meter locations suggest that the flow regimes were super-critical. The Simmons meter, in particular, showed flow velocities in excess of 8 feet per second (fps), which is high compared to typical velocities of 2 – 5 fps. The super-critical regime in this stretch of the Simmons interceptor may be due to a steeper pipe layout approximately 500 ft downstream of this meter and that after another 800 ft, this interceptor falls into the main interceptor through an approximately 10 ft drop.

Consistent with typical modeling practices, the hydraulic simulation results detailed in the subsequent sections assumed pipe capacities can be evaluated based on standard parameters. The City's cleaning and inspection program should be reviewed to see if there are any areas of unusual build-up or areas that require rehabilitation.

D. Hydraulic Modeling Simulation

The hydraulic model developed using the City GIS system, as discussed in earlier sections, was populated with existing conditions as well as future REU increased flow conditions and then simulated. The existing condition design event hydrographs were input in the upstream reaches of the individual branches, providing simulation results which tend to be on the conservative side. The future condition REU inputs were handled in the same manner. With the exception of the Hudson and Simmons districts, the future REUs were predominantly tributary to downstream pumping stations. The results are discussed below. All simulation result figures, including hydraulic profiles at select sections are contained in Appendix F.

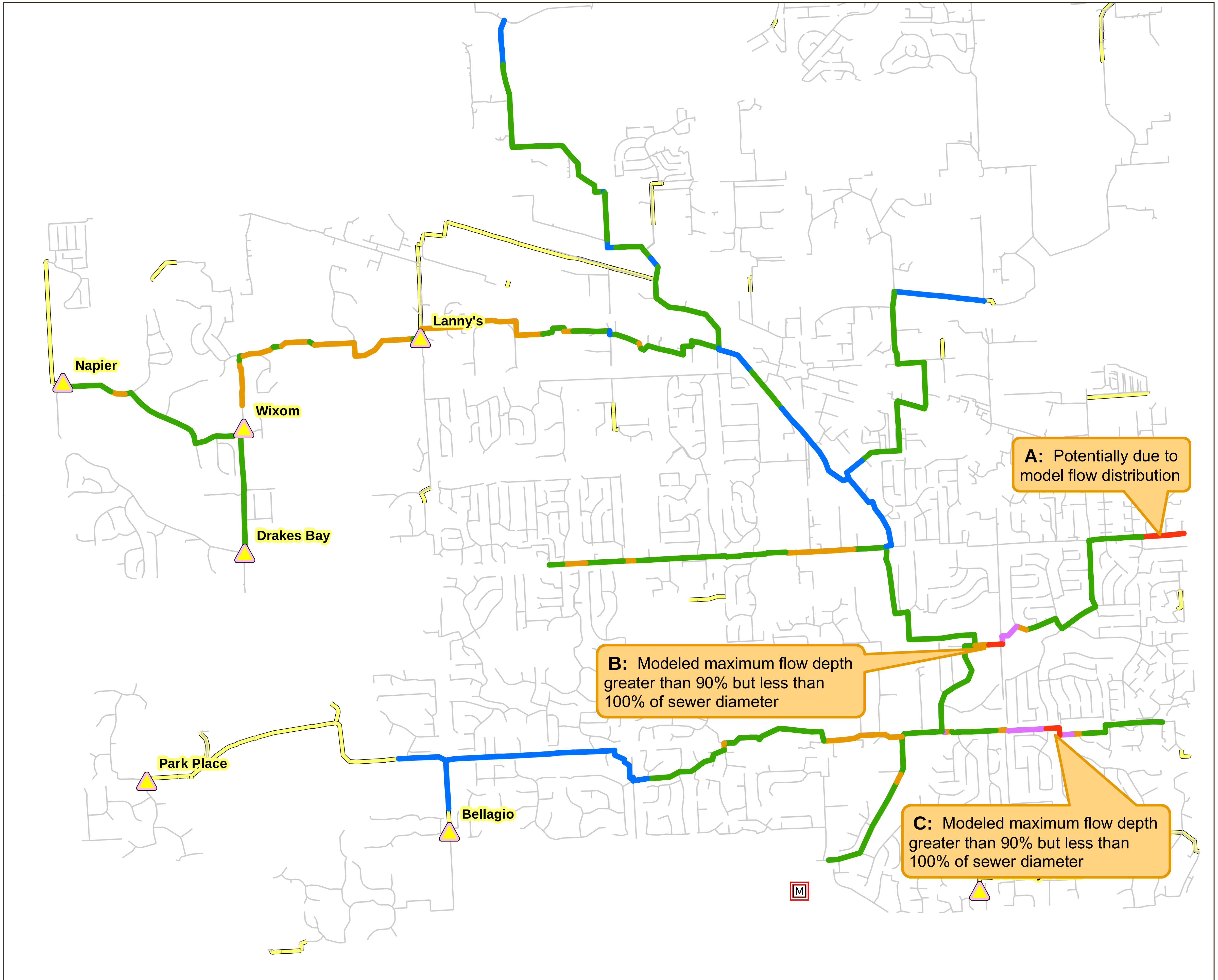
Figure 4 on the following page shows the simulated hydraulic capacity variation under existing conditions. The color grading on the legend in the figure is based on the ratio of flow area divided by the full area of the pipe. Therefore, if this ratio is 0.4, the pipe would be assumed 40% full. A 1.0 value would indicate that the pipe is 100% full and has started to become pressurized, which would suggest that the pipe is not suitable to carry the simulated peak flows under gravity flow conditions. It is also important to note that, for the purpose of being vigilant, proactive, and increase level of service delivery for its residents, the City requested the identification of sanitary sewer capacities larger than 90% full as hydraulically high risk sewers.

