CHAPTER 6

Conveyance System Evaluation

Chapter 6 - Conveyance System Evaluation

6.1 Introduction

The conveyance system evaluation is a follow-up to the conveyance system analysis provided in the 2006 Facilities Plan. The purpose of this evaluation is to take a more comprehensive view of the recommendations laid out in the previous report and confirm, or add to the body of information related to, the capacity of the system to carry current and projected flows throughout the Roger Road WRF (includes Randolph Park WRF) and Ina Road WRF service areas.

Unless expanded, the Roger Road treatment plant will have insufficient capacity to accommodate the future flows generated by population growth. Therefore, a major component of this study is a more detailed analysis of an interconnecting pipeline to transfer flows from Roger Road WRF to Ina Road WRF. Four routing alternatives were given consideration and a recommended route identified.

To determine effective capacity to meet CMOM requirements, a need to quantify wet weather flows had to be developed. Current sewer design calls for leaving 15 percent of each pipe's capacity available for wet weather flows. An analysis of 12 months of flow monitoring data plus some additional wet weather flow data was performed to develop a wet weather factor based on observed wet weather flows. The wet weather flow analysis described herein is not without its limitations. A forthcoming hydrologic modeling project commissioned by Pima County Regional Wastewater Reclamation Department (PCRWRD) should improve upon the wet weather flow estimates provided in this evaluation of the conveyance system.

A hydraulic sewer model was developed in a modeling program called MOUSE for routing existing and proposed flows through the conveyance system. This model is based on a complete inventory of sewers 15 inches and larger and was the primary tool in which pipe capacities were analyzed. From the analysis of the conveyance system, other than the plant interconnect pipeline, relatively few trunk and interceptor sewer replacement projects are required to accommodate dry and wet weather flows through the year 2030.

6.2 Existing Conveyance System

6.2.1 Previous Studies – 2006 Metropolitan Area Facility Plan Update

The 2006 Facility Plan outlined the conveyance needs for the Roger Road WRF and Ina Road WRF service areas over a 25-year plan planning period. The goals of the facility plan were to evaluate how growth, regulatory requirements, and system rehabilitation needs affect the system, and to develop a CIP to effectively plan for these needs. Relying on population estimates developed by the Pima Association of Governments (PAG), this report estimated that the 2030 flow that must be handled by the system will equal 85-mgd ADWF based on a rate of 85 gpcd. The 85 gpcd was derived from a joint agreement between Tucson Water and PCRWRD on per capita flows used for planning. These same population data and per capita flow rates were used in this report.

In summary, the 2006 Facility Plan report listed the following recommendations with regard to the conveyance system:

- Continue closed circuit television (CCTV) and condition assessment for over 3,100 miles of sewer lines
- Implement approved comprehensive CMOM and asset management programs
- Rehabilitate 1,500+ manholes. (More recent manhole inspections will likely increase the number of manholes requiring rehabilitation to approximately 3,000.)
- Rehabilitate portions of the system with condition assessment grades of 4 "poor", or 5 "immediate attention required"
- Construct a plant interconnect from the Roger Road WRF to the Ina Road WRF to transfer flow between these facilities
- Perform engineering studies on interceptors identified as having potential capacity issues

The last two recommendations are evaluated further in this report.

6.2.2 Existing Capacity / Condition Issues

As part of the 2006 Facility Plan, 230 miles of the interceptor system were televised in 2005. All Class 4 pipe in the interceptor system was inspected in 2006. This analysis identified several reaches of interceptor sewer in poor condition and recommends rehabilitation as soon as possible. Approximately 3,000 miles of sewer remain to be inspected and evaluated. The initial manhole condition assessment evaluation has identified 1,500 and further inspection has identified up to 3,000 manholes in need of repair.

The Facility Plan identified the Northwest Outfall (NWO) sewer to be flowing at or above 85 percent capacity and in need of immediate attention. However, this assessment may have been based on local pipes' Manning's capacities, which does not reflect the true capacity of this interceptor. An analysis of 12 months of flow monitoring data revealed that the water level in this pipe is not likely to exceed 65 percent of its diameter once every 10 years. However, as population growth within the service area continues, the portion of this pipe's capacity available for wet weather will be reduced. This will be addressed in more detail in later in this chapter.

In an effort to develop a more comprehensive understanding of the system-wide conveyance system, PCRWRD has commissioned the development of a functional and highly calibrated, hydrology based hydraulic model. The advanced model will be integrated into the conveyance system Geographic Information System (GIS) platform and will be an extension of previous modeling developments used by PCRWRD. The advanced model will afford PCRWRD with many engineering and planning tools, the three most important are: 1) ability to identify current capacity issues, 2) ability to effectively plan for anticipated growth, and 3) enables planning for inflow/infiltration improvements. The project deliverable will be a completely calibrated and validated model of one basin within the conveyance system, as well as, less detailed but calibrated and validated models of the rest of the conveyance system using InfoWorks CS model. Less detailed basins will be fully developed by PCRWRD staff after the delivery of conveyance model and receipt of training.

6.2.3 Peak / Wet Weather Flows

6.2.3.1 Objectives

Objectives of the wet weather analysis are:

- Identify the parts of the existing conveyance system that are impacted by wet-weather flows
- Estimate the peak flow rates and/or water depths at various parts of the existing conveyance system that corresponds to a hypothetical 10-year storm event (CMOM requirement).
- Develop a strategy for accounting for future wet weather flows as population and conveyance system expand.

6.2.3.2 Approach

Ideally, the system response to actual wet weather events would be evaluated via an extensive review of rainfall and flow monitoring data. Correlations between rainfall intensities / volumes and system flows would be developed, and hydrologic models prepared and calibrated to these actual events. These models are used to predict the system response to hypothetical design storms for existing and future system configurations.

For this study, a simpler approach based exclusively on flow monitoring data was utilized. This was done for several reasons. First, the isolated nature of rainfall events in the Southwest makes it challenging to derive correlations between rainfall and flow for large sewersheds. Second, it was beyond the scope of this study to perform a comprehensive wet weather analysis. Finally, the high volume (one year) of continuous flow monitoring data that is available makes the probabilistic method described below a suitable approach to make an assessment of current and previous conveyance system work.

6.2.3.2.1 Probabilistic Method

Historical (one-year) flow rates and water depth monitoring data, recorded every minute, at 27 locations in the conveyance system were analyzed using the probabilistic method described below.

- 15-minute average values of water depth and flow rate at each site were calculated from the 1-minute monitoring data. The daily peak 15-min values of water depth and flow rate at each site were tabulated.

- Frequency analyses were conducted to each site's daily peak values. The resultant distribution is a means of estimating the likelihood that a particular flow rate/depth might be exceeded on any given day. This method provided a basis for determining whether a site is impacted by wet weather flow during this one-year observation period.

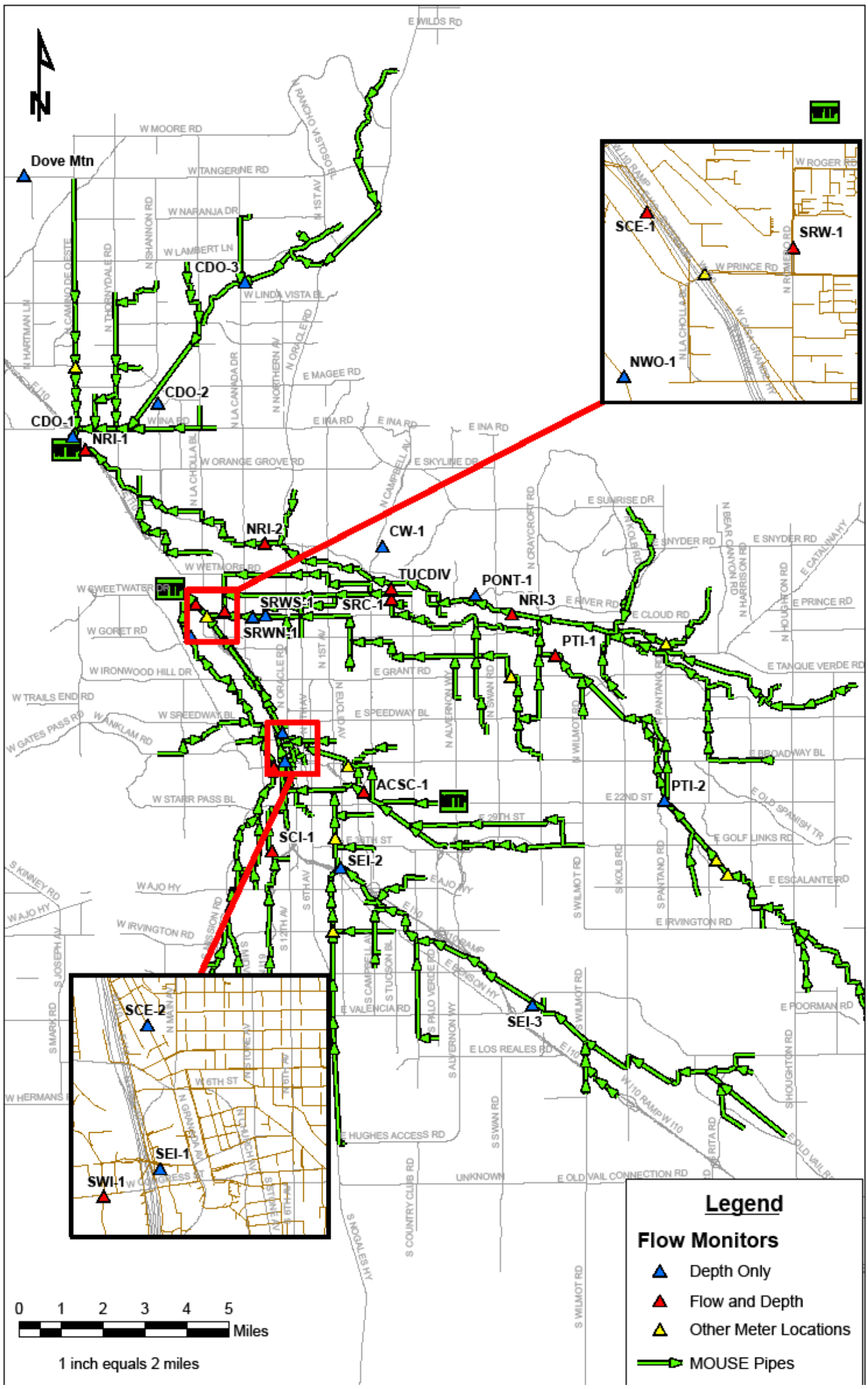
- The frequency analysis data at each site, presented as flow rate/water depth versus recurrence interval, were numerically fit using non-linear regression models. The best fit models were extrapolated to estimate the peak flow rates/water depths at a 10-year recurrence interval. The 10-year frequency is consistent with the regulatory requirements for wet weather flows. The outcome provides a basis for site-specific peaking factors which incorporate the peak 10-year flows.

6.2.3.2.2 Flow Data Summary

The data set used in this analysis was provided by PCRWRD and covers the period from July 1, 2005 to June 30, 2006. The data include depth readings at all 27 monitoring sites. Thirteen sites also recorded flow rates. Flow rates are calculated values derived from measured estimates of flow velocity and water depth, and therefore should not be considered true measurements, because of errors and inaccuracies resulting from the limitations of flow monitoring equipment. Locations of these monitoring sites are shown on **Figure 6-1**, which indicates a fairly good coverage of the existing conveyance system.

Data readings were logged at one-minute intervals resulting in more than 14 million of water depth data points and more than 6 million flow rate data points. During the analysis, abnormal data points were checked and questionable data points were excluded. Questionable data is documented.

Figure 6-1
Flow Monitoring Locations



6.2.3.3 Results

Results summarized at each site include:

- *1-year 10, 50, 90 and 100 Percentile Flow Rates*– The 10-percentile value means that 10 percent of the recorded flow rates are equal to or less than that value. The 100-percentile value is the maximum flow rate ever observed during the one-year period.

- *Calculated 10-year Flow Rate*– Values extrapolated from regression models. These values are considered to be the peak 15-min flow rates / depths likely to occur once every 10 years.

- *Design Capacity at Each Site* – Values taken from the previous study and included in this table as a reference. However, these values are merely the Manning's capacities derived from local pipe size and slope, and do not necessarily represent the pipes' true carrying capacities. More accurate design capacities should be those based on the hydraulic model developed in the course of this study

- *Wet weather Flow Rate* – Values calculated as the difference between the 10-year flow rates and the 50-percentile, or median, flow rates.

- *Wet Weather Peaking Factor* – Values calculated as the ratio of the 10-year peak flow rate to the 50-percentile peak flow rate, multiplied by a factor of 1.4. The factor was derived from an evaluation of flow monitoring data in which the typical peak dry weather flow is approximately 1.4 times the average daily dry weather flow (Q_{ADF}). Development of the factor is as follows:

$Q_{50\%}$ = Median Peak Daily Flow
PF_{DW} = Dry Weather Peaking Factor
PF_{WW} = Wet Weather Peaking Factor
From flow monitoring data, typical PF_{DW} = 1.4, so,

$$PF_{DW} = 1.4 = \frac{Q_{50\%}}{Q_{ADF}}$$

Therefore, $Q_{ADF} = 0.71 Q_{50\%}$

So, $$\boxed{PF_{WW} = \frac{Q_{10-yr}}{Q_{ADF}} = \frac{Q_{10-year}}{0.71 Q_{50\%}} = 1.4 \frac{Q_{10-yr}}{Q_{50\%}}}$$

- *12-month 10, 50, 90 and 100 Percentile Water Depths* – Presented as water depth to pipe diameter ratios at each site

- *Calculated 10-year Water Depth* – Presented as water depth to pipe diameter ratio at each site

Summaries of the results and observations for each site are discussed in the remainder of this chapter. Flow and depth figures can be found in **Appendix H**.

The 10-year wet weather peaking factors indicated in **Table 6-1** were plotted against the standard sewer design peaking factor curve (**Figure 6-2**). For the upstream sites (those with flows less than 10 mgd), there is a strong correlation that produces a curve similar in shape to the design standard, but with an increased peaking factor. Sites further downstream, North Rillito Interceptor (NRI)-2, South Rillito Interceptor-Central (SRC)-1, NRI-1, Santa Cruz-East Interceptor (SCE)-1, are on older parts of the system and are unique cases necessitating different techniques for estimating future wet weather flows. However, for those sites with Q_{ADF} less than 10 mgd, the revised curve would seem to be a viable method for evaluating system capacity to convey peak wet weather flows.

Table 6-1
Historical and Calculated Flow Rates and Water Depths

Site	Flow Rate, mgd					Design Capacity[2], mgd	Wet Weather Flow[3]	PF[4]	Water Depth/Pipe Diameter					Pipe Diameter (inch)
	Historical[1]				Calculated				Historical[1]				Calculated	
	10%	50%	90%	100%	10-year				10%	50%	90%	100%	10-year	
ACSC-1	2.69	3.56	4.60	8.77	10.19	19.2	6.63	4.00	0.22	0.26	0.29	0.42	0.51	42
AV-1	1.10	1.45	1.87	3.28	3.93	9.1	2.48	3.80	0.30	0.34	0.38	0.53	0.57	24
CDO-1	-	-	-	-	-	94.8	-	-	0.29	0.32	0.36	0.39	0.43	48
CDO-2	-	-	-	-	-	31.1	-	-	0.24	0.25	0.28	0.33	0.38	36
CDO-3	-	-	-	-	-	-	-	-	0.27	0.31	0.33	0.48	0.49	24
CW-1	-	-	-	-	-	-	-	-	0.19	0.20	0.21	0.31	0.25	12
Dove Mtn	-	-	-	-	-	-	-	-	0.16	0.17	0.20	0.21	0.22	15
GV-1	1.26	1.69	2.34	2.82	3.21	-	1.52	2.66	0.34	0.39	0.46	0.51	0.50	21
NRI-1	16.96	19.40	25.54	30.82	30.56	38.3	11.16	2.21	0.54	0.56	0.66	0.74	0.76	42
NRI-2	12.82	15.32	20.04	32.40	33.29	-	17.97	3.04	0.45	0.50	0.56	0.68	0.73	39
NRI-3	8.19	9.49	10.10	11.93	11.78	20.1	2.29	1.74	0.33	0.36	0.37	0.41	0.46	33
NWO-1	-	-	-	-	-	28.7	-	-	0.48	0.52	0.55	0.60	0.63	48
PONT-1	-	-	-	-	-	16.18	-	-	0.27	0.31	0.33	0.44	0.35	12
PTI-1	9.37	10.57	11.66	14.44	12.87	29.8	2.30	1.71	0.36	0.38	0.40	0.44	0.46	36

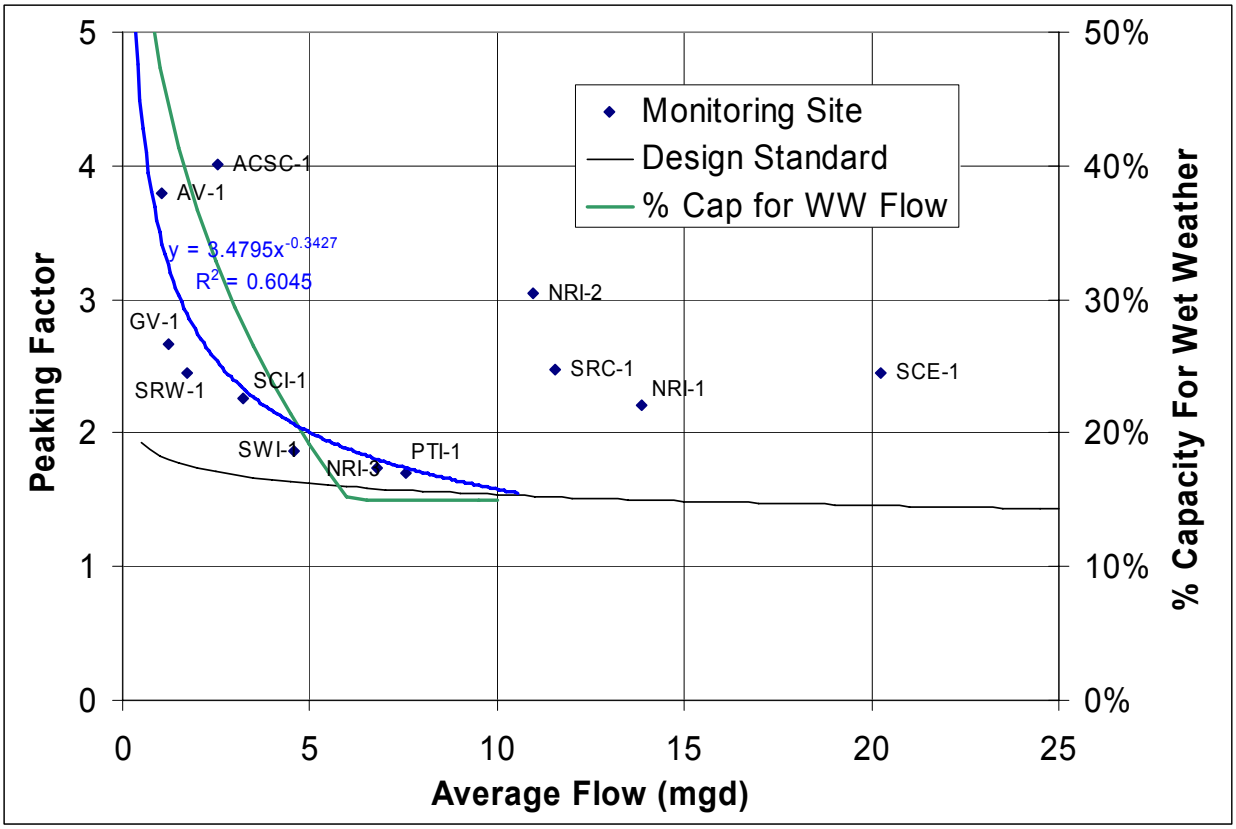
(1) Based on data recorded from July 1, 2005 to June 30, 2006
(2) From previous study: 2006 Metropolitan Area Facility Plan Update. Included here for reference only
(3) Wet weather flow, estimated as the difference between the calculated 10-year flow and historical 50-percentile flow
(4) PF: Peaking factor, estimated as 1.4 x (calculated 10-year flow/ historical 50-percentile flow)
ACSC = Aviation Corridor to Santa Cruz Interceptor; AV = Aviation Corridor; CDO = Canada Del Oro; CW = Campbell Wash; Dove Mtn = Dove Mountain; GV = Green Valley WRF; NRI = North Rillito Interceptor; NOW = Northwest Outfall; PONT = Pontatoc Wash; PTI = Pantano Interceptor

