## Section 4

## CRITERIA

### 4-1 General

Establishing performance standards is an important part of evaluating existing wastewater collection systems, as it forms the basis for system analysis and system improvement recommendations. These standards include methodology for estimating wastewater design flows and minimum design standards for the collection system pipes, pump stations, and force mains.

Average wastewater flows can be reasonably estimated from land use and their corresponding unit flow factors. The results are then compared to measured flows. Peaking factors are needed for estimating peak dry weather and peak wet weather flows. Peak wet weather flows include an allowance for inflow and infiltration (I/I).

Collection system design standards include minimum pipe size, minimum flow velocity, and depth of flow to pipe diameter ratio (d/D). Pump station criteria includes the capacity and number of pumps, wet well and force main sizes, redundancy, emergency power, remote monitoring capabilities, as well as safety and regulatory agency requirements. Finally, facility useful lives are needed for adequately scheduling replacement of the aging infrastructure.

### 4-2 Unit Flow Factors

***Flow Monitoring Data***

Data collection and review is essential in developing unit flow factors, calibrating the system model, and estimating the ultimate average day and peak flows.

In order to estimate the residential, commercial, and industrial wastewater flows in the City's existing sewer system (Old Model Colony), a temporary flow monitoring study was conducted by ADS Environmental Services from November 4, 2006 to December 12, 2006 at fifteen locations. The selected flow monitoring locations and a summary of the results are shown on Figure 4-1 and in Table 4-1. Due to limited availability of City field staff during installation and operation of the flow metering equipment, the flow measurements were not taken simultaneously. Data was obtained in five different 14-day time periods. The measured flows are graphically depicted on Figure 4-2. The flow monitoring raw data can be found in Appendix A.

The flow monitoring sites were strategically selected to aid in the development of unit flow factors, calibration of the model, and the determination of flows at locations where two pipes exit the manhole (flow splits). Sites were selected in an attempt to get a good sampling of data across the study area. At the same time, the areas tributary to each site must generate depths of flow large enough to develop accurate flow rates.