As can be seen in Figure 4, even though the existing conditions system would not experience surcharging during design event flows, it does include high risk hydraulic capacity sections as per the above stated criteria. A small section of sewer does surcharge – area A in Figure 4, which is upstream of the Chatman meter, in the Willow Brook district. However, as the profile for this area suggests (see Appendix F), the surcharging is small compared to the depth of the sewer and may be induced by the presumed flow distribution and not necessarily a reflection of anticipated surcharging in this reach of the system. Areas B and C are not surcharge areas but indicate hydraulically high risk stretches of the interceptor (for profiles of these areas, see Appendix F).

Figure 4
Sewer System

Existing Condition Design Event
Sewer Capacities

CITY OF NOVI
10.25.2014



Legend

Modeled Sewers

Sewer Capacities

- hmax/D <=0.25
- 0.25 < hmax/D <=0.5
- 0.5 < hmax/D <=0.75
- 0.75 < hmax/D <=0.9
- hmax/D >0.9
- Sanitary Force Main
- M Permanent_Outlet_Meter

0 0.35 0.7 1.05 1.4 Miles

Source: Data provided by Oakland County and the City of Novi. Orchard, Hiltz and McCliment does not warrant the accuracy of the data and/or the map. This document is intended to depict the approximate spatial location of the mapped features within the Community and all use is strictly at the user's own risk.