Table 6-1 (Continued)
Historical and Calculated Flow Rates and Water Depths

Site	Flow Rate, mgd					Design Capacity[2], mgd	Wet Weather Flow[3]	PF[4]	Water Depth/Pipe Diameter					Pipe Diameter (inch)
	Historical[1]				Calculated				Historical[1]				Calculated	
	10%	50%	90%	100%	10-year				10%	50%	90%	100%	10-year	
PTI-2	-	-	-	-	-	13.5	-	-	0.35	0.38	0.41	0.45	0.48	30
SCE-1	25.23	28.33	34.05	44.32	49.52	148.1	21.19	2.45	0.28	0.30	0.31	0.37	0.36	78
SCE-2	-	-	-	-	-	7.8	-	-	0.30	0.38	0.42	0.62	0.58	30
SCI-1	4.19	4.51	4.97	8.34	7.29	12.1	2.78	2.26	0.38	0.40	0.42	0.62	0.57	30
SEI-1	-	-	-	-	-	92.8	-	-	0.36	0.37	0.38	0.46	0.47	60
SEI-2	-	-	-	-	-	-	-	-	0.30	0.31	0.33	0.98	0.52	36
SEI-3	-	-	-	-	-	21.4	-	-	0.33	0.36	0.39	0.57	0.53	30
SRC-1	14.93	16.17	18.03	26.26	28.53	11.8	12.36	2.47	0.35	0.37	0.38	0.45	0.48	54
SRW-1	2.22	2.42	2.69	3.90	4.23	7.8	1.81	2.45	0.30	0.32	0.33	0.51	0.51	30
SRWN-1	-	-	-	-	-	127.9	-	-	0.20	0.24	0.25	0.28	0.28	66
SRWS-1	-	-	-	-	-	9.7	-	-	0.48	0.50	0.53	0.71	0.77	27
SWI-1	5.48	6.39	7.10	9.46	8.52	43.9	2.13	1.87	0.28	0.31	0.33	0.89	0.35	33
TUCDIV	1.37	3.13	7.61	11.98	14.92	30.2	11.79	6.67	0.20	0.30	0.40	0.96	0.70	33

(1) Based on data recorded from July 1, 2005 to June 30, 2006
(2) From previous study: 2006 Metropolitan Area Facility Plan Update. Included here for reference only
(3) Wet weather flow, estimated as the difference between the calculated 10-year flow and historical 50-percentile flow
(4) PF: Peaking factor, estimated as 1.4 x (calculated 10-year flow/ historical 50-percentile flow)
SCE = Santa Cruz-East Interceptor; SEI = Southeast Interceptor; SRC = South Rillito Interceptor-Central;
SRW = South Rillito Interceptor-West; SRWN = South Rillito Interceptor-West, North Line; SRWS = South Rillito Interceptor-West, South Line;
SWI = Southwest Interceptor; TUCDIV = Tucson Boulevard Diversion

Figure 6-2
Revised Peaking Factor Curve



6.2.3.4 Details of Flow Data Analysis

Flow data for each of the existing 27 monitoring sites was evaluated and interpreted for use in the conveyance system capacity analysis. The specific details by interceptor are included in **Appendix H.** It also details any suspect data that was excluded from the analysis.

6.2.3.5 July 2006 Flow Analysis

During the course of this analysis it was observed that the flow monitoring period selected for analysis was relatively dry and might result in underestimating the magnitude of peak wet weather flows. It was also noted that a series of significant events occurred near the end of July 2006 that might assist in verifying the probabilistic approach. Of these storms, the one that occurred on July 29 was found to have caused the greatest system flows and was selected for additional statistical analyses.

As **Figure 6-3** indicates, rainfall volumes varied greatly throughout the service area. Therefore individual rain gauge data were grouped into three regions (Canada, Tucson, and Tanque) and consolidated into "composite" rainfall hyetographs. A statistical analysis of this event determined its recurrence interval ranged from 1 to 10 years depending on location and duration (see **Figure 6-4**).

Figure 6-3
July 29, 2006 Rainfall Totals

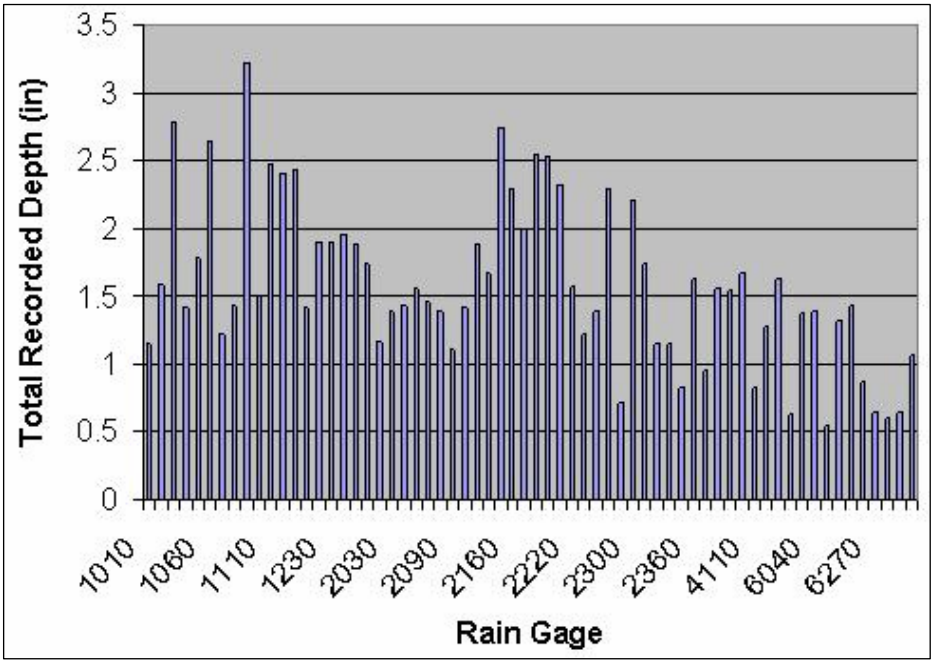
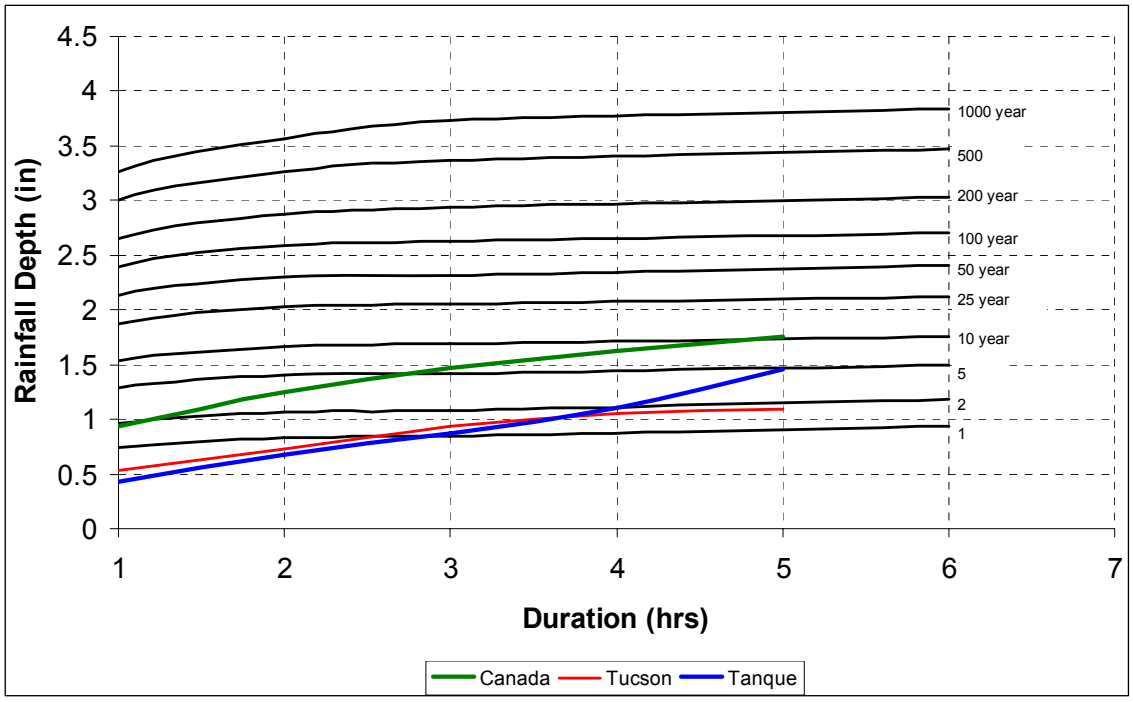


Figure 6-4
July 29th Rainfall Statistical Analysis



At each monitoring site that recorded both depth and flow a new data point for this particular event was added to the flow frequency distributions. Figures indicating where these points fell relative to the flow frequency distributions have been included in **Appendix H**. The majority of the data points fall relatively close to the frequency curves which would seem to validate the approach. However, there are a few exceptions where the data points fall off the curves. This is likely in large part due to the fact that the spatial variability of the rainfall makes the development of correlations between individual events and system flows challenging. This difficulty was a major factor in the decision to utilize the probabilistic method to approximate wet weather flows. It eliminates the need to derive such correlations as it is based exclusively on the frequency of flow without attempting to correlate such flows to rainfall.

6.2.3.6 Summary and Conclusions

While the Pima County conveyance system does experience an increase in flow in response to wet weather events, flow data indicate it has adequate excess capacity to convey these flows in accordance with CMOM criteria. On the other hand, as the service area population grows, excess system capacity will be reduced and the ability for the system to reliably convey peak wet weather flows will subsequently be reduced. There are some portions of the system that will most likely need to be augmented in the future in order to maintain adequate excess capacity for wet weather flows. The method detailed in Chapter 6.4.4 was used to account for wet weather flows when evaluating system hydraulics.

There are some notable limitations to the probabilistic method. First, this technique presumes that one year of flow data is sufficient to project the flows and depths that are likely to occur once every 10 years. Second, this technique makes no attempt to associate measured flows with rainfall. Finally, this technique estimates flows, not volumes. Therefore, while it may be a reasonable means of estimating wet weather flows, it has no means of estimating what sort of detention volumes might be employed as an alternative to increased conveyance capacity.

While this study has clearly shown that there are some significant wet weather impacts, a more comprehensive wet weather study is warranted and has been commissioned by PCRWRD. It is recommended that this study evaluate the correlation between measured rainfall and flows. These measurements should then be used to assist in the calibration of a hydrologic model capable of simulating the response of the collection system to actual and hypothetical rainfall events. This model could be used to validate the recommendations of this report, and could also be used to evaluate alternatives to increased conveyance, such as detention and flow equalization.

6.3 Future Conveyance System Capacity Requirements

6.3.1 Population / Flow Estimates

Model loads were derived largely from population estimates. The same traffic analysis zone (TAZ) population data that had been developed by PAG and was used for the 2006 Facility Plan was also used as the basis for estimating inflows into the MOUSE model described in Chapter 6.5. In general, a per capita wastewater flow rate of 85 gpcd was applied to the population estimates to determine model inflows. In some areas these population values were adjusted to account for customers on septic systems.

6.3.2 Previous Studies – 2006 Metropolitan Area Facility Plan Update

The same population estimates used in the 2006 Facility Plan were used for this study. The table below consolidates these estimates into a single table. For future flow scenarios it was assumed that all properties currently on septic would be served by the collection system.

6.4 Conveyance System Evaluation Criteria

6.4.1 Design Standards

Majority of the standards that were used to evaluate the system and develop the CIP have been established via State legislation. These are discussed in Chapter 6.4.2.

6.4.2 Regulatory Issues

Any Master Planning project must be done with existing and anticipated relevant rules and regulations in mind. A review of the Arizona Administrative Register[1] revealed several anticipated rules that impact this project. While this document is not yet policy, it is anticipated that it will be adopted as policy by 2008. Those proposed rules that affect master planning pertain primarily to the State's recommendation that a regulated CMOM standard be applied. Those proposed rule changes found to be most relevant to this master planning effort are:

- **R18-9-C305 2.05 General Permit**: Capacity, Management, Operation, and Maintenance of a Sewage Collection System – Specifically, this general permit will allow an operator to operate and maintain a collection system under the terms of a CMOM Plan. This permit applies to existing facilities. The CMOM plan must include:
  - Operation and maintenance of ability of system so as to maintain capacity to convey peak dry and wet weather flows, prevent SSOs, and respond to and report releases.
  - Capital improvement plan.
  - How to maintain adequate capacity to base flows and peak wet weather flows of a 10-year-24-hour storm event for all parts of the conveyance system.
  - Identification of conveyance system components that do not meet preceding criteria, and develop a CIP such that these components will be compliant within 10 years.

- PCRWRD received "notice of recording" from ADEQ from the type 205 General Aquifer Protection Permit on November 27, 2006.

- **R18-9-E301 4.01 General Permit**: Sewage Collection Systems – This permit pertains to newly constructed sewage collection systems.
  - Sewage collection system must be designed and operated such that it:
    - Provides adequate wastewater flow capacity for the planned service area;
    - Maintains proper flow velocities so as to minimize sedimentation;
    - Prevents SSOs via proper sizing and I/I reduction measures;
    - Minimizes exfiltration losses;
    - Provides for adequate inspection, maintenance, and testing;
    - Maintains structural integrity; and
    - Minimizes septic conditions in the collection system.
  - Design Requirements
    - Apply appropriate dry weather peaking factor and add wet weather I/I rate;
    - Maximum d/D of 0.75 during dry weather;
    - All collection system appurtenances should be designed such that any part of the system, when flowing full, can accommodate a peak wet weather flow calculated by

[1] Secretary of State, Arizona Administrative Register Notices of Proposed Rulemaking, Volume 11, Issue 2, January 7, 2005.

multiplying the sum of the upstream dry weather flows by a dry weather peaking factor based on upstream population and adding a wet weather I/I rate based on either a percentage of peak dry weather flow or a gallons per acre rate of flow;

- Minimum radius of curvature = 200 feet;
- Minimum depth of cover = 3 feet[1];
- Minimum velocity = 2 feet per second[2]
- Maximum velocity = 10 feet per second[3]
- Maximum manhole spacing according to **Table 6-2**:

Table 6-2
Manhole Spacing

Sewer Pipe Diameter (inches)	Max. Manhole Spacing (feet)
Less than 8	400
8 to less than 18	500
18 to less than 36	600
36 to less than 60	800
60 or greater	1300

6.4.3 Diurnal Flows / Peaking Factors / Flow Equalization

Dry weather peaking factors (PFs) established in the Arizona State Register are shown in **Table 6-3**:

Table 6-3
Dry Weather Peaking Factors

Upstream Population	Dry Weather Peaking Factor
1,001 – 10,000	$PF = (6.330 * p^{-0.231}) + 1.094$
10,001 – 100,000	$PF = (6.177 * p^{-0.233}) + 1.128$
More than 100,000	$PF = (4.500 * p^{-0.174}) + 0.945$

[1] Unless ductile iron or pipe or other design of equivalent or greater tensile and compressive strength is used.
[2] When flowing full, assuming Manning's formula and roughness coefficient of 0.013 are used.
[3] Unless ductile iron, or material of equivalent erosion resistance is used.

These three equations were consolidated to a single equation by assuming a per capita flow rate of 85 gpcd, plotting the results, and using regression analyses. The resultant curve fit and equation are indicated on **Figure 6-5**.

Peaking factor is of particular concern with regard to the evaluation of the plant interconnect pipeline. This is because this 5-mile large diameter sewer must be designed to convey peak flows, not just average daily flows. **Figure 6-6** represents the typical dry weather diurnal flow that is expected to reach the Roger Road WRF in the year 2030.