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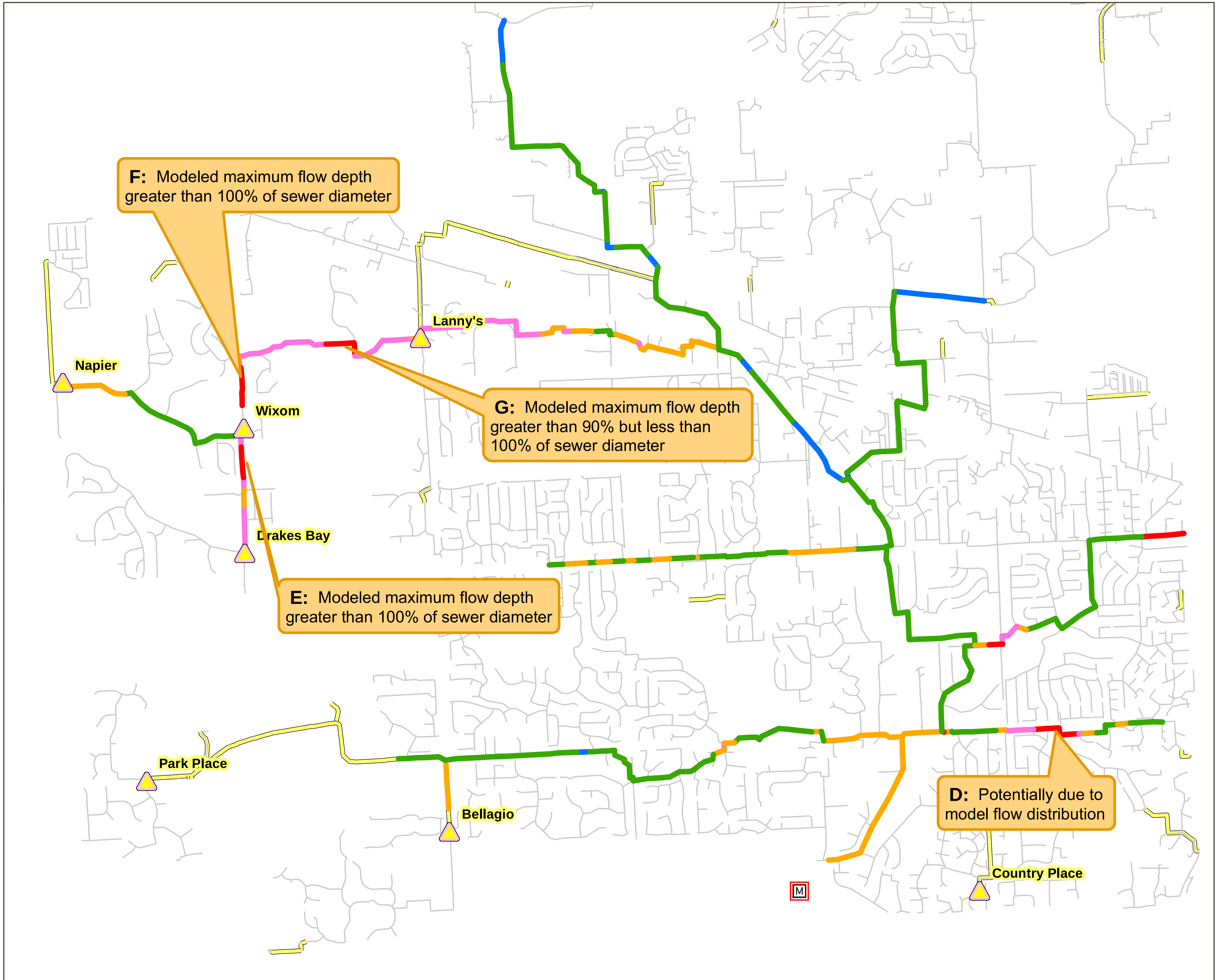
The future condition simulation results are depicted in Figure 5 on the next page. Three surcharge locations can be observed: two are upstream and downstream of the Wixom pumping station (areas E and F) and one is upstream of the Ennishore meter, i.e. in the Turtle Creek district (area C in existing conditions or area D in Figure 5). Area G, located immediately upstream of the Lannys pumping station, does not surcharge but the modeled maximum depth is greater than 90% of the sewer diameter in that area. It should be noted that Figure 5 simulation results are based on the worst case presumption of the Lannys station failing and all the flow reaching this station flowing through the overflow bypass downstream of the Lannys station. Area D may be the result of how flows were distributed in the future model rather than anticipated surcharge risk and thus would be recommended to be placed on a list of locations to monitor. In addition, it is understood that the Lanny's station was built in order to circumvent anticipated future flow capacity restrictions at the now overflow gravity line. However, the future flow model simulation performed as part of this study suggests that the gravity sewer would be able to handle the anticipated future flows without surcharging.

Figure 5

Sewer System

Future Condition Design Event
Sewer Capacities

CITY OF NOVI
10.25.2014



Legend

Modeled Sewers

Sewer Capacities

- hmax/D <=0.25
- 0.25 < hmax/D <=0.5
- 0.5 < hmax/D <=0.75
- 0.75 < hmax/D <=0.9
- hmax/D >0.9
- Sanitary Force Main
- Permanent_Outlet_Meter

0 0.35 0.7 1.05 1.4 Miles

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These hydraulic simulation results suggest the following potential future system upgrades:

- Pumping station upgrades
    - Park Place
    - Country Place
    - Drakes
    - Wixom
- Surcharged (modeled flow depth greater than sewer diameter) sanitary sewer upgrades
    - Area E
    - Area F

Table 8 approximates planning level cost estimates associated with these upgrades. The details of these approximations are contained in Appendix K. It should also be noted that a detailed study was conducted for the Park Place pumping station as per request by the City. The details of this evaluation are presented in Appendix J. As stated in the technical report in Appendix J, the City prefers an alternative, which allows for system redundancy and, which is cheaper than the alternatives evaluated. Therefore, Alternative 2 emerges as the favorable alternative. This alternative involves both the upgrading of the Park Place pumping station as well as the construction of a parallel force main.

Table 8: Planning Level Cost Estimate (PLCE) of Anticipated Improvement Needs

CIP I.D.	Size	Planning Level Cost Estimate (PLCE)
Area E	15" diameter	$430,000
Area F	21" diameter	$500,000
Par k Place Station	2x15 HP station	$910,000*
Country Place Station	2x15 HP station	$700,000
Drakes Station	2x15 HP station	$290,000
Wixom Station	2x35 HP station	$550,000
Outlet	0.3 MG storage	$1,300,000
	Total:	**$4,680,000**

* Corresponds to Alternative 2 (non-redistricting) in Appendix J and includes two 15HP pumps and new force main.

E. Re-Districting Alternative

In the future condition simulation results, it was observed that the Nine Mile district interceptor had more capacity than the interceptor upstream of the Lannys pumping station. After discussions with the City staff, it was deemed possible to divert approximately 268 REU (equating to a peak flow rate of approximately 0.38 cfs) away from the Lannys and into the Nine Mile district (upstream of the Park Place pumping station). The results of this simulation are shown in Figure 6. This simulation results in a decrease in both the extent of surcharged sewers and sewers deemed hydraulically high risk. In an effort to understand the impact on future horse power requirements of the existing pumping stations due to potentially reduced flows, an analysis was performed. This analysis took into consideration the anticipated decrease in future flows due to re-districting but also the anticipated increase in friction head requirements. It should be noted that, as per City staff input, the Drakes Bay station force main is less than approximately 10 ft in length and the Wixom station force main is only approximately 600 ft in length.

The Wixom pump station (which currently is an approximately 15 HP station, assuming a pump efficiency of approximately 60%), would, in the future conditions scenario, increase to a 37 HP station. Re-districting would decrease the HP value to approximately 33 HP. In other words, an approximately 35 HP station would be able to service both conditions and thus, would not result in an appreciable cost difference between the re-districting and non-redistricting alternatives. The details of these calculations can be found in Appendix L.

Table 9 summarizes the anticipated planning level cost estimates for this re-districting alternative.

Table 9: Planning Level Cost Estimate (PLCE) of Re-Districting Alternative

CIP I.D.	Size	Order of Magnitude Preliminary engineering Estimate (OMPEE)
Area H (equivalent to F)	21" diameter	$500,000
Park Place Station	2x15 HP station	$1,650,000*
Country Place Station	2x15 HP station	$700,000
Drakes Station	2x15 HP station	$290,000
Wixom Station	2x35 HP station	$550,000
Outlet	0.3 MG storage	$1,300,000
	Total	**$4,990,000**

* Corresponds to Alternative 2 (re-districting) in Appendix J and includes two 15HP pumps and new force main as well as wet well and vault modifications.