Figure 6-5
Consolidated Dry Weather Peaking Factor Curve

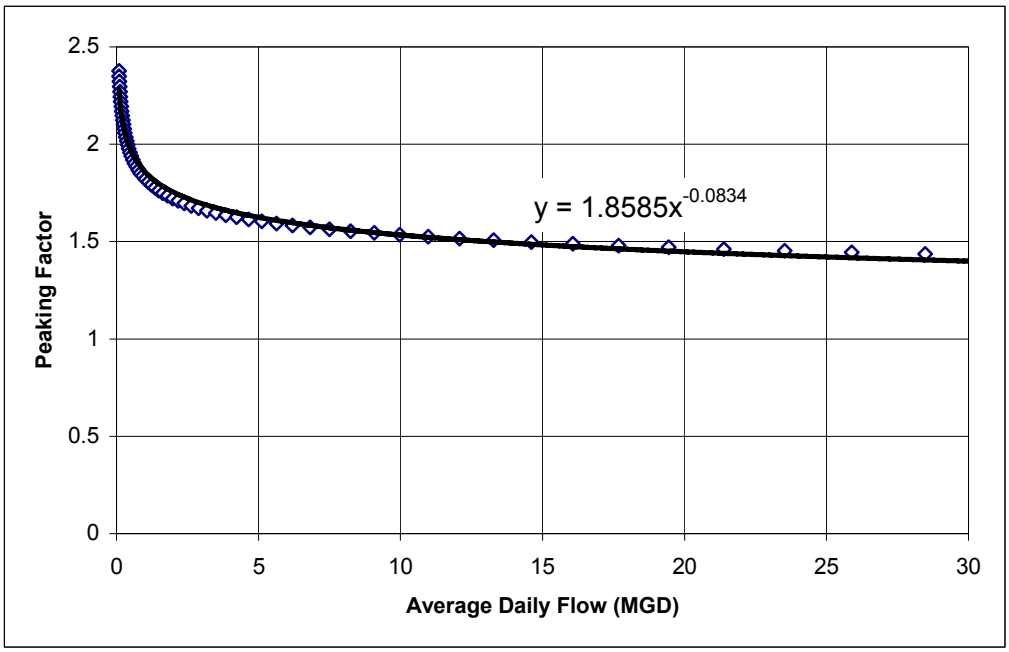


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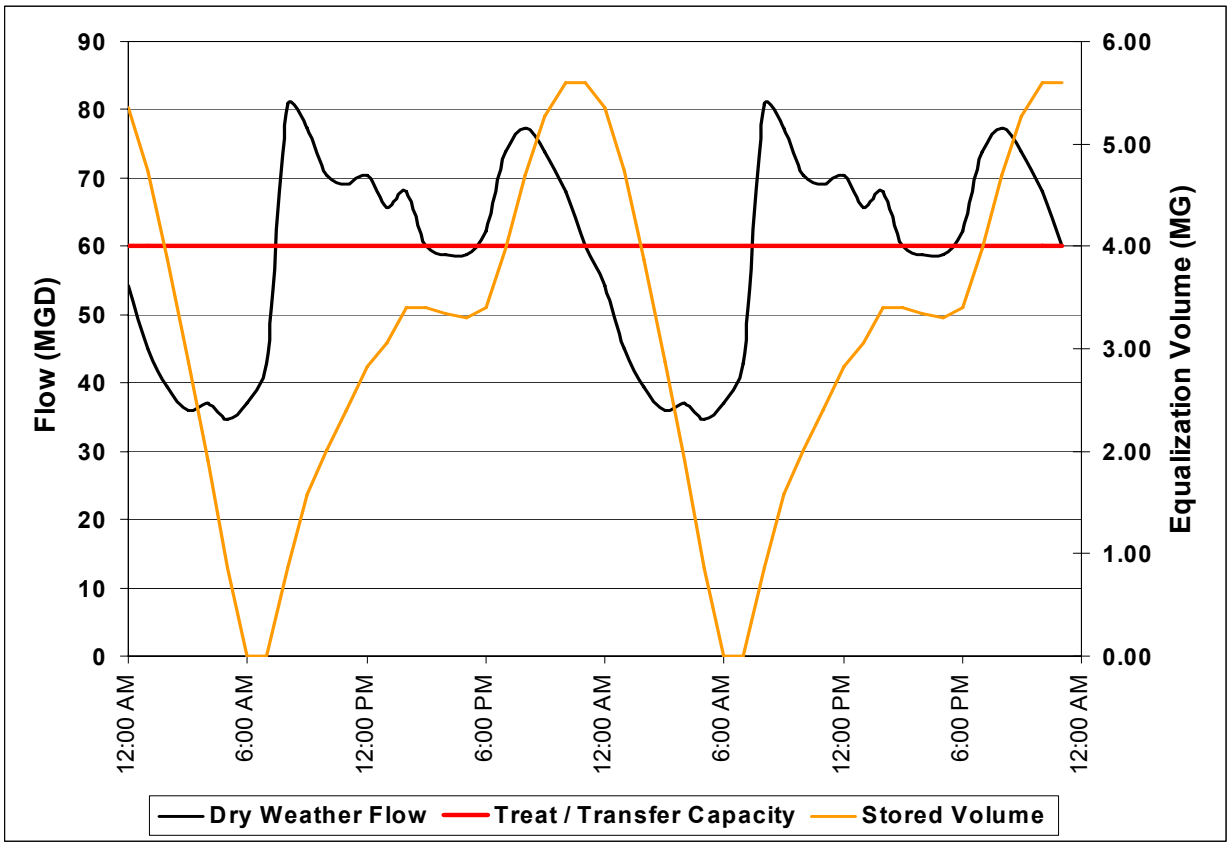
Figure 6-6
Diurnal Flow Equalization



6.4.4 Wet Weather Flow Allowance

The State Standards are somewhat vague in specifying how communities should design conveyance systems to accommodate peak wet weather flows. This is due to the site specific nature of wet weather events, which makes it difficult to develop a standardized approach to accommodate wet weather flows in a manner that would be appropriate for all conveyance systems. The standards state only that new collection system appurtenances should be designed to convey the 10-year event and that existing facilities should be able to do so within 10 years (See Chapter 6.4.2.). However, the standards also recommend allowing for wet weather flow as a percentage of peak dry weather flow or on a gpd/acre basis. Both techniques are applied to this analysis.

Referring to **Figure 6-2**, statistical flows data analyses seem to indicate that flows less than 10 mgd trend along a revised peaking factor curve that provides for sufficient capacity for system appurtenances to convey the peak 10-year flow without surcharging. The percentage of pipe capacity that remains for wet weather flows is also indicated.

However, for pipes with average dry weather flows over 10 mgd (those in more downstream reaches of the system as indicated by monitoring sites NRI-1, NRI-2, SRC-1, and SCE-1) the revised peaking factor curve is not applicable, most likely due to the unique nature of I/I in these downstream portions of the system. One possible explanation for the relatively high I/I recorded is that some of the older interceptors were constructed in the storm water washes, which can permit storm water flow to enter the sanitary system through manhole covers. Therefore, pipes with dry weather flows greater than 10 mgd the weighted average per acre wet weather flow rates from measured flows as indicated in **Table 6-4** were applied when evaluating pipe capacities. Peak flows in the portion of the system tributary to Roger Road were substantially higher than those to the Ina Road tributary area, therefore different peaking factors were used for their conveyance systems.

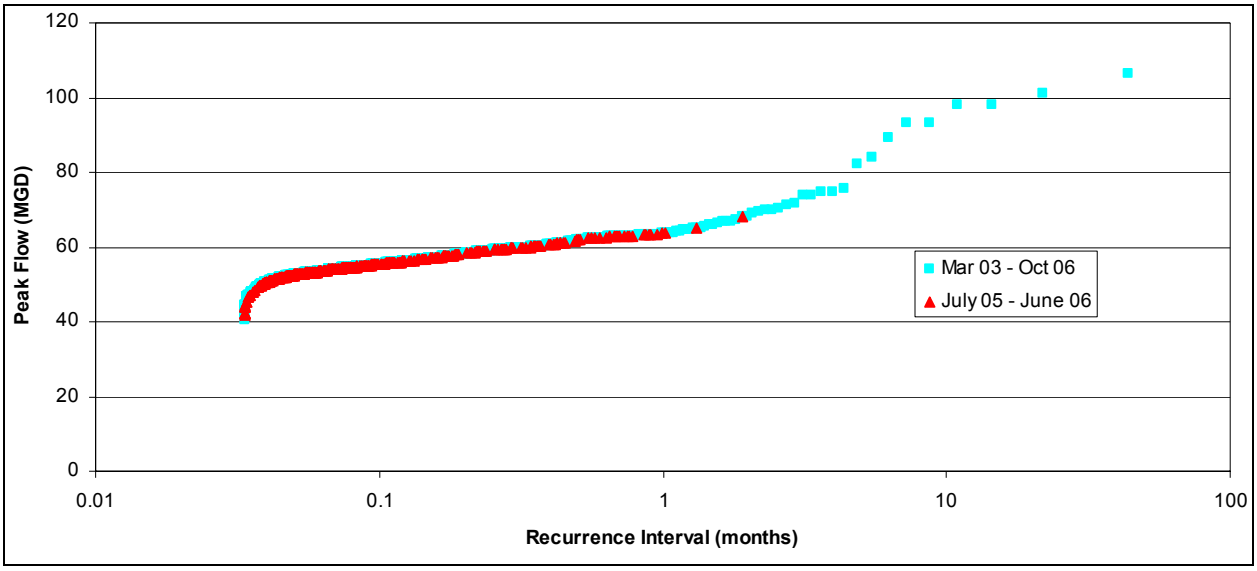
Table 6-4
Per Acre 10-Year Wet Weather Flow Rates

Meter Site	Service Area (acres)	10-Year Wet Weather Flow (mgd)	gpd/acre
NRI-1	110,190	11.16	101
NRI-2	101,249	17.97	177
Ina Road Service Area Weighted Average			**138**
SRC-1 (Roger Road WRF)	41,880	12.36	295

6.4.4.1 3-Year Roger Road WRF Flow Analysis

During the course of the development of the wet weather flows, it was noted that the 12-month flow monitoring period of July 2005 through June 2006 was relatively dry and might underestimate wet weather flows. Since the design of the plant interconnect pipeline is of particular concern, an analysis of 3.5 years of Roger Road WRF flows was performed to determine whether the 295 GPD/acre value is sufficiently conservative. The results of this analysis, utilizing plant flow data from March 2003 through October 2006, are presented on **Figure 6-7**. This frequency distribution of the peak flows into Roger Road WRF results in a 10-year peak wet weather flow to the plant of 110 mgd. Subtracting the median peak dry weather flow of 56 mgd results in a peak 10-year wet weather flow of 54 mgd or 264 gpd/acre (based on a service area of ~200,000 acres), approximately 10 percent less than what had been determined in the preceding analysis. The more accurate value was presumed to be 264 gpd/acre as it was based on a longer data period and has been used in subsequent pipe capacity analyses.

Figure 6-7
Roger Road WRF Peak Flow Frequency Distribution



6.4.5 Effluent Reuse

Effluent use for reclaimed water service does not impact the service area wastewater conveyance systems upstream of the wastewater treatment facilities at either Roger Road WRF or Ina Road WRF. The reclaimed water uses treated wastewater downstream of these facilities.

6.4.6 Odor Control / Conveyance Time

Odor generation, largely a function of conveyance time, was not evaluated as part of this study. A separate odor control model is being utilized for this purpose. However, the odor control model is based largely upon the hydraulic model used for this study. The pipe network and average daily flows generated by the hydraulic model described in the next chapter were used as input into a model that simulates odor generation. Odor control within the conveyance system is covered in a separate report on system-wide odor controls under development at the time of this report.

6.5 Evaluation of Conveyance System

A detailed engineering analysis of the capacity of the skeletal conveyance network to convey both dry and wet weather flows was performed. This analysis relied heavily on a hydraulic model to evaluate system performance under a variety of flow and system configurations.

6.5.1 Hydraulic Model Development

The 2006 Facility Plan utilized a "BASIN" model which relied on TAZ population data and subcatchment delineations to estimate inflows into the conveyance system. Wastewater flows were routed through the branched network by summing flows as they accumulated towards the treatment plants. This model evaluated the adequacy of conveyance system capacity by comparing estimated flows to the pipes' Manning's capacities, which are the theoretical full pipe capacities based on size, slope, and roughness. While a good base level tool, the BASIN model approach does not have the robust hydraulic computational ability to accurately evaluate system capacity.

In particular, this approach assumes that the Manning's capacity at a single location is representative of the pipe's overall capacity. However, in reality pipe slopes are not uniform, and if the segment used to calculate the Manning's capacity had a relatively flat slope in comparison to the upstream and downstream segments, the pipe might be able to carry far more flow than the initial Manning's capacity would indicate.

> *"Thus, individual sewer reaches within a sewer system can carry more than their calculated design capacity with no adverse consequences, while individual sewers within the sewer system can backup, overflow, and/or flood even while carrying flows well within their calculated design capacity. Consequently, it is necessary to define a system capacity independent of the calculated design capacity of the individual reaches of sewer."*[1]

The 2006 Facility Plan highlighted interceptors as orange and red if their capacities were found to be deficient. "The Orange classification identifies sections of the interceptor with potential capacity problems and suggesting only an engineering study to determine the severity of the problem." Further the report states, "The pipe capacity of the interceptor in the identified basin requires an engineering evaluation before being certified as being deficient." In addition, this model could not adequately predict the behavior of flow splits as stated, "This module worked with no problems for most cases, however at the time of model development, no reasonably effective way could be found for the GIS module to accommodate the numerous flow splits, such as the Tucson Boulevard Diversion…" The model developed as part of this study has the ability to simulate flow split hydraulics by inputting the physical parameters of the diversion structures.

The more robust model detailed in the remainder of this chapter will provide the capability to perform the engineering evaluations recommended in the Facility Plan. The model selected for performing engineering analyses is MOUSE, which is described in greater detail in the next chapter.

[1] Water Environment Federation, Guide to Managing Peak Wet Weather Flows, 2006.

6.5.1.1 Model Capabilities / Limitations

Developed by the Danish Hydraulic Institute (DHI), MOUSE is a powerful conveyance system computational modeling package. Similar to InfoWorks, MOUSE finds a complete solution to the St. Venant equations, which enables it to simulate virtually any hydraulic phenomena including open channel flow, surcharged and pressurized flow, reverse flow, weirs, gates, pump stations, overflows, backwater effects, all within a dynamic simulation that enables modelers to account for travel time, system storage and peak flow attenuation. MOUSE uses a variable time-step kinematic wave formulation to solve for the flow conditions, resulting in a high degree of computational accuracy and stability. In addition, DHI has developed MikeUrban, a GIS-centric model manager that greatly enhances the power of both modeling and GIS applications.

The graphical post-processor, MikeView, enables modelers to quickly evaluate and analyze simulation results. Quantities of interest such as conduit flows, flow depths, manhole flooding, available pipe capacity, are readily analyzed and displayed. Profiles of selected sewer segments can be viewed and animated to better comprehend system limitations and bottlenecks.

One of the limitations of MikeUrban is that it is challenging to have multiple constraints, boundary conditions, and loads applied to a subset of data, such as different per capita flows in different basins. Rather it uses a “population equivalent” (PE) value to account for variability in per capita flow rates. This issue arose during the Ina Road WRF sewershed calibration and ultimately, the population values were adjusted to a PE value that resulted in the appropriate wastewater flow rate. This issue will be addressed further in Chapter 6.5.1.3.

Most importantly the model is only as accurate as the information that was used to create it. The conveyance system GIS database was relied upon heavily for model development. Generally, only pipes 15-inch and larger were included. Pipe segments missing invert elevation data were identified and were interpolated for using the nearest upstream and downstream invert elevations. Approximately 13 percent of the pipe inverts were missing. In addition, there were obvious erroneous values which prevented network flows to be conveyed downstream; these were also corrected via interpolation. While all of these estimates were reasonable and should not adversely affect model accuracy within the scope of this master planning study, minor differences between simulated and actual hydraulics may occur.

6.5.1.2 Set-Up / Configuration

Model network was first simplified, or “skeletonized”, to remove manholes along pipes of uniform size and slope, thereby reducing simulation times without sacrificing accuracy. The modified network was imported into MikeUrban for final debugging. Plans for key hydraulic structures such as weirs, gates, and diversions, were used as the source documents for adding appurtenances to the model.

Using the basin population estimates from the TAZ data, loads were assigned to the model nodes. Generally, a visual inspection of pipe network connectivity was performed such that subcatchment loads were assigned to model nodes at the point on the interceptor where the majority of the subcatchment

flows seemed to discharge into it. This necessary simplification could result in excessive flows passing through some portions of the system, while some flows may short circuit other parts of the interceptor network. In general, these limitations should be considered minimal as subcatchment boundaries were delineated so as to minimize these errors.

Several point loads listed in the 2006 Facility Plan were added into the model. These flows came from sources such as the University of Arizona, the Davis-Monthan Air Force Base, and Tucson Mall. These loads could not be accounted from population databases.

There are several diversion structures in the collection system that regulate and split flow into the various interceptors. The following structures were included in the model so as to accurately simulate their hydraulics: Craycroft, Tucson Boulevard, 18th and Vine, and Alameda Siphon (**Figure 6-8**).

6.5.1.3 Calibration Summary

Hydraulic model of the conveyance system was calibrated using data from the BASIN model of the sewersheds and the flow meter data. The Roger Road WRF and the Ina Road WRF inflows were included. As for the BASIN model, for the Roger Road WRF system a per capita flow rate of 85 gpcd was used to estimate the load from each basin sub-basin. For the Ina Road WRF system, the variable per capita flow rates were accounted for recalculating the "population equivalent" to a value that matched the BASIN flow rates while using 85 gpcd.

A significant step in the model calibration was simulating the multiple diversion structures contained in the network. Examples include the Alameda Siphon, the Tucson Boulevard Structure, and the Craycroft Diversion (**Figure 6-8**). Some of the elevations contained within the record drawings did not match GIS data to preserve the hydraulics of the various structures, therefore relative elevations were used.

Once the model was debugged, a simulation was run using average dry weather flows (ADWF) and the results were compared to the meter data and the BASIN model flows. This comparison is shown graphically on **Figure 6-9**. Since the model essentially used the same loading technique as the BASIN model, MOUSE model flows would be expected to generally match those of the BASIN model rather than the flow monitoring data. However, the BASIN model used estimated flow splits at the diversion structures whereas the MOUSE model simulated diversion hydraulics resulting in some minor differences between the two.

Figure 6-8
Diversion Structure Locations

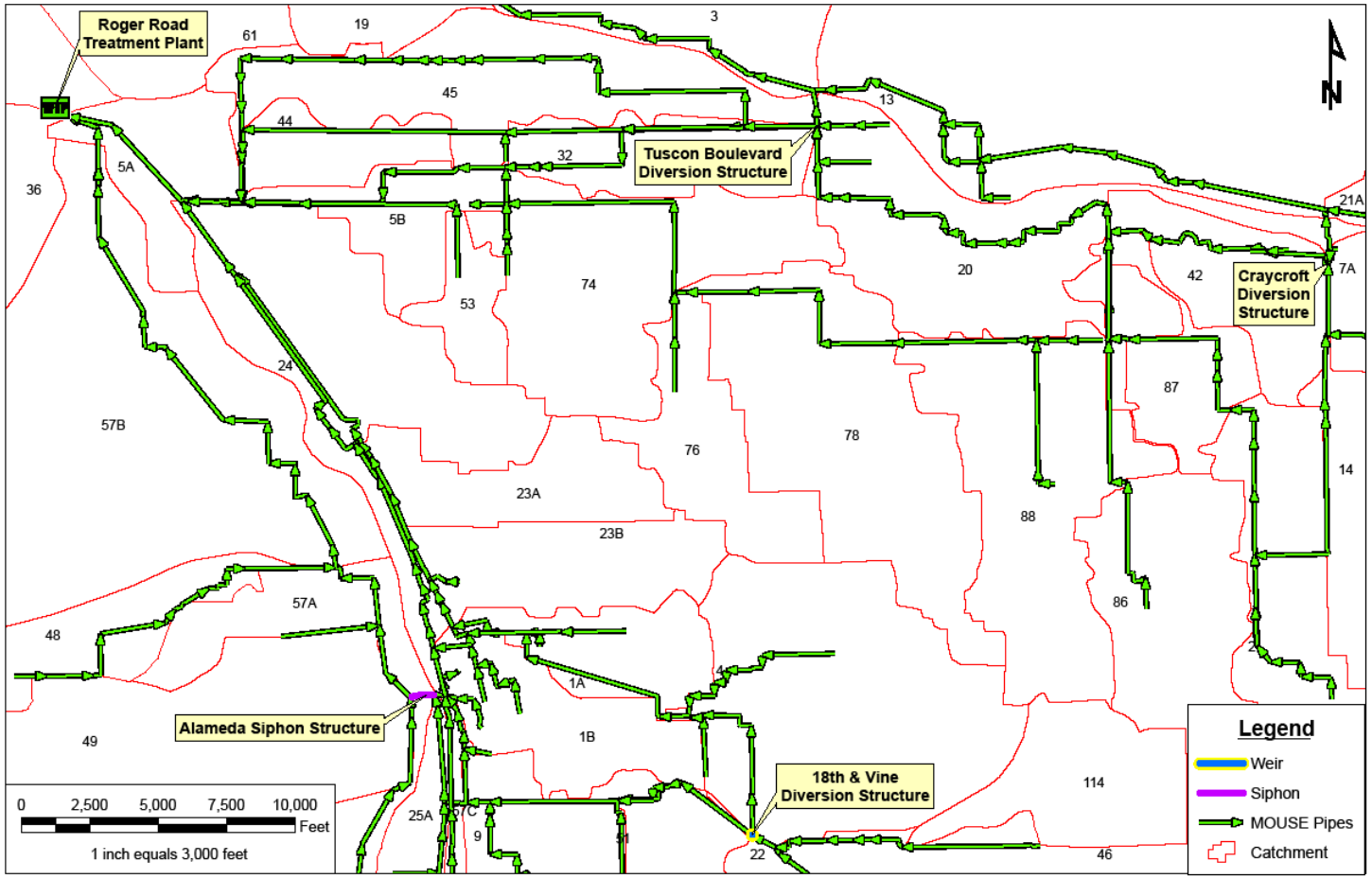
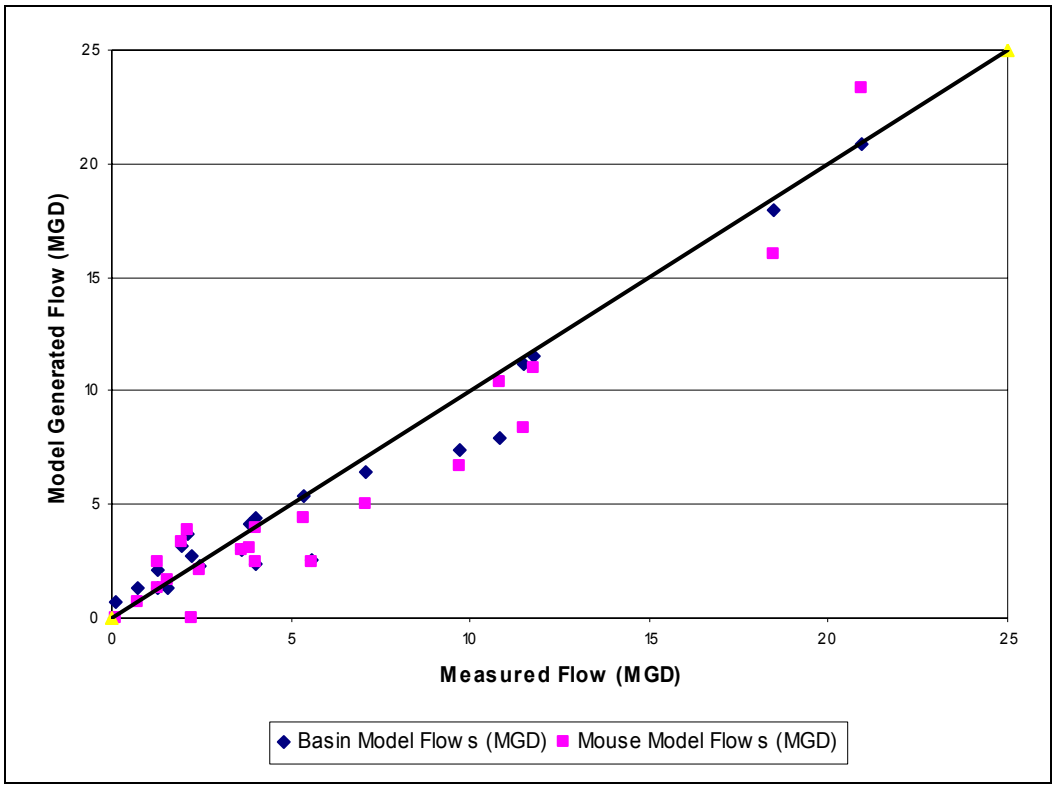


Figure 6-9
MOUSE Model Calibration

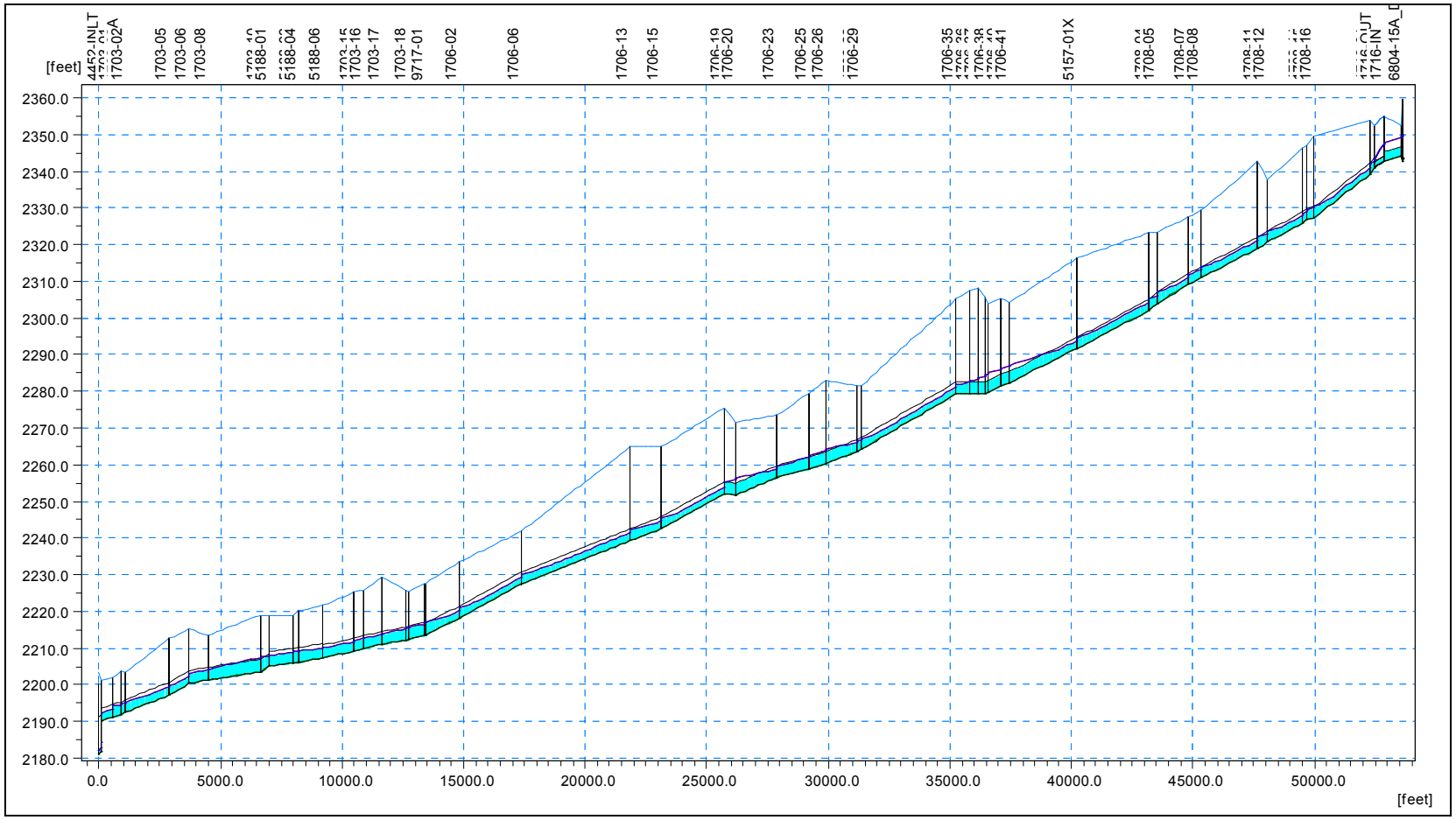


6.5.1.4 Tucson Boulevard Diversion Capacity Test

There has been some uncertainty as to the maximum amount of flow that could be diverted to the North Rillito Interceptor (NRI) via the Tucson Boulevard diversion structure. On July 28, 2005 a stress test was performed to maximize the flow through the diversion to the Ina Road WRF. This was done by closing the gate that allowed the flow to travel west to the Roger Road WRF, thereby transferring all flow north to the NRI. The test revealed that diverting approximately 12 mgd of flow into the NRI would reach the limit of its conveyance capacity without exceeding capacity

The conveyance system model was modified to simulate the same Tucson Boulevard diversion configuration to confirm that the results compared favorably to those of the stress test. For this test slightly more (13.5 mgd) of flow was diverted to the NRI. Flow monitoring data indicated depth of flow in the diversion to be 17 inches. The model predicted a depth of 25 inches. The difference is due in large part to the fact that the model simulated an additional 1.5 mgd of flow. The pipe profile on **Figure 6-10** is that of the simulation result. While the NRI is flowing slightly more than full in a few places, it is able to convey a maximum flow of 13.5 mgd of diverted flow reasonably well. However, additional flow may lead to excessive surcharging in some locations.

Figure 6-10
NRI Profile during Tucson Boulevard Diversion Stress Test



6.5.2 Planned System Upgrades

As of the completion of this evaluation the new Santa Cruz Central interceptor was the only planned system upgrade that was of a magnitude that warranted consideration. As of the date of publication of this report this sewer in under construction and not in service. For analyses it was presumed that this sewer would be placed into operation by the year 2010. Plans were obtained to determine the necessary physical sewer information to represent this new sewer in the hydraulic model.

6.5.3 Hydraulic Deficiencies / Excess Capacity

Referring to **Table 6-1**, there was no observed system surcharging during the 12-month flow monitoring period. This would seem to indicate that the system currently has sufficient capacity to accommodate existing dry and wet weather flows. However, a hydraulic simulation was run to evaluate the hydraulic adequacy of all elements of the skeletal network.

Once the model calibration was completed, a simulation was run under dry weather conditions with a typical diurnal pattern applied to system loads. **Figure 6-11** is a profile of the NWO sewer during peak dry weather flow. It is, for the most part, able to convey these flows at less than 85 percent of its peak capacity (75 percent full).

As a check of model results, the d/D of the model at the flow monitoring site was compared to the 12-month peak water levels from meter NWO-1. The model predicted a d/D of 0.56 while the median daily peak d/D from the monitoring data was 0.52, a difference of 8 percent.

This analysis indicates that a substantial portion of the downstream reaches of the NRI may be flowing at over 85 percent capacity. As a check of model results the peak d/D from the model was compared to that of the meter at site NRI-1. The model predicts a peak d/D of 0.65 while the median measured value was 0.56, indicating the model is conservative by 16 percent. A profile of this portion of the system from Shannon Road to the Ina Road WRF is shown on **Figure 6-12**. The portion of the sewer between manholes 1703-10 and 1703-06, which parallels I-10 just before crossing the interstate near Ina Road WRF, appears to be near capacity.

Figure 6-11
NWO Sewer Profile during Peak Dry Weather Flow

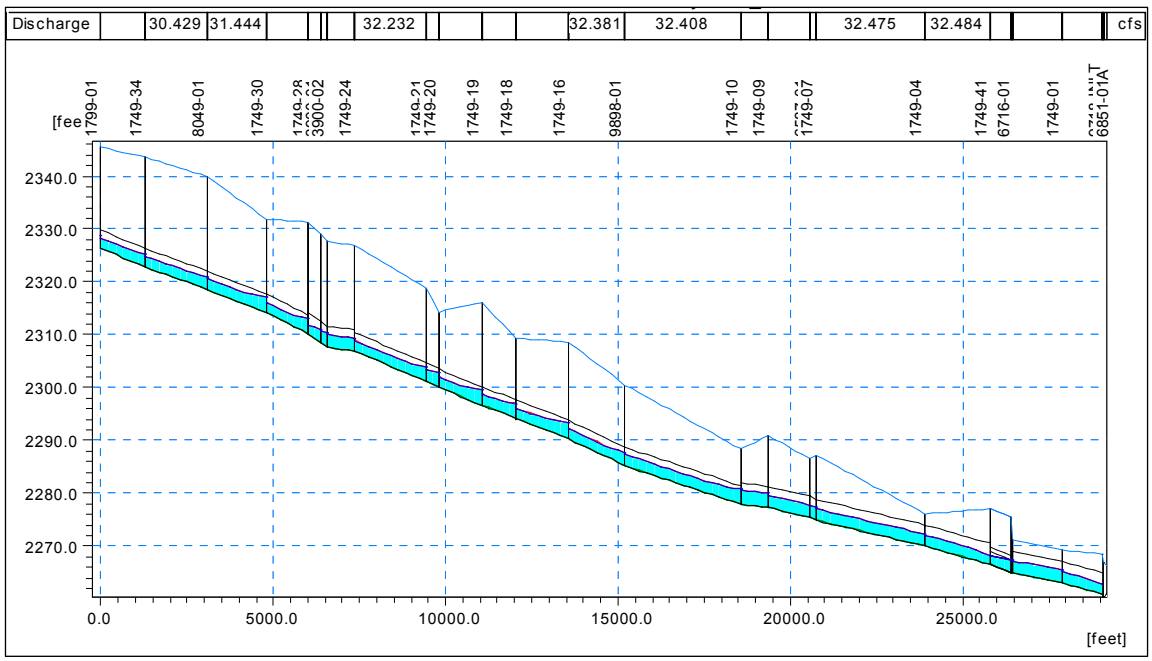
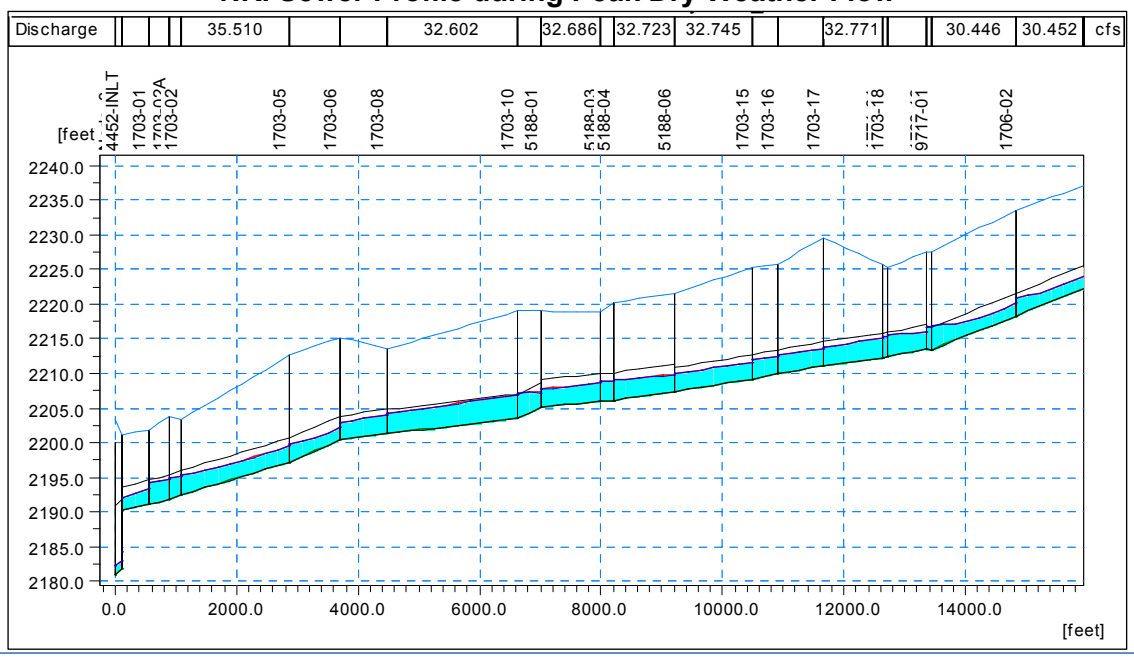
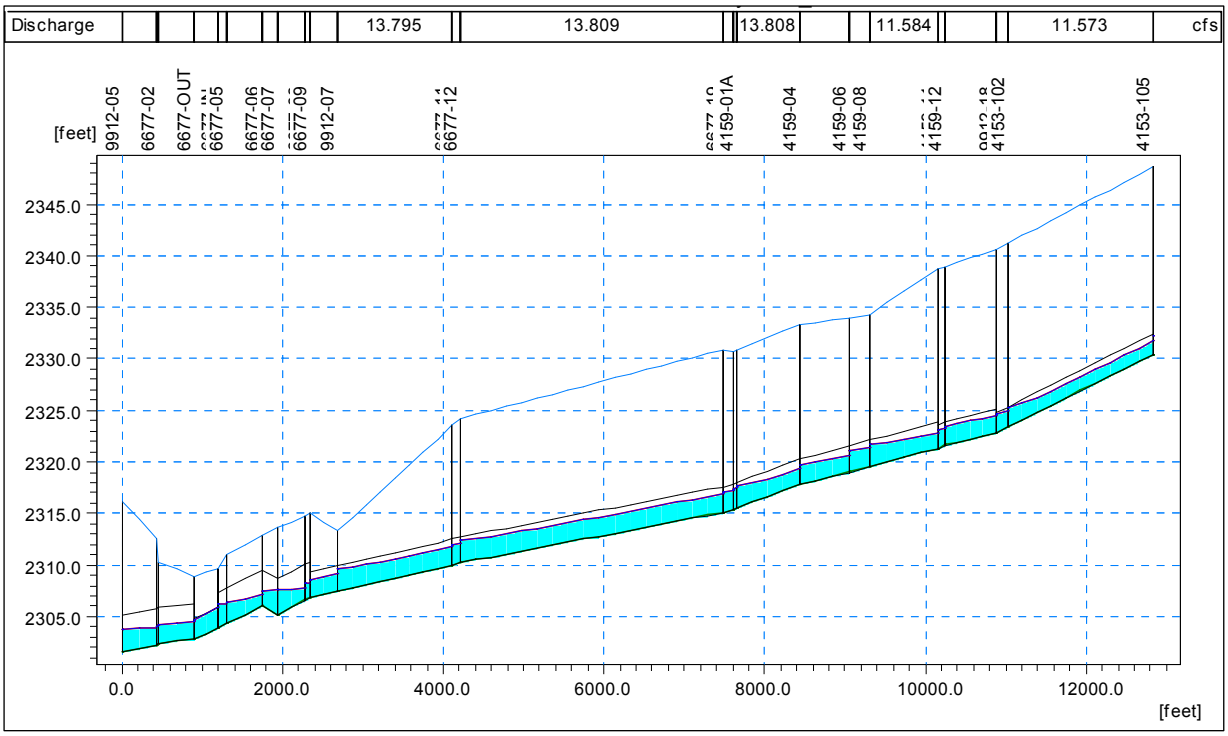


Figure 6-12
NRI Sewer Profile during Peak Dry Weather Flow



Portion of the Santa Cruz Central Interceptor (SCC) between the Alameda Siphon and Grant Road also appears to be flowing above 85 percent of its design capacity, however, no flow metering data are available with which to confirm these results. Nevertheless, a profile of this sewer shown in **Figure 6-13,** indicates that substantial lengths of this sewer are flowing near capacity. Installation of the new Santa Cruz Central interceptor by the year 2010 should alleviate the capacity issues of the NWO and SCC interceptors.