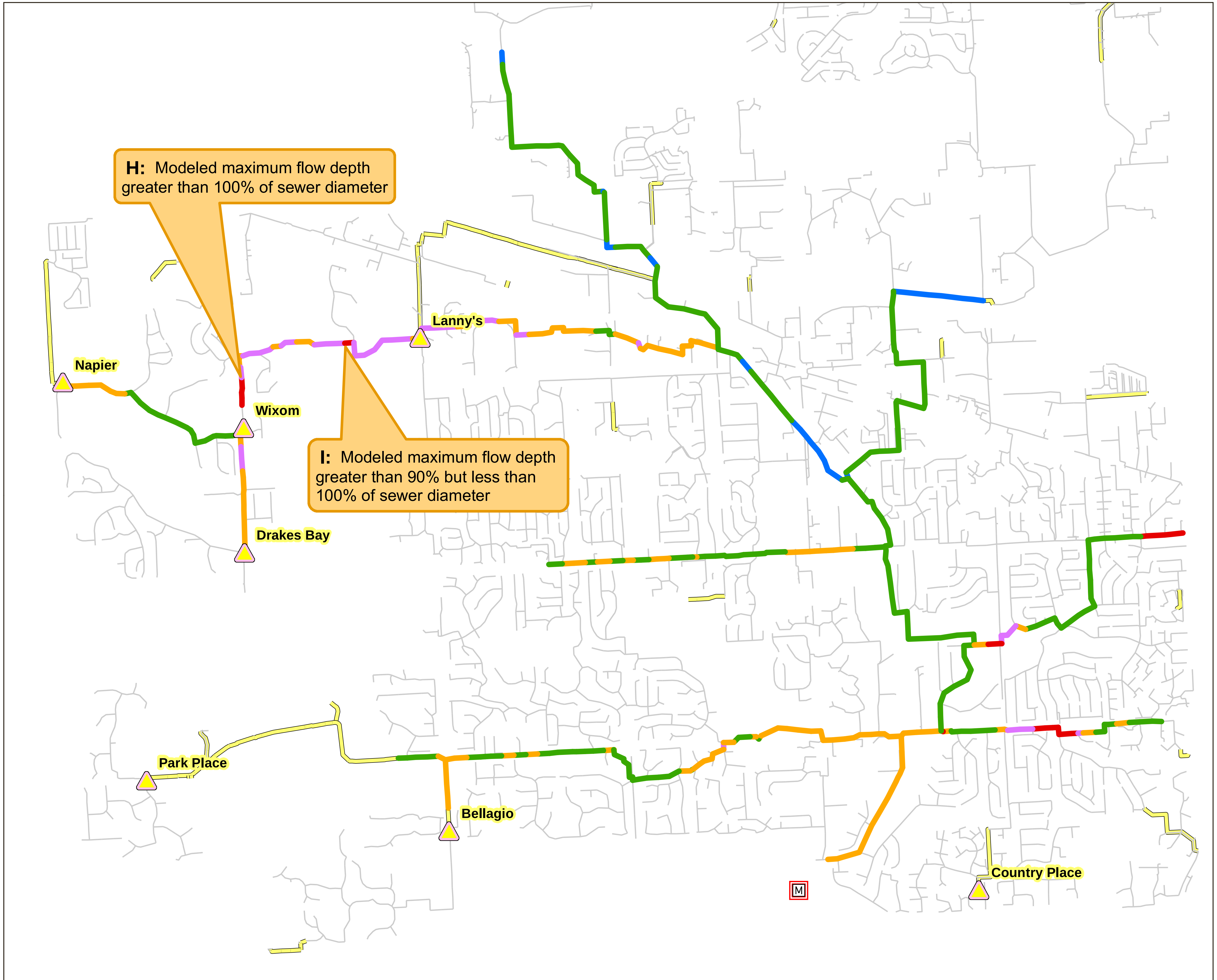

Figure 6

Sewer System

Future Re-Districting Condition
Design Event
Sewer Capacities

CITY OF NOVI
10.27.2014

Legend

Modeled Sewers

hmax_D

- hmax/D <=0.25
- 0.25 < hmax/D <=0.5
- 0.5 < hmax/D <=0.75
- 0.75 < hmax/D <=0.90
- hmax/D >0.90
- Sanitary Force Main
- M Permanent_Outlet_Meter