Figure 6-13
SCC Sewer Profile During Peak Dry Weather Flow



Further review of the conveyance system seems to conclude that with the exception of those sewer segments discussed above, the majority of the conveyance system currently has excess capacity available as indicated in **Figure 6-14**. The construction of the new Santa Cruz Central interceptor and plant interconnect pipeline should relieve these current hydraulic deficiencies.

A preliminary evaluation of the adequacy of the system to convey the 10-year event was also performed. This was done by applying the peaking factor curve to the ADWF for pipes with ADWF greater than 10 mgd. For those with ADWF less than 10-mgd wet weather flow was derived from the per acre 10-year wet weather flow rates. The resultant peak 10-year flow estimates were then compared to pipe design capacities as shown in **Figure 6-15**. Results of this analysis, has the pipes color coded according to the percentage of their capacities utilized during the 10-year event. For this evaluation flows are not considered excessive until they exceed 100 percent rather than 85 percent as the wet weather flow as discussed under peak pipe flows

Figure 6-14
2005 Hydraulic Capacity Evaluation

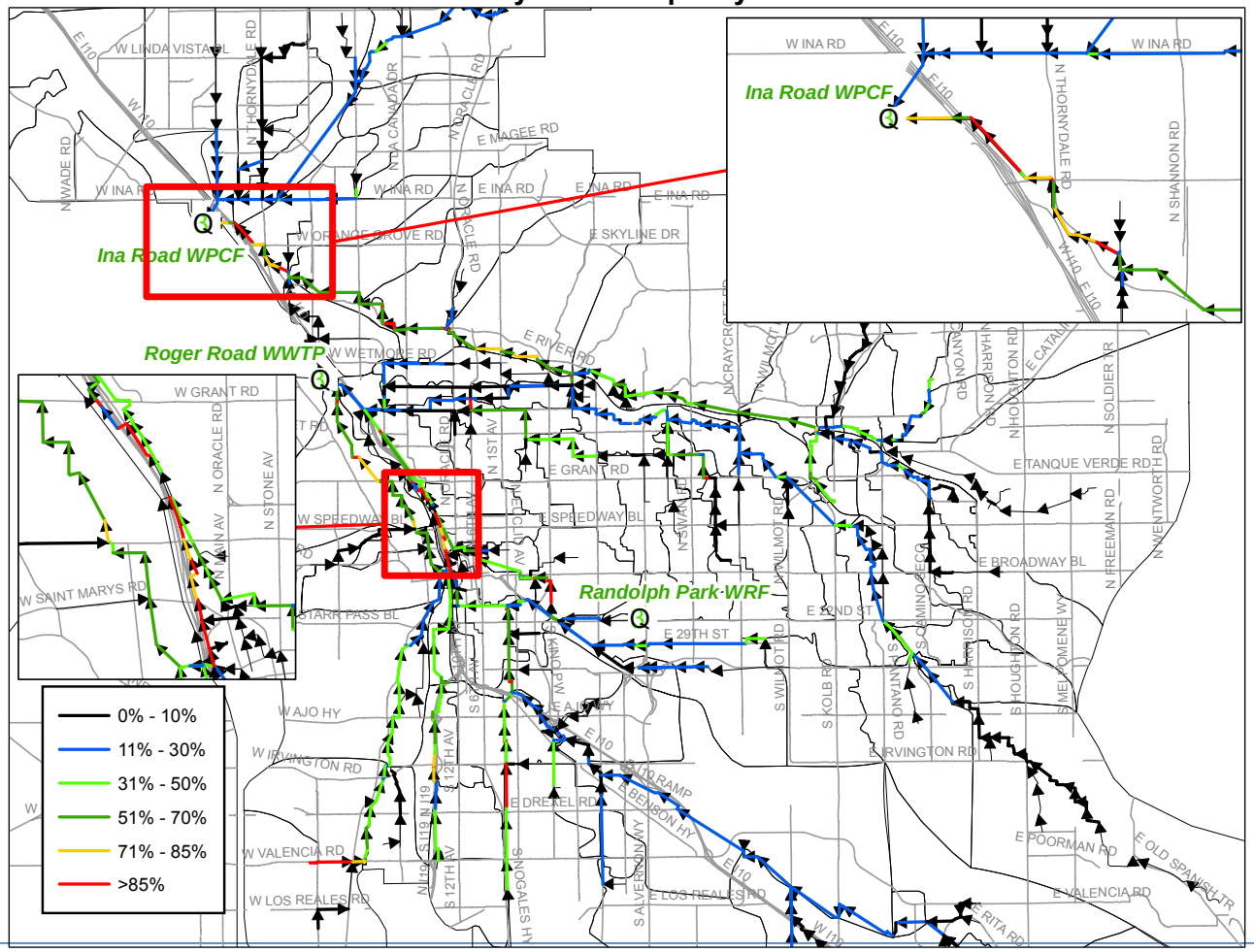
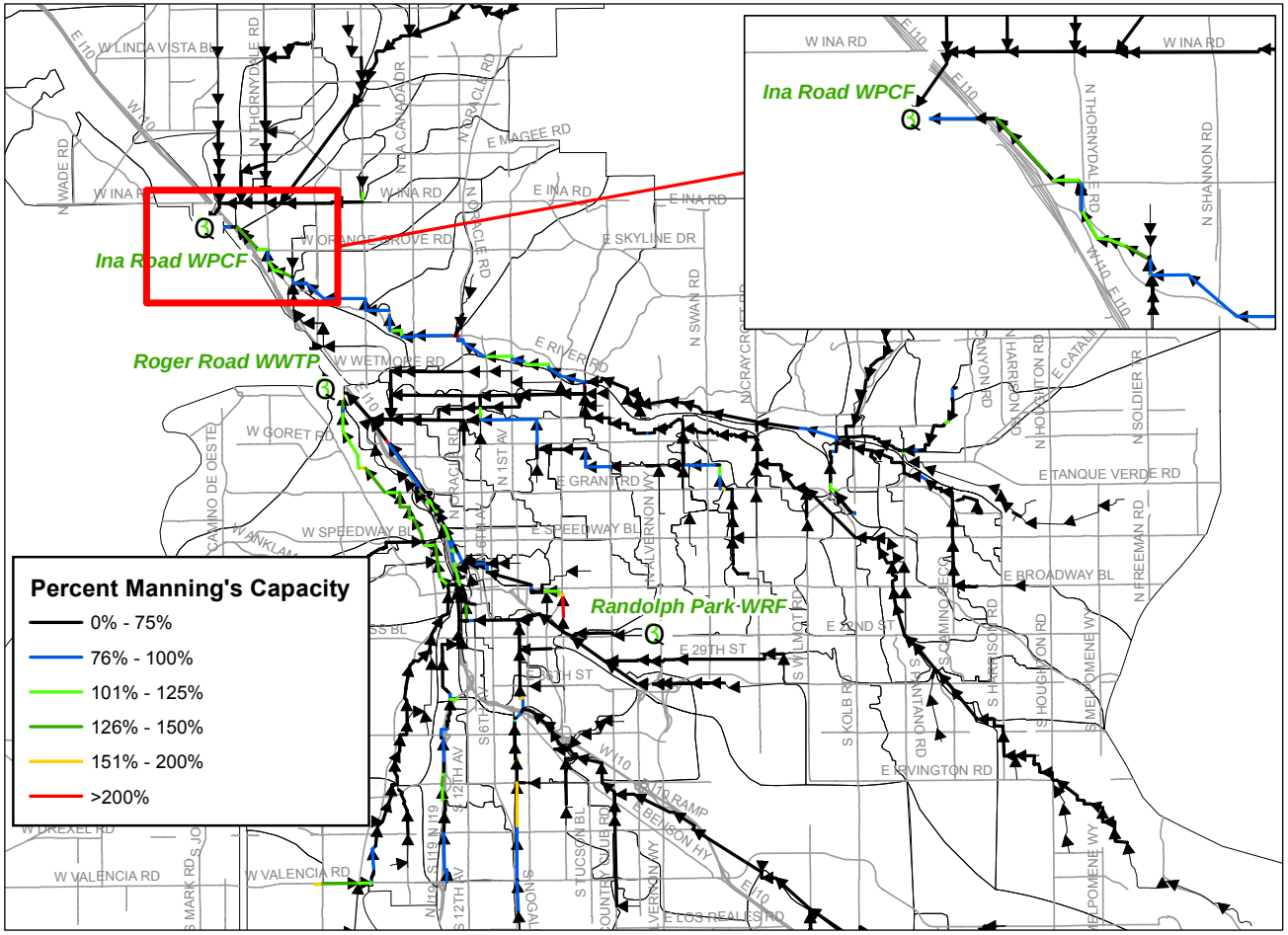


Figure 6-15
2005 Wet Weather Hydraulic Evaluation



6.5.3.1 2030 System Hydraulic Evaluation

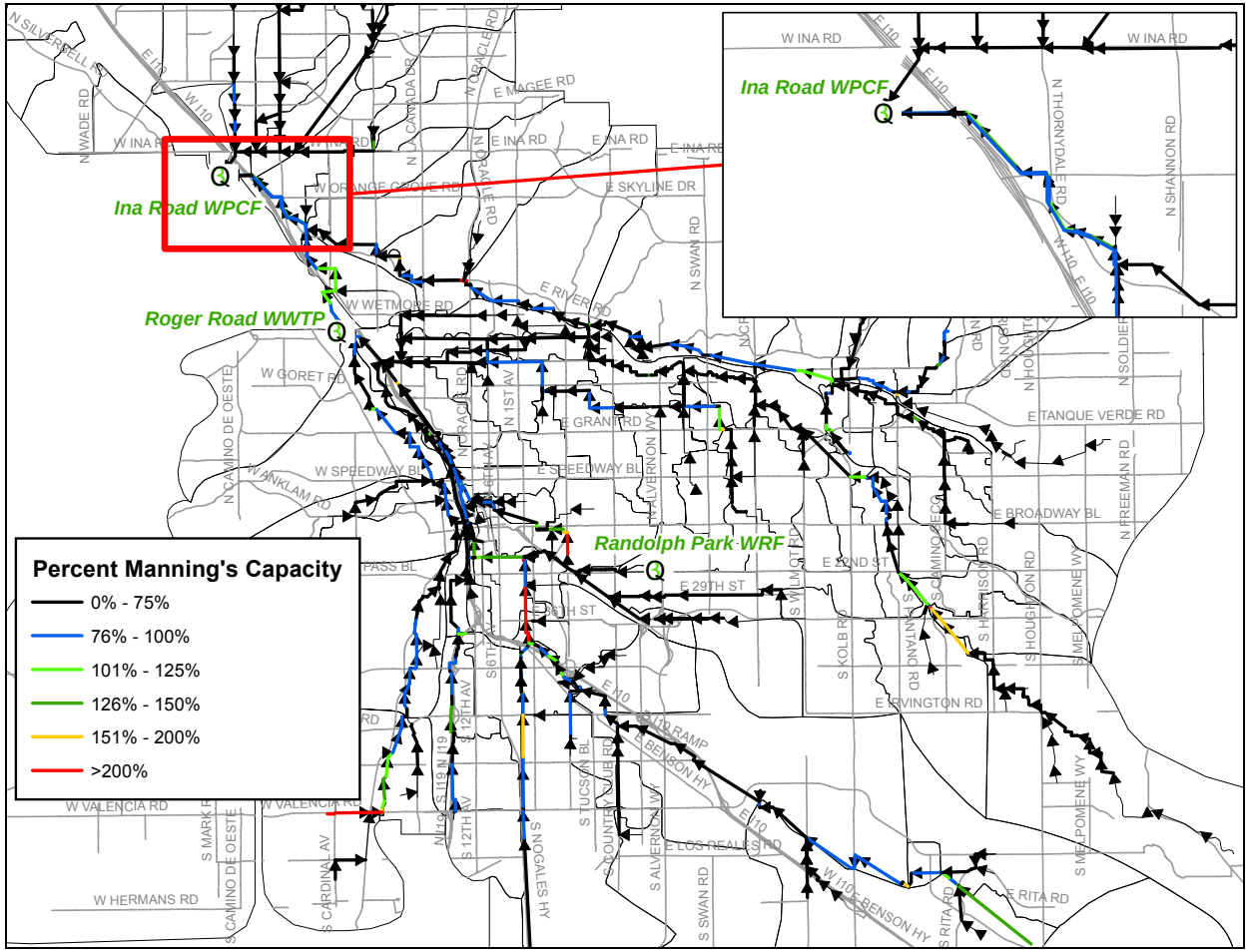
For this analysis a slightly more conservative approach was taken for the conveyance capacity evaluation than was used within the 2006 Facility Plan analysis. First of all, the Facility Plan presumed that as the Ina Road WRF service population expands, the per capita rates would remain at their 2005 levels. In addition, the Facility Plan presumed that those customers on septic would remain so in 2030. For this analysis, however, a per capita flow rate of 85 gpcd was used and all customers and those currently on septic were presumed to be served by PCRWRD by the year 2030. Finally, the revised peaking factor curve accounting for wet weather was used to compare estimated 2030 peak wet weather flows to pipe capacities.

Results of the hydraulic evaluation are presented on **Figure 6-16**, with pipes color coded according to the ratio of peak 10-year flow to pipe capacity. Non-Metro reaches of the SEI and PTI interceptors have inadequate capacities to convey the 10-year wet weather flows. Additional capacity will have to be provided for approximately 6 miles and 2.3 miles of these sewers, respectively. Of particular concern is the 2-mile stretch of the SEI flowing north from Interstate 10 (I-10) to 18th. This stretch of sewer will not have adequate capacity to pass the 2030 dry weather flows, and widespread sewer backups could occur during rainfall events.

From the model results an optimal flow balance will not be achieved between the existing Santa Cruz and new Santa Cruz interceptors. This is due in large part to the hydraulics of the structures at Alameda that divide the flows between these interceptors. This need not be a cause for great concern however, because as flows back up in one interceptor, it will push additional flow into the other, thereby balancing the flows during an actual peak wet weather event.

The North Rillito interceptor also appears to have some capacity issues in the year 2030. However, these could be mitigated by reversing the flow in the Tucson Boulevard diversion or Craycroft diversion to send additional flow to the SRI, which has excess capacity. Flow could then be transferred from Roger Road WRF service area to Ina Road WRF via the plant interconnect pipeline. The optimal balance of flow between the NRI and SRI is ultimately dependent on the route selected for the plant interconnect pipeline.

Figure 6-16
2030 Wet Weather Hydraulic Evaluation



6.5.3.2 Houghton Area Master Plan (HAMP)

A detailed hydraulic study of the Houghton Area Master Plan (HAMP) development was performed to analyze the impact that it will likely have on the flows in the interceptors that are expected to receive the additional flow.

A study report on the HAMP for the year 2030 included two sub-basin configuration scenarios. The first configuration matches that of the 2006 Facility Plan, while the second scenario redirects a portion of Basins 43B and 97 into Basin 113, with the resultant flow being redirected from the SEI to the Pantano Interceptor (PTI). This modification changes the populations that are connected to the SEI and PTI interceptors which were consequently updated in the model. The second scenario was replicated in the hydraulic model by overlaying TAZ and the revised sewer basins boundaries and updating the 2030 sub-basin populations accordingly. The model was then re-run and the output analyzed.

Results of scenario 1, which match the Facility Plan sub-basin delineation, is presented on **Figure 6-17**. Those pipe segments that are expected to be flowing at above their design capacities are highlighted. The corresponding labels indicate the pipes' expected 2030 flows, and the percent increases in capacity that would need to be provided to convey these flows with an appropriate peaking factor. Similarly, **Figure 6-18** indicates the simulation results with a portion of the HAMP flows redirected to the PTI. Under this scenario only the PTI would require an increase in capacity.

The overall effect of the second scenario is that flow is unloaded from the Southeast Interceptor and is re-routed into the PTI. While this proposal relieves the Southeast Interceptor of a significant portion of overloading, it does cause a large amount of the flow to backup in the PTI. Portions of the sewer limit the conveyance capacity and would need to be upgraded.

Figure 6-17
HAMP Capacity Analysis, SEI Scenario

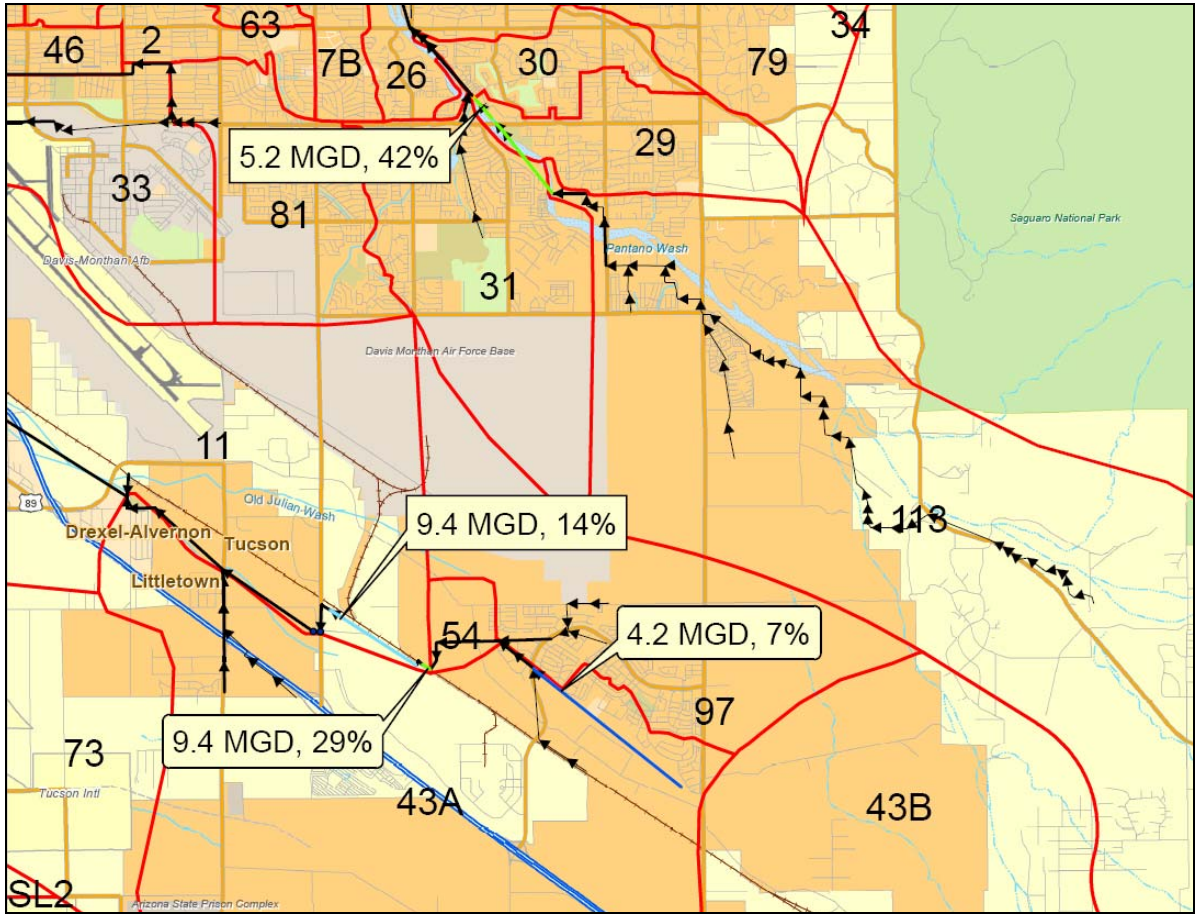
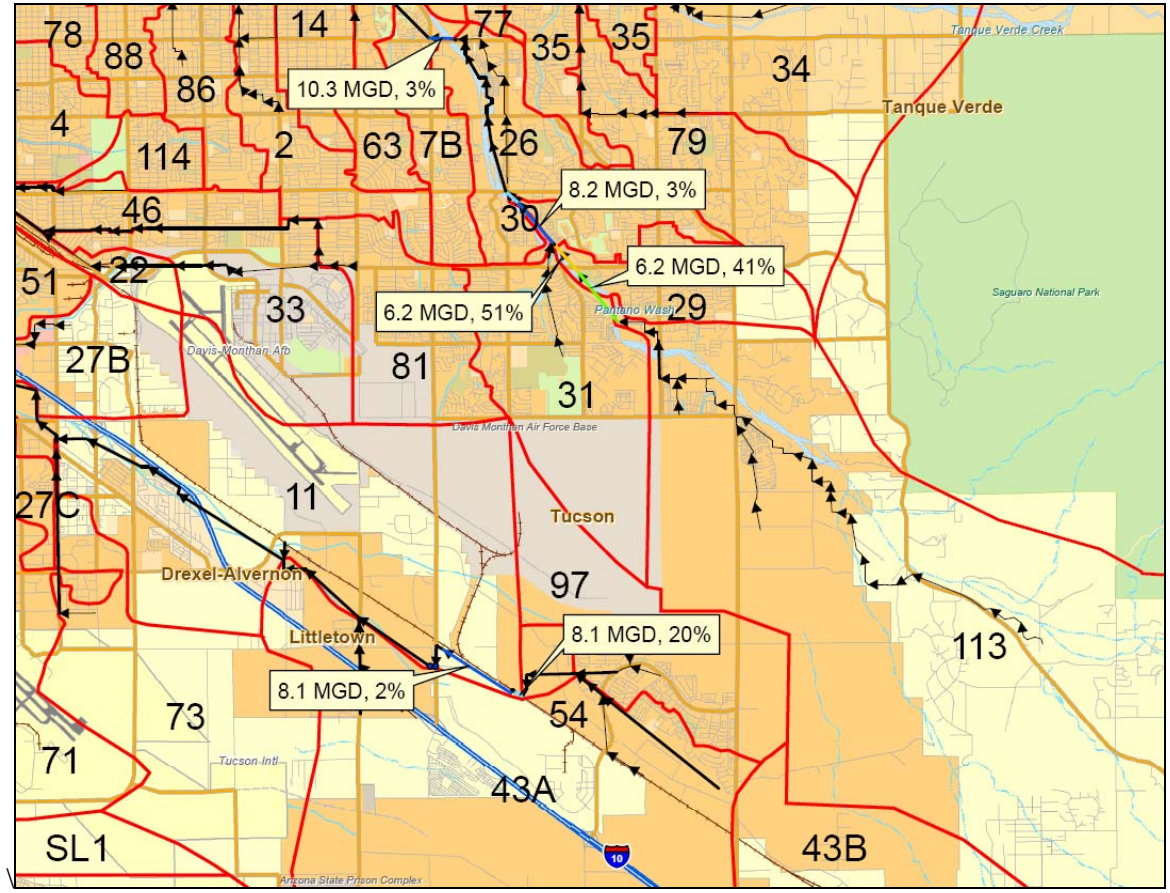


Figure 6-18
HAMP Capacity Analysis, PTI Scenario



6.5.4 Plant Interconnect

Ability to transfer flow between the Roger Road service area and Ina Road WRF is critical for the management of wastewater to accommodate growth. Based on current growth projections, it is anticipated that Roger Road WRF's capacity will be reached by the year 2011, or 2012. This interconnect, or some other method of transferring supplemental flows, to the Ina Road WRF must be placed into operation by this time. Because of the capacity limitations at Roger Road WRF, there is urgency in advancing the construction of the plant interconnect pipeline (Santa Cruz Interceptor Phase IV).

Three alternatives were given consideration for transferring flow between facilities. These include implementing the "Existing" plan specified in the 2006 Facility Plan, transferring "some" flow between the two facilities, and transferring all flow from the Roger Road service area to Ina Road WRF. The "Existing" plan is the selected option that best fits within the overall wastewater plan.

Regardless of which alternative is selected, there are effluent demands that must be met. For example, Tucson Water has stated that it requires 30 mgd at its Sweetwater facility for reclaimed water use. In addition, a minimum of 2 mgd to the Santa Cruz River may be required for maintenance of riparian habitat. Therefore, any alternative which involves treating less than 32 mgd at the Roger Road facility will require that the balance be pumped back from Ina Road WRF to supplement the effluent needs at that location.

6.5.4.1 Routing Alternatives

There are four route alternatives that were given consideration for conveying the wastewater from the Roger Road service area to the Ina Road WRF as shown on **Figure 6-19**. These are:

- Alternative 1 – Parallel existing sludge force main interconnect;
- Alternative 2 – Silverbell Road;
- Alternative 3 – Prince and Romero to El Camino Del Terra and De La Tierra;
- Alternative 4 – Alternative 3 route to El Camino Del Cerro, under I-10, then Alternative 1.

Alternative 3 makes it difficult to transfer the flow in the Santa Cruz interceptors and impossible to capture flows in the NWO, therefore it was eliminated from further consideration.

Which route is selected has a bearing on the design capacity of the plant interconnect pipeline. The reason is that the third and fourth alternatives involve joining the plant interconnect pipeline and the NRI at W El Camino Del Terra and N De La Tierra, and "augmenting" the NRI from that point to the Ina Road plant. Under this scenario that upstream portions of the NRI would need to be increased in order to accommodate the 10-year peak flow. However, additional capacity requirements are slight (generally less than 10 percent), and peak flows could likely be forced through with a minimum of surcharging until the point where the interconnect would meet it. From this point to the Ina Road plant, referred to as "Augmentation", the existing NRI will not have adequate capacity to convey the 10-year storm in 2030.

Therefore, a portion of its flow (13 mgd) would need to be transferred into the new plant interconnect pipeline, which would be sized to accommodate the additional flow.

The other two routing alternatives do not utilize any portion of the NRI. Therefore, in order to alleviate the capacity limitations in the downstream reaches of the NRI.Figure 6-16, the Tucson Boulevard diversion or Craycroft diversion (but not both) could be reversed in order to transfer flow from the NRI to the SRI, which has excess capacity. Once the transferred flow reaches the Roger Road plant site it could be conveyed to Ina Road WRF via the plant interconnect pipeline. Under this scenario, 13 mgd of the peak 10-year flow would need to be transferred from the NRI to the SRI, roughly half the flow in the NRI at this point.

The existing sewer configuration prevents a reversal of flow through either flow management structures to accomplish the diversion. Two alternatives were developed for reversing the diversion of flow through either the Tucson Boulevard or Craycroft diversion structures. The locations of these facilities are depicted on **Figure 6-20**. For Tucson Boulevard approximately 2150 feet of 36-inch pipe would need to be constructed in order to direct flow from the NRI to the SRI as indicated on Figure 6-21. This distance does not include the siphon under Rillito Creek, which could likely be used as is. Two options were considered for reversing the flow through the Craycroft structure (**Figure 6-22**). Option 2 is not considered a viable alternative as this would require surcharging the upstream collection system in order to divert the 13 mgd required. Option 1 would require constructing approximately 2950 feet of 30-inch pipe. The costs of the Tucson Boulevard and Craycroft - Option 1 alternatives are approximately $1.1M and $1.26M, respectively.