0 0.35 0.7 1.05 1.4 Miles

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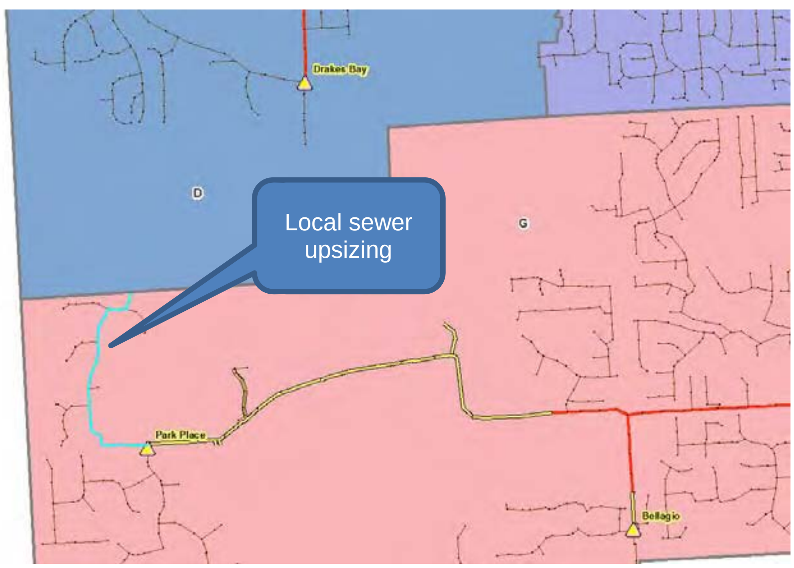
It should be noted that as part of this alternative, the existing local sewer, as shown in Figure 7, was evaluated in terms of carrying capacity for assessing the potential need to upsize this stretch of the sewer to accommodate the additional REUs to the Park Place station. As stated earlier, the additional anticipated peak flows from re-districting would result in a 0.38 cfs increase in the existing 4,000 ft, 8" diameter existing local sewer. Assuming a 0.40 percent slope for this stretch of the system (which is both the average of the sewer slopes as shown in the City's GIS as well as the minimum slope requirement for an 8" diameter sewer as per the 10-State Standards), the Manning carrying capacity of this local sewer was approximated as 0.75 cfs. Using City GIS and REU estimates from the previous CMOM study (2004), it is approximated that the existing sewer services less than approximately 168 REUs, resulting in an existing peak flow of approximately 0.33 cfs. When this flow is added to the anticipated future flow of 0.38, it results in an anticipated future flow in the existing sewer of approximately 0.63 cfs, which is less than the existing sewer carrying capacity, thus, this sewer may not need an upgrade in the future.

Figure 7: Upsizing of Local Sewer as Part of the Re-districting Alternative



The comparison between the non-redistricting and re-districting alternatives, including City staff comments, can be summarized as follows:

- The planning level cost difference between the two alternatives is small.
- If wet well and valve pit upgrades are needed at the Park Place station to accommodate re-districting flows, a larger footprint for the station could not be accommodated in the existing Park Place station easement.
- Sending re-districted flows through the Park Place station would require routing of flows through the 8" sewer in Bella Terra subdivision (Figure 7 above), which would use nearly 90% of the capacity of this sewer, potentially necessitating upsizing this sewer, especially if wet weather flows increase in the future.

VI. Key Findings

The following key findings can be drawn from this study:

1. Under current design event flow conditions, the City is anticipated to exceed its current outlet contract capacity with the RVSDS by approximately 2 cfs.

2. Under future conditions, the outlet capacity is anticipated to be exceeded by approximately 5.4 cfs.

3. A future storage facility of approximately 450,000 gallons at the outlet would allow the design peak outflow rate to stay within the current contract capacity.

4. Re-districting the Lannys and Nine Mile sewer districts in order to re-direct approximately 268 REUs of anticipated future flows to the Nine Mile sewer district may not be feasible because of possible system limitations, including easement of the Park Place pumping station and the size of the local sewer between the proposed developments and the Park Place pumping station (see Figure 7).

5. Analysis of select pumping station firm capacities indicate the Park Place and Country Place pumping stations will need capacity upgrades to handle existing flows.

6. Analysis of select pumping station firm capacities indicate the Drakes Bay and Wixom pumping stations will need capacity upgrades to handle anticipated future flows.

7. Evaluation of the system against the EPA wet weather standards suggested that the City system, for the large part, does not exhibit excessive wet weather response characteristics. This may be partly due to the City doing a good job managing the system through its CMOM program.

8. A capital improvement plan has been identified in Section VIII of this report. The Park Place pumping station is the highest priority improvement. A separate study has been initiated to look at alternatives for this area.

VII. Operational Recommendations

The following operational recommendations can be drawn from this study:

1. Perform draw down tests at all the sewage lift stations to evaluate current capacities.

2. Perform additional flow monitoring at system locations where capacity issues were identified in the hydraulic model for existing conditions. These sites include:

    i. Area "A" in the Chattman Sewer District
    ii. Area "B" in the Chattman Sewer District
    iii. Area "C" in the Ennishore Metering District
    iv. Area "G" in the Lannys Sewer District

3. Perform additional flow monitoring at the pump stations where capacity issues were noticed to confirm the multismart flow measurements at the stations.

4. The City should consider installing semi-permanent flow meters at the temporary meter locations used for this study. Historic flow information will help the City monitor future flow rates that would trigger improvements identified in future development scenarios identified in this report. In addition, the City may consider the use of non-contact meters to minimize ongoing maintenance of the meters.

5. The City should evaluate system redundancy, especially as is relates to pump stations with long force mains (> 1000 feet). Specifically, the City should focus on the Hilton, Country Place and Park Place pump stations as these have the longest force main outlets.

6. Hydraulic analysis of the temporary metering data suggested some level of either sedimentation build-up or downstream blockage at the temporary flow meter locations. The data from the City's cleaning and inspection program should be compared to the scatter plots and an evaluation prepared.

7. A long-term corrective action plan study is currently underway for the RVSDS. When the study determines costs for the regional alternatives, the City can consider participation in the regional alternative to obtain the required outlet capacity or the local storage facility outlined in this study.

8. The City should continue the CMOM Program. In areas where I/I removal takes place, the City should flow monitor pre and post to gauge the effectiveness.

VIII. Capital Improvement Recommendations

Table 10 summarizes the capital improvement projects recommended from a result of this study. Figure 8 accompanies this table. It is recommended that upgrades at the pumping station in particular be preceded by a detailed preliminary engineering and cost study. The capital improvement recommendations in this report have been based on the non-redistricting future system conditions alternative, as summarized in Table 10. Finally, the City should consider participation in a regional alternative for the RVSDS to obtain additional outlet capacity or the local storage facility, the size of which is identified in this study and summarized in Table 10.

Table 10: Capital Improvement Projects

Project ID	Description of Project	Priority and General Schedule	PLCE*
PP-1	Park Place Pump Station & force main capacity upgrades	Immediate – Pump Station over capacity	$910,000
CP-1	Country Place Pump Station & force main capacity upgrades	Immediate – Pump Station over capacity	$700,000
DB-1	Drakes Bay Pump Station capacity upgrades	Short Term (2-3yrs) Pump Station @ 80% capacity (under existing conditions design event flows)	$290,000

WXM-1	Wixom Pump Station capacity upgrades	Short Term (2-3yrs) Pump Station @ 90% capacity (under existing conditions design event flows)	$550,000
OTL-1	Outlet capacity upgrades (OMPEE based on 0.3 MG storage facility)	Short Term (2-3yrs) existing condition design event peak flow > Outlet capacity	$1,300,000
E-1	Drakes Bay effluent sewer upgrades	Medium Term (5yrs) - Surcharging @ Wixom, N of 11 Mile (~800 LF of pipe upgrade)	$430,000
F-1	Lanny's influent sewer upgrades	Medium Term (5yrs) - Surcharging @ Wixom, S of 11 Mile (~700 LF of pipe upgrade)	$500,000
		Total	**$4,680,000**