Figure 6-19
Plant Interconnect Routing Alternatives

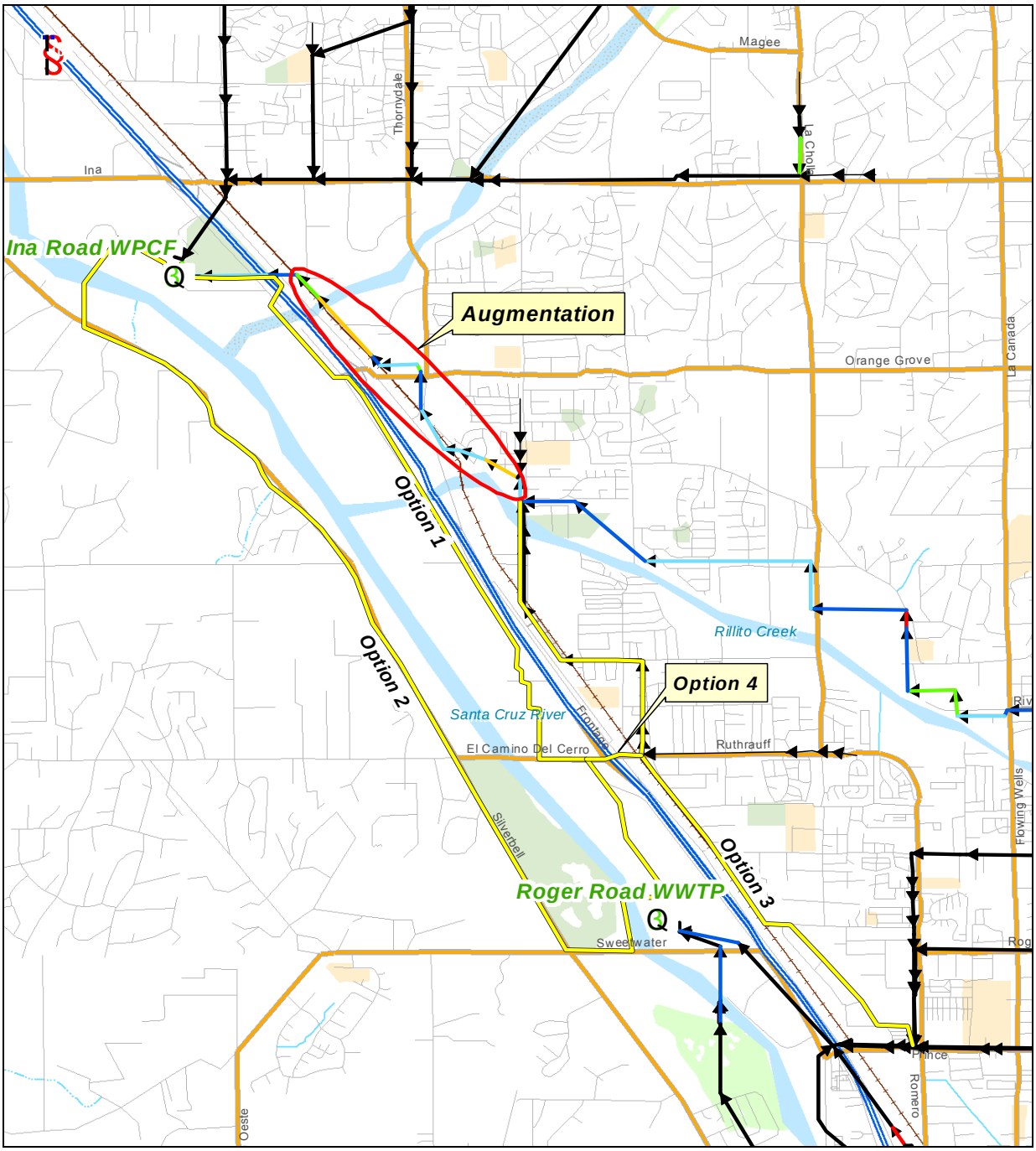


Figure 6-20
Tucson Boulevard and Craycroft Diversion Locations

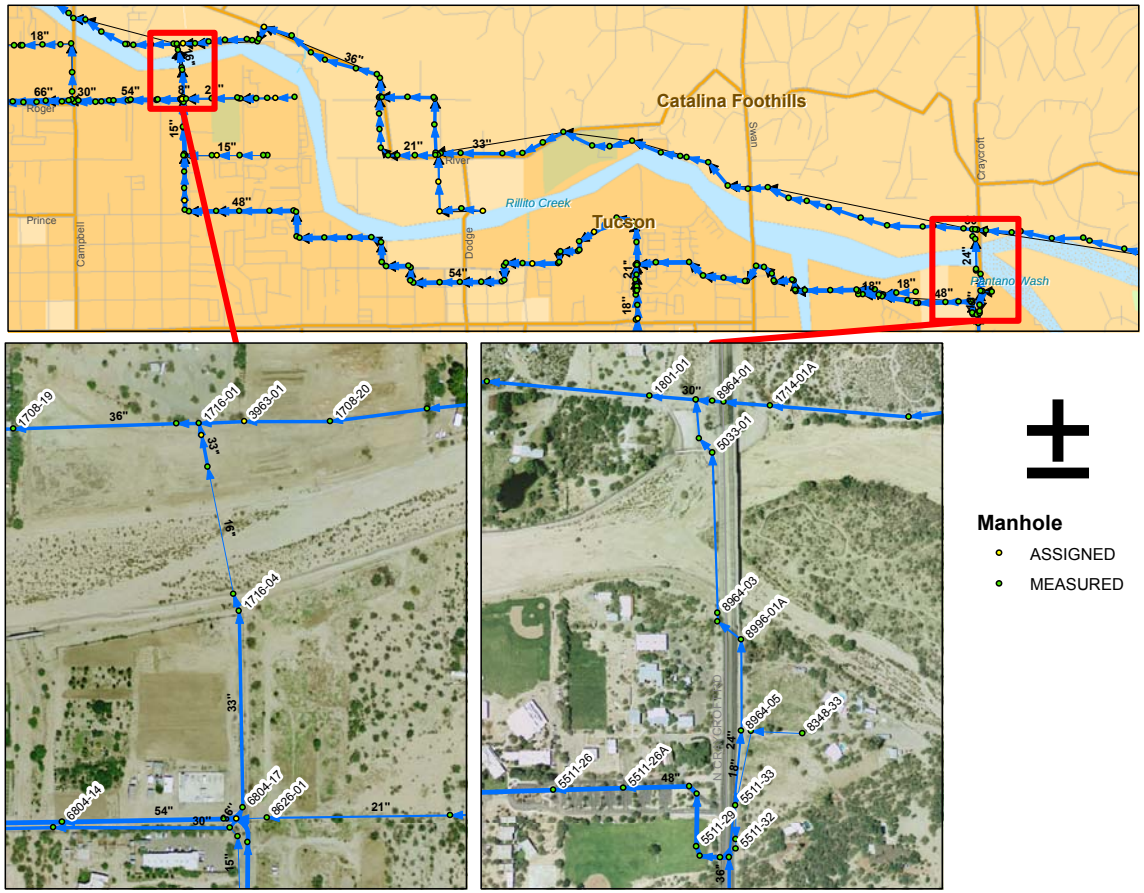


Figure 6-21
Tucson Boulevard 36-Inch Reverse Sewer

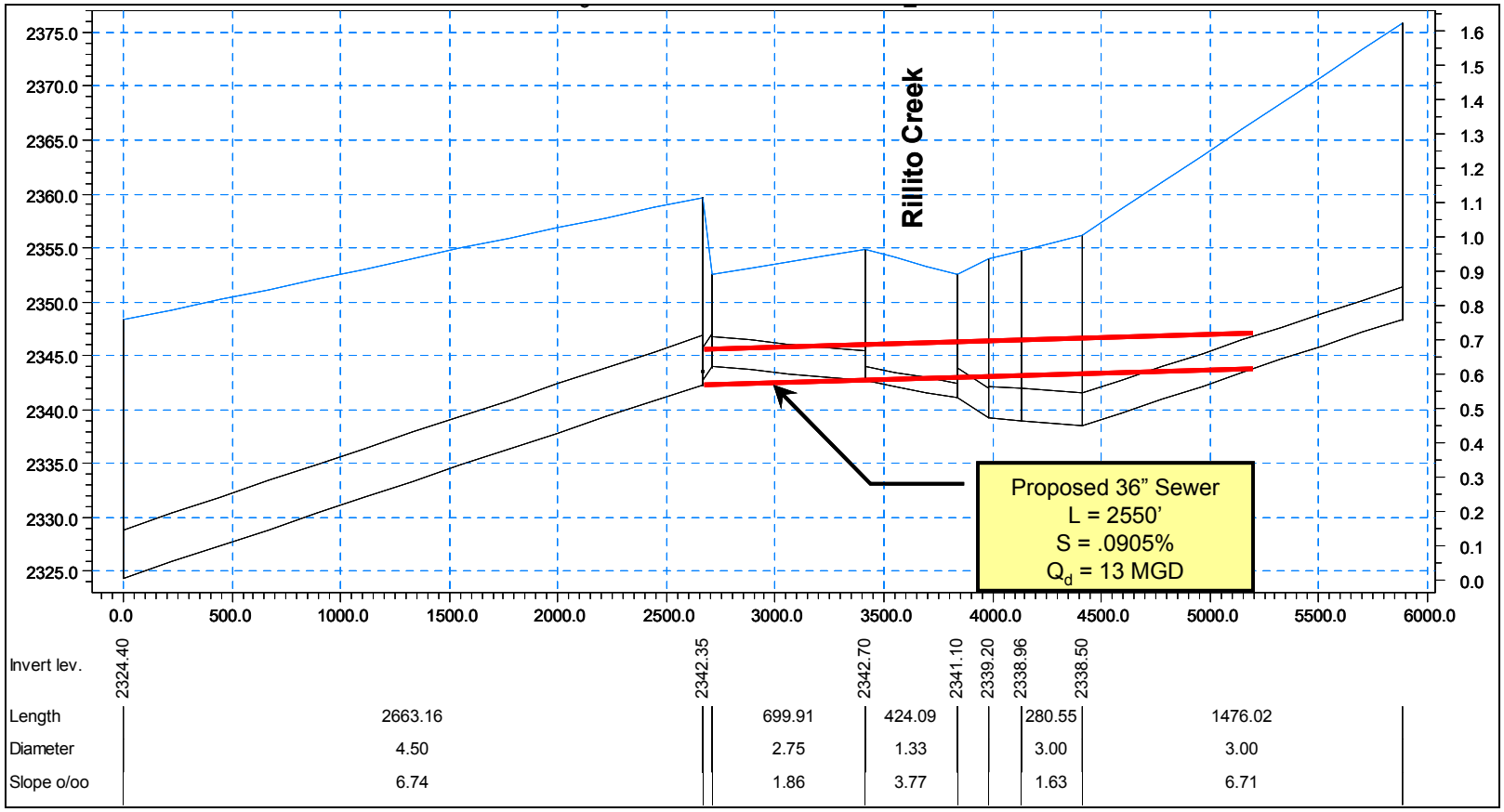
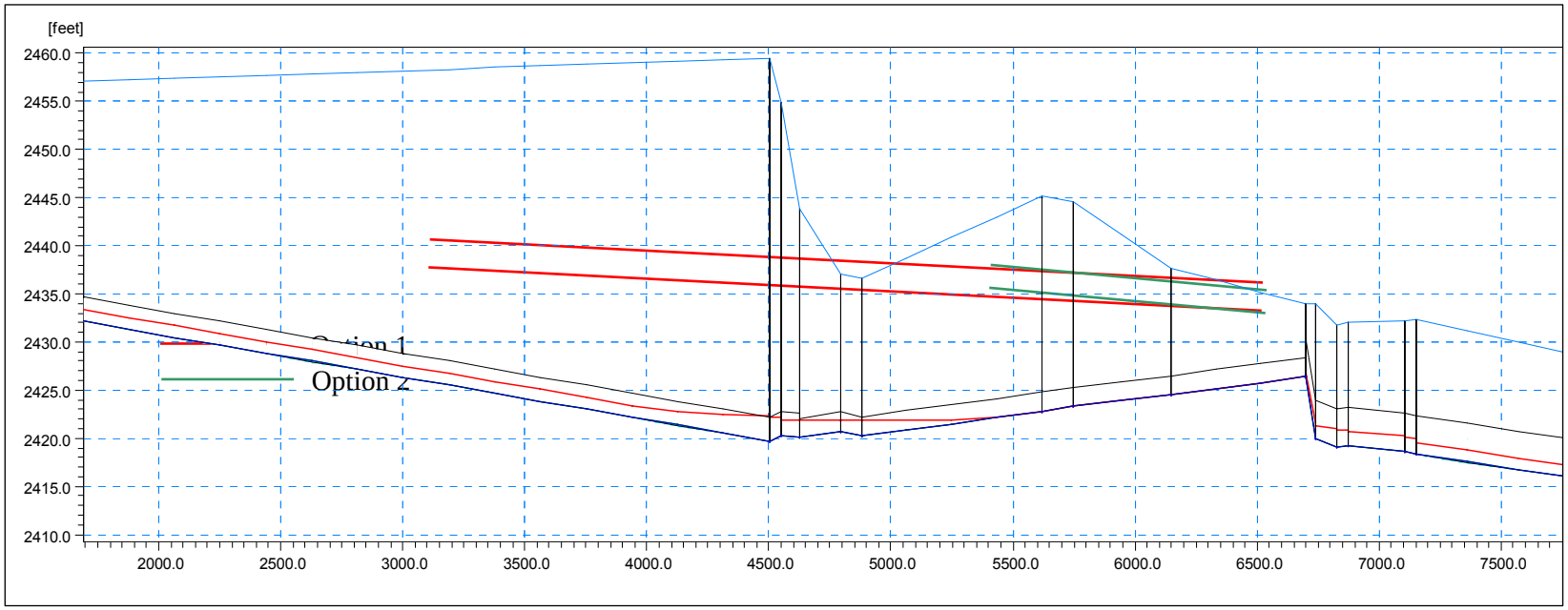


Figure 6-22
Craycroft 30-Inch Reverse Sewer



6.5.4.2 "Existing" Plan

The interconnect configuration for the existing plan calls for treating 32 mgd of the 60 mgd year 2030 ADWF at Roger Road WRF and transferring the remaining 28 mgd from the Roger Road service area to Ina Road WRF. However, these are average daily flows and do not consider peaking factors and wet weather impacts. To the ADWF of 60 mgd a peaking factor of 1.38 needs to be applied. With a service area of ~200,000 acres an additional 54 mgd of wet weather flow is added to be treated either at Roger Road WRF or conveyed to Ina Road WRF. Presuming the Roger Road plant will have a peaking factor of 2, the division of dry and wet weather flows between Roger Road WRF and Ina Road WRF could be as indicated in **Table 6-5**. This operational configuration would enable Roger Road WRF to operate in a steady-state mode during dry weather by transferring all dry weather flow diurnal variability to Ina Road WRF. In this operational configuration, in the year 2030, 3 to 50 mgd would be transferred to Ina Road WRF during dry weather. This transfer could increase to 72 mgd during wet weather. If additional flow is transferred to Roger Road WRF to alleviate the NRI, then the plant interconnect pipeline would require an additional 13 mgd (85 mgd total) of capacity. This was the target capacity used in the design of routing alternatives 1 and 2.

Table 6-5
2030 Flow Split, "Existing" Plan, Roger Road WRF Steady State

	Q_{avg} (MGD)	Q_{min} (MGD)	Q_{max} (MGD)	$Q_{WetWeather}$ (MGD)	Q_{Peak} (MGD)
Roger Road	32	32	32	32	64
Transfer To Ina Road	28	3	50	22	72
Total	60	35	82	54	136

Another operational alternative could be to split the variability of dry weather flows between the Roger Road WRF and Ina Road WRF. This option would result in 2030 dry weather flows transferred to Ina Road t WRF o range from 17.5 to 41 mgd. However, as **Table 6-6** indicates, when wet weather flows are considered the design capacity of the plant interconnect pipeline becomes independent of the operational configuration of Roger Road.

Table 6-6
2030 Flow Split, "Existing" Plan, Roger Road WRF Variable Flow

	Q_{avg} (MGD)	Q_{min} (MGD)	Q_{max} (MGD)	$Q_{WetWeather}$ (MGD)	Q_{Peak} (MGD)
Roger Road	32	17.5	41	23	64
Transfer To Ina Road	28	17.5	41	31	72
Total	60	35	82	54	136

If either routing alternatives 1 or 2 are selected, the plant interconnect pipeline will need to have an additional 13 mgd of capacity in order to accommodate the transfer of flow from the NRI, resulting in a design capacity of 85 mgd. If routing option 4 is selected, the pipe would require a design capacity of 72 mgd until it reaches the NRI, at which point its capacity would need to be increased to 85 mgd to alleviate overloading of the NRI.

6.5.4.3 Transfer Some

This alternative is similar to the "Existing" alternative, but calls for treating 20 mgd of ADWF at Roger Road WRF. A delineation of flows treated at Roger Road WRF and to be transferred to Ina Road WRF is indicated in **Table 6-7**. A treatment and wastewater transfer analysis has determined that this option is not economically viable.

Table 6-7
2030 Plant Flow Split, Transfer Some

	Qavg (mgd)	Qmin (mgd)	Qmax (mgd)	Q (Wet Weather) (mgd)	Q peak (mgd)
Roger Road WRF	20	12	26	14	40
Transfer to Ina	40	23	56	40	96
Total	60	35	82	54	136

6.5.4.4 Transfer All

Under this alternative the Roger Road WRF would be decommissioned. Therefore all peak dry and wet weather flows would need to be transferred from the Roger Road service area to Ina Road WRF. The resultant design capacity of the plant interconnect pipeline is 136 mgd.

If this alternative is selected, provisions will need to be provided to pump up to 37 mgd from Ina Road WRF back to Tucson Water's Sweetwater facility and the Santa Cruz River fro riparian habitat. The existing reclaim pipeline network does not have sufficient capacity to convey this volume of flow, so a new pump station and force main would be required. To reduce construction costs, this force main should be designed and constructed at the same time and in the same trench with the plant interconnect pipeline.

It is further noted that Tucson Water intends to construct additional infiltration basins on City of Tucson property along Interstate 1-10 north of Sweetwater Drive. The placement of the additional basins must be considered in the alignment of the plant interconnect pipeline.

6.5.4.5 Summary of Interconnect Alternatives

Preliminary designs of Routes 1, 2, and 4 under the "Existing" plan option were developed. These are indicated on **Figure 6-23**, **Figure 6-24**, and **Figure 6-25**. Pipes were sized according to the capacity requirements described above. Ground elevation profiles were derived from Pima County contour and digital elevation map (DEM) data. Pipe sizes and slopes were set to follow grade to the maximum extent practicable. Siphons were avoided to the maximum extent possible, because they can cause operational difficulties. Nevertheless, all routes will require at least one siphon.

Pipe profiles include information on scouring flows for each segment of pipeline. This is the minimum flow required to maintain a scouring velocity of 2.5 fps in the pipe. At flows less than the scour flow sedimentation is more likely to occur and accumulate within the system.

Figure 6-23
Alternative 1 Plant Interconnect, Existing Sludge Force Main Route Plan

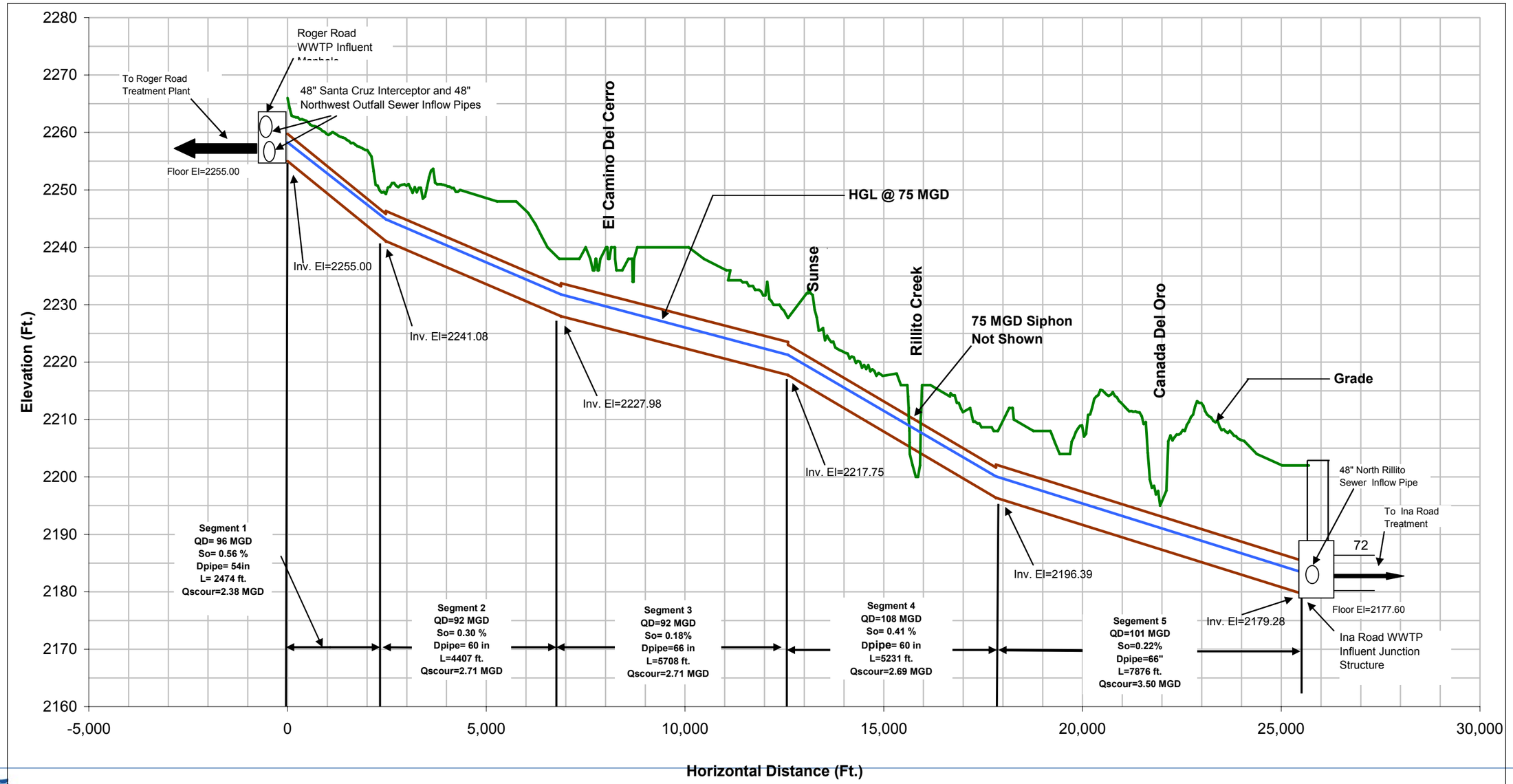


Figure 6-24
Alternative 2 Plant Interconnect, Silverbell Road Route

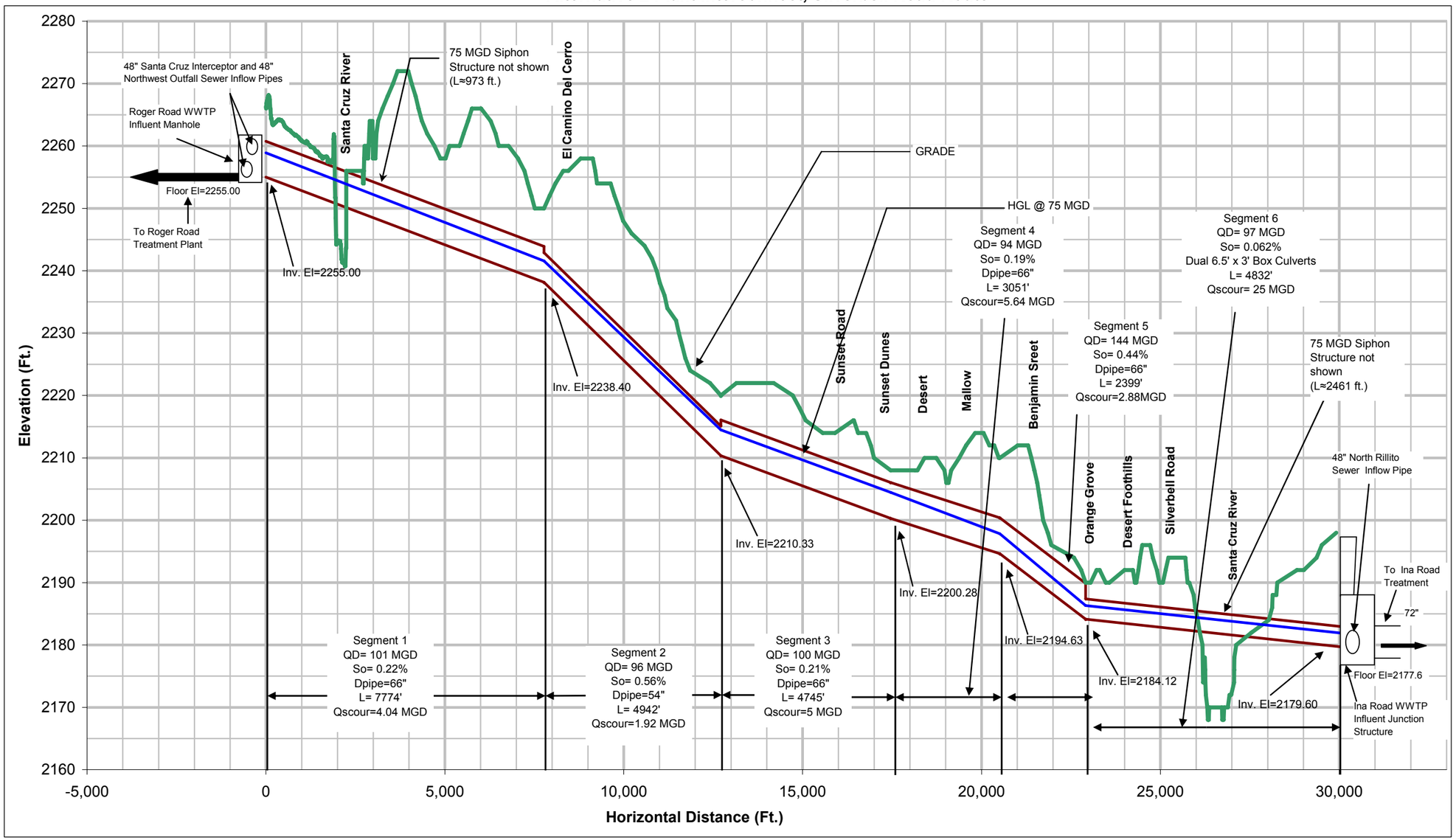
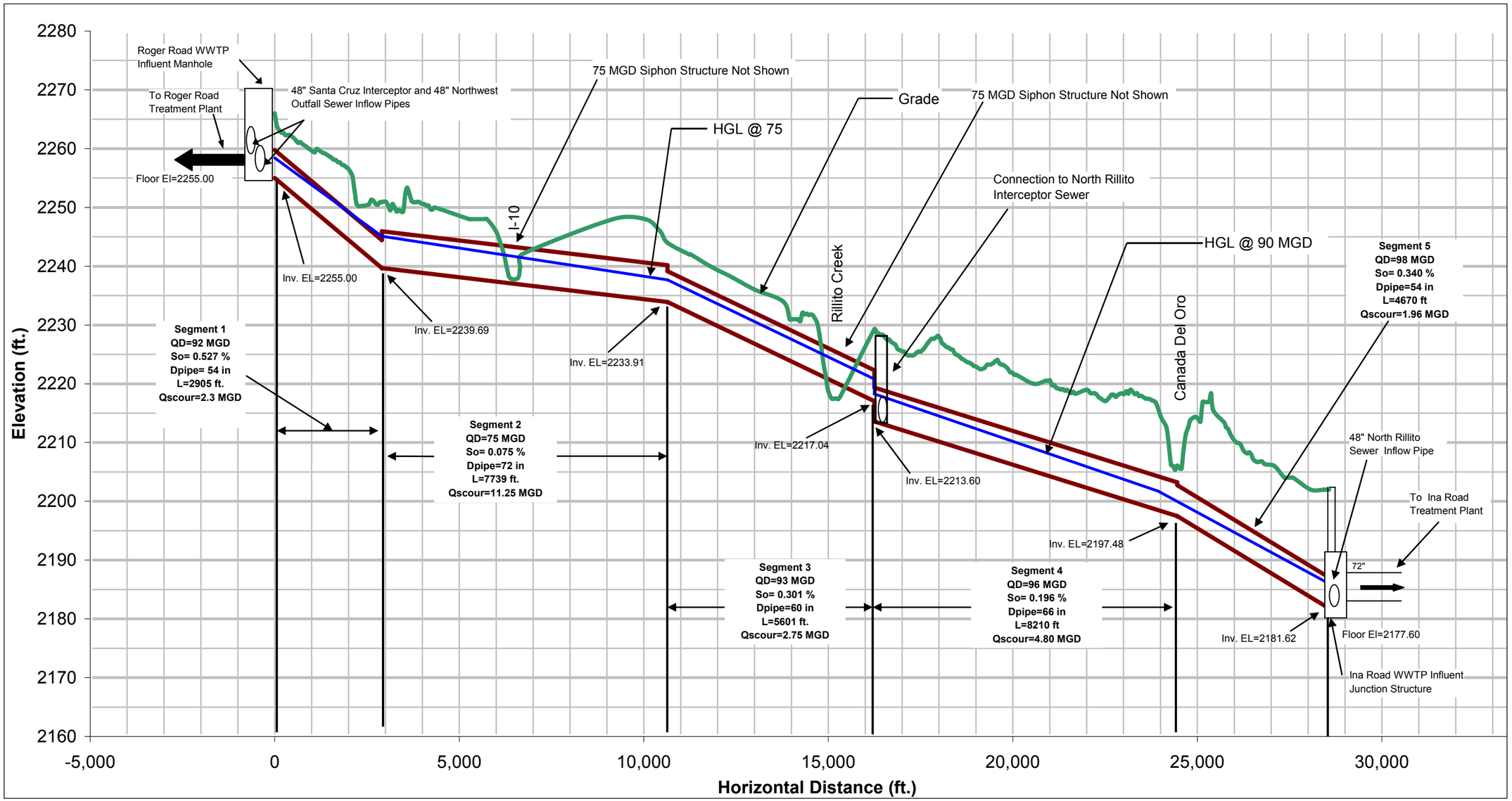