* Planning Level Cost Estimates

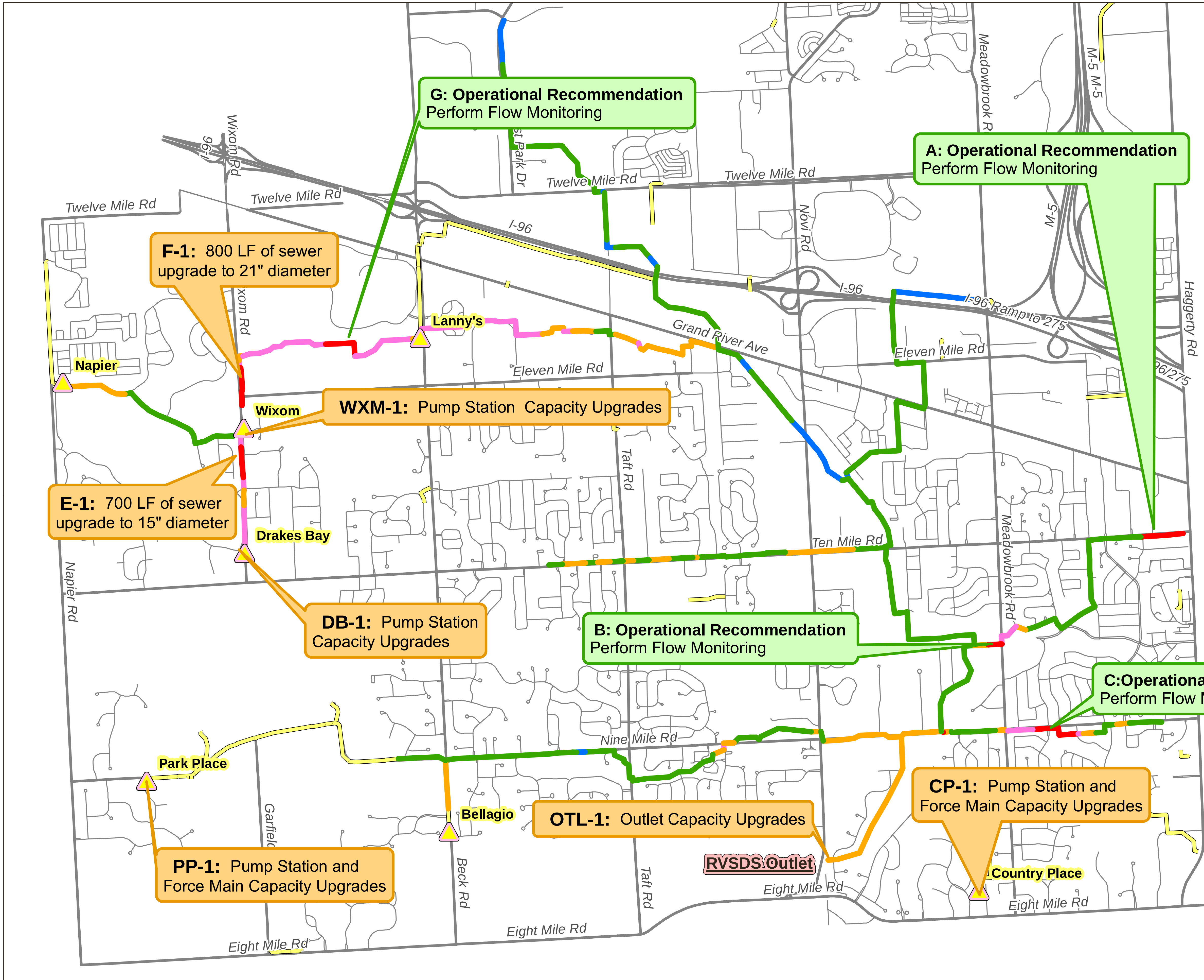


Figure 8
Sewer System
CIP Improvement Needs

Future Condition Design Event
Sewer Capacities

CITY OF NOVI
11.06.2014

Legend

Modeled Sewers

Sewer Capacities

- hmax/D <=0.25
- 0.25 < hmax/D <=0.5
- 0.5 < hmax/D <=0.75
- 0.75 < hmax/D <=0.9
- hmax/D >0.9
- Sanitary Force Main

0 0.35 0.7 1.05 1.4 Miles

Source: Data provided by Oakland County and the City of Novi. Orchard, Hiltz and McCliment does not warrant the accuracy of the data and/or the map. This document is intended to depict the approximate spatial location of the mapped features within the Community and all use is strictly at the user's own risk.

Coordinate System: NAD 1983 StatePlane Michigan South FIPS 2113 IntlFeet

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