Figure 6-25
Alternative 3 Plant Interconnect, Combine Alternative 1 and 3 Route



Findings of the plant interconnect pipeline analyses is provided in **Table 6-8**. Costs were derived using the unit costs described in a later subchapter of this chapter. These are for construction costs only, and do not include engineering or contingency costs for purchase of right-or-way or realignment of the existing sludge force main in the Route 1 alternative.

Table 6-8
Summary of Interconnect Alternatives

	Route	Pipe Size	Cost (millions)	Not Included
Existing Plan	Route 1	54" - 66"	$18.90	R.O.W. acquisition
	Route 2	54" - 66"	$29.60	
	Route 4	54" - 72"	$28.00	
Transfer Some	Route 1	54" - 60"	$17.80	R.O.W. acquisition
	Route 2	54" - 66"	$28.90	
	Route 4	48" - 66"	$25.90	
Transfer All	Route 1	66" - 96"	$26.70	R.O.W. acquisition
	Route 4	72" - 84"	$37.10	

6.5.4.6 Interim Solution – Transfer of Additional Flow to Ina Road WRF

Plant capacity is anticipated to be exceeded at the Roger Road WRF in the year 2011 or 2012. The plant interconnect pipeline should be operational by then, although an interim solution for transferring additional flow to Ina Road WRF may be necessary as a contingency. While not preferable due to the limited capacity of the NRI, analyses revealed that as much as 12 mgd of additional flow could be diverted from the SRI to the NRI during dry weather. This capacity for dry weather flow may provide up to an additional 6 mgd of transfer flow to Ina Road WRF. This may serve as an interim solution during the design and construction of the plant interconnect pipeline, if needed.

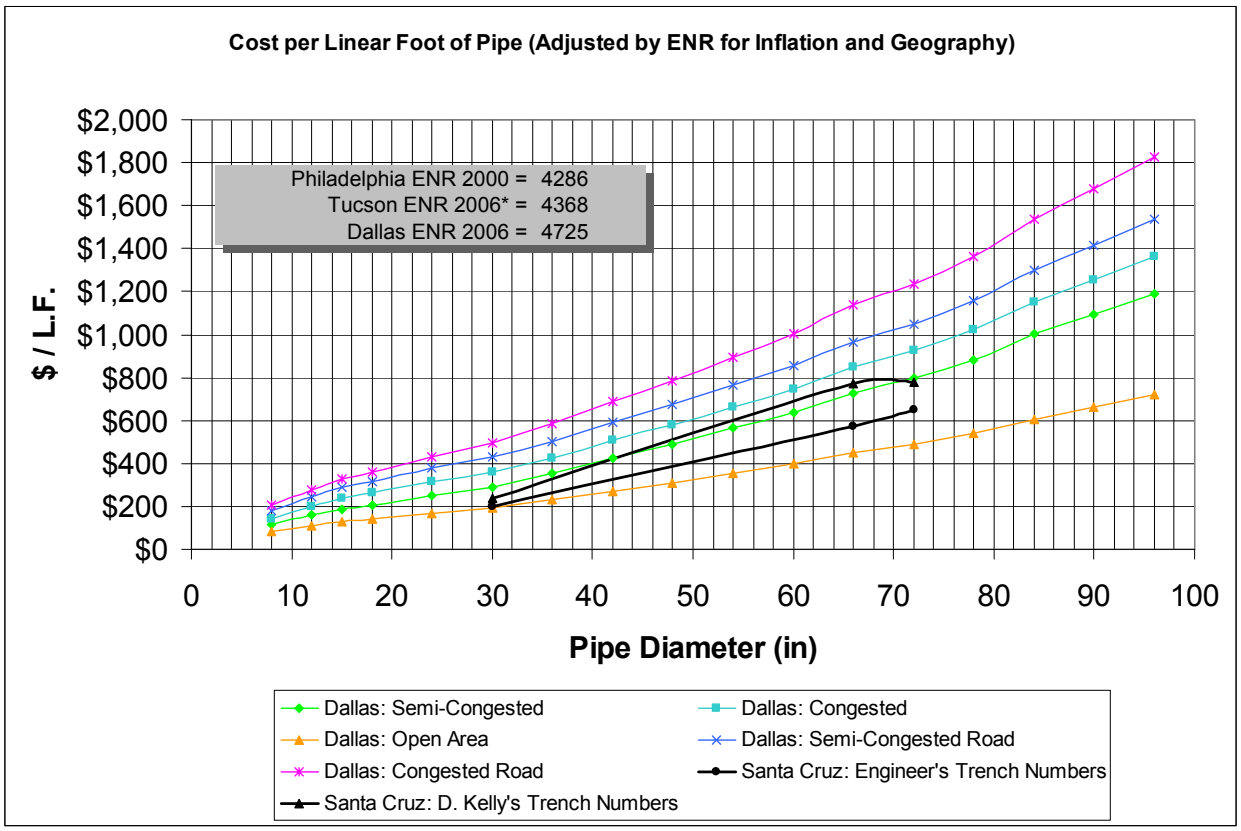
However, this transfer of additional flow should only take place during periods of dry weather. Wet weather flows can quickly exceed the capacity of the NRI contributing to sewer surcharging. Therefore, the diversion of the additional flow would need to be limited to dry weather, or when the NRI has

adequate capacity available. Control of the diversion of flow from the SRI to the NRI may require the installation of a set of gate operators and controls to avoid overloading the NRI.

6.6 Cost Comparisons

Preliminary cost curves were developed which depict pipe unit costs vs. size. The result is a family of curves with each cost curve representing a uniform level of construction "congestion" (**Figure 6-26**). These cost curves had originally been developed in 2000 for the City of Richmond, Virginia, with the nearest ENR Index city being Philadelphia. These unit costs were converted to the 2006 values of the ENR Index city of Dallas, Texas, which represents similar cost factors to Pima County. Finally, these curves were calibrated to match the lowest bid for the construction of the new Santa Cruz interceptor. This was done by creating a Tucson ENR Index which was adjusted to obtain a reasonable fit between the "semi-congested" cost curve and the lowest construction cost bid tab.

Figure 6-26
Sewer Construction Cost Curves



For expediency in developing cost estimates, pipe costs were simplified into a $/inch*ft basis. The result is a single unit cost for each level of congestion as indicated in
Table 6-9. The weighted average value at the bottom of each column was used in the development of all sewer costs.

Table 6-9
Sewer Construction Unit Costs

	$ / (inch*ft)				
D_{pipe} (in)	Semi-Congested	Congested	Open Area	Semi-Congested Rd.	Congested Road
8	14.32	18.06	10.22	22.52	25.44
12	13.37	16.76	8.92	20.29	23.08
15	12.58	15.88	8.42	19.17	21.84
18	11.55	14.56	7.78	17.64	20.17
24	10.53	13.08	6.93	15.70	18.07
30	9.74	11.98	6.53	14.28	16.51
36	9.77	11.86	6.43	14.01	16.28
42	10.05	12.05	6.46	14.02	16.33
48	10.20	12.12	6.45	14.00	16.36
54	10.43	12.30	6.53	14.13	16.56
60	10.61	12.44	6.60	14.25	16.74
66	11.06	12.88	6.78	14.66	17.25
72	11.04	12.82	6.75	14.56	17.16
78	11.33	13.09	6.90	14.83	17.50
84	11.91	13.70	7.19	15.46	18.28
90	12.17	13.96	7.36	15.74	18.64
96	12.41	14.19	7.53	16.02	19.00
Weighted Average =	**$11.26**	**$13.22**	**$7.01**	**$15.18**	**$17.81**

6.7 Capital Improvement Plan

The hydraulic analyses and the unit costs development were compiled to develop a CIP for the conveyance system over the 25-year planning period. Recommended system improvements are indicated on **Figure 6-27**. The 2030 wet weather flow simulation was the primary means of determining which sewers would require upsizing. However, not every pipe expected to flow at over 100 percent of design capacity in the year 2030 was recommended for replacement. From experience it was presumed that pipes could carry more than their design capacities with an acceptable amount of surcharging in a 10-year event. Only those expected to carry over 125 percent of their design capacity during a 10-year storm and having a length of at least 500' were recommended for replacement.

Corresponding costs are detailed in **Table 6-10**. The costs are provided in 2006 dollars and include an allowance for engineering at 15 percent and a contingency allowance of 30 percent. Years in which these system improvements are forecast to be constructed are included on the table.

Figure 6-27
Capital Improvement Projects

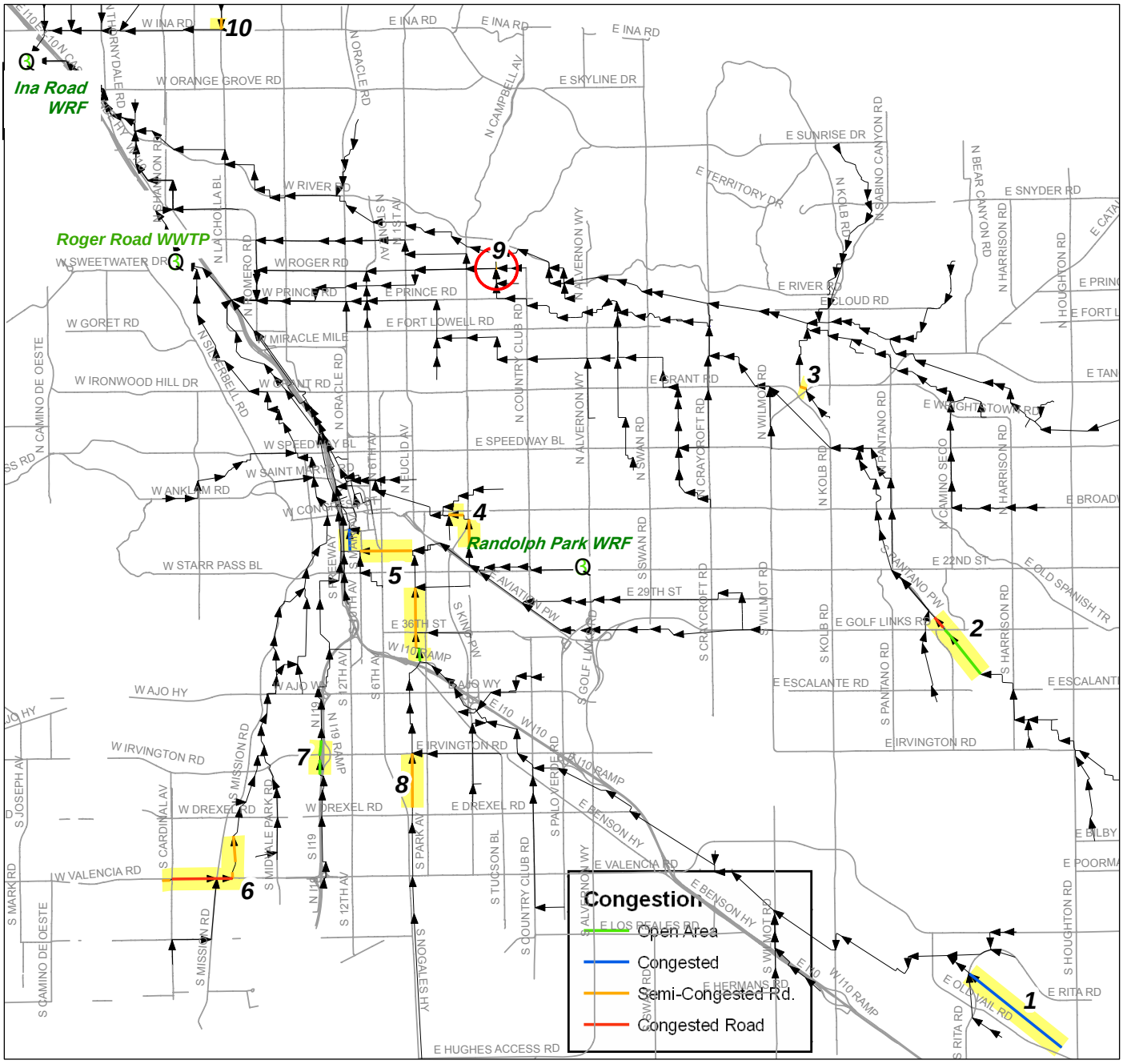


Table 6-10
Capital Improvement Plan Costs

Project	MH Start	MH End	Year	Item	Qty.	Units	Unit Cost	Costs				Type
								Constr. (X1,000)	Eng. Services (X1,000)	Contingency (X1,000)	Total (X1,000)	
1	4636-30A	4190-05A	2020	Replace 18" pipe with 21"	10,336	FT	$290.22	$3,000	$450	$900	$4,350	Congested
				Project 1 Subtotal							*$4,350*	
2	6036-21	3979-101	2015	Replace 18" pipe with 24"	1,183	FT	$433.58	$513	$77	$150	$740	Congested Road
	3919-01	8031-01		Replace 18" pipe with 24"	5,486	FT	$166.33	$913	$137	$270	$1,320	Open Area
				Project 2 Subtotal	*6,669*						*$2,060*	
3	8635-11	8635-10	2020	Replace 18" pipe with 21"	620	FT	$350.02	$217	$33	$60	$310	Semi-Congested Road
				Project 3 Subtotal	620						*$310*	
4	5627-08A	9907-49	2010	Replace 18" pipe with 21"	5,050	FT	$376.69	$1,902	$285	$573	$2,760	Semi-Congested Road
				Project 4 Subtotal	*5,050*						*$2,760*	
5	8130-01	1779-02	2015	Replace 30" pipe with 42"	1,153	FT	$271.29	$313	$47	$90	$450	Open Area
	9917-20	9910-21		Replace 33" pipe with 36"	5,402	FT	$504.30	$2,724	$409	$817	$3,950	Semi-Congested Road
	5667-01	5662-01		Replace 30" pipe with 42"	3,875	FT	$588.64	$2,281	$342	$687	$3,310	Semi-Congested Road
	1779-02	5667-01		Replace 30" pipe with 42"	1,415	FT	$588.64	$833	$125	$252	$1,210	Semi-Congested Road
	9910-21	9910-19		Replace 36" pipe with 42"	550	FT	$506.07	$278	$42	$80	$400	Congested
				Project 5 Subtotal	*12,395*						*$9,320*	
6	8149-05	1726-29	2010	Replace 15" pipe with 21"	3,943	FT	$401.46	$1,583	$237	$480	$2,300	Congested Road
	3983-05	8149-05		Replace 15" pipe with 24"	887	FT	$376.69	$334	$50	$96	$480	Semi-Congested Road
	1726-29	1726-26		Replace 18" pipe with 24"	1,388	FT	$433.58	$602	$90	$178	$870	Congested Road
	1726-26	1726-19		Replace 24" pipe with 30"	3,776	FT	$428.37	$1,618	$243	$489	$2,350	Semi-Congested Road
				Project 6 Subtotal	*9,994*						*$6,000*	

Table 6-10 (Continued)
Capital Improvement Plan Costs

Project	MH Start	MH End	Year	Item	Qty.	Units	Unit Cost	Costs				Type
								Constr. (X1,000)	Eng. Services (X1,000)	Contingency (X1,000)	Total (X1,000)	
7	4630-09	4630-02	2015	Replace 21" pipe with 24"	3,013	FT	$166.33	$501	$75	$154	$730	Open Area
				Project 7 Subtotal	*9,994*						*$730*	
8	1751-09	1751-01	2010	Replace 18" pipe with 24"	4,712	FT	$376.69	$1,755	$266	$529	$2,570	Semi-Congested Road
				Project 8 Subtotal	*4,712*						*$2,570*	
9	8626-01	6804-15A	2010	Replace 8" pipe with 15"	100	FT	$287.54	$29	$4	$7	$40	Semi-Congested Road
				Project 9 Subtotal	*100*						*$40*	
10	9521-02	9549-05	2020	Replace 15" pipe with 18"	1,006	FT	$317.51	$319	$48	$93	$460	Semi-Congested Road
				Project 10 Subtotal	*1,006*						*$460*	
Misc			2010	Fully Open Gate at Manhole 9910-21 Diversion								
	1708-22	6804-15A		36" Reverse Sewer	2,150	FT	$504.00	$1,084	$163	$323	$1,570	
Grand Total								**$20,819**	**$3,123**	**$6,228**	**$30,170**	

*Project Numbers referenced on **Figure 6-27**