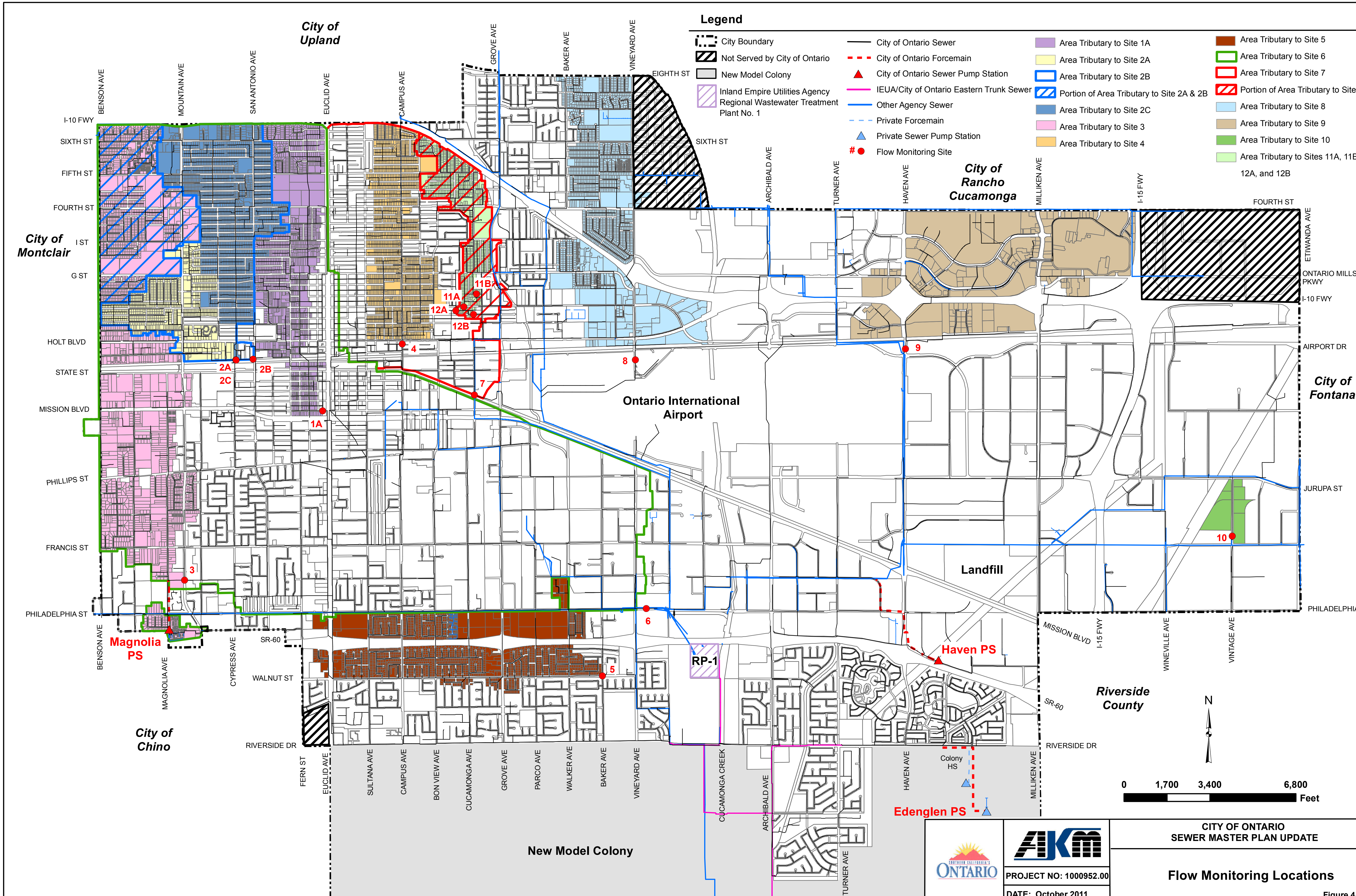
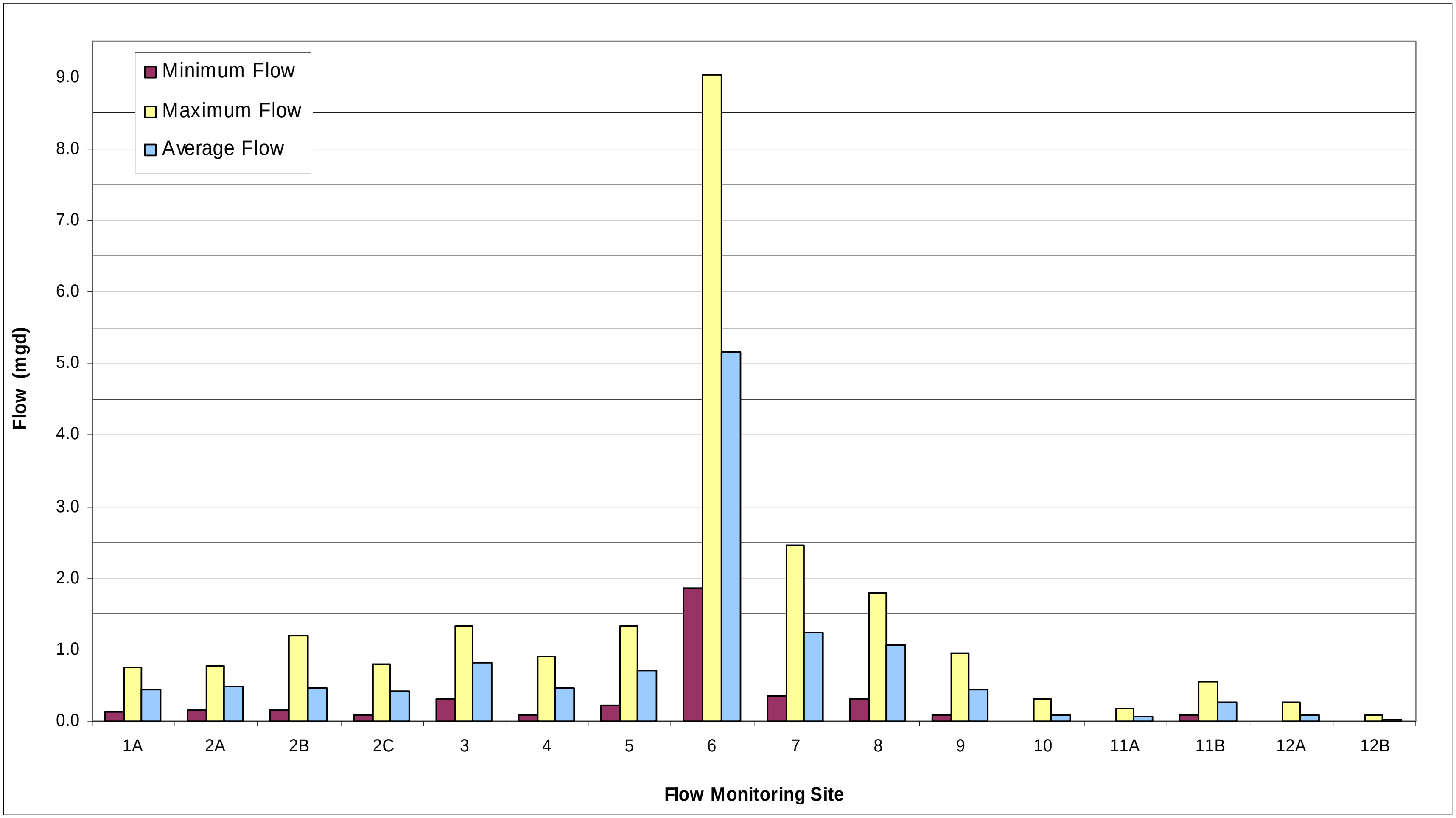

Legend
City Boundary
Not Served by City of Ontario
New Model Colony
Inland Empire Utilities Agency Regional Wastewater Treatment Plant No. 1
City of Ontario Sewer
City of Ontario Forcemain
City of Ontario Sewer Pump Station
IEUA/City of Ontario Eastern Trunk Sewer
Other Agency Sewer
Private Forcemain
Private Sewer Pump Station
# Flow Monitoring Site
Area Tributary to Site 1A
Area Tributary to Site 2A
Area Tributary to Site 2B
Portion of Area Tributary to Site 2A & 2B
Area Tributary to Site 2C
Area Tributary to Site 3
Area Tributary to Site 4
Area Tributary to Site 5
Area Tributary to Site 6
Area Tributary to Site 7
Portion of Area Tributary to Site 7
Area Tributary to Site 8
Area Tributary to Site 9
Area Tributary to Site 10
Area Tributary to Sites 11A, 11B, 12A, and 12B
City of Upland
City of Montclair
City of Rancho Cucamonga
City of Fontana
City of Chino
Riverside County
Ontario International Airport
Landfill
New Model Colony
Magnolia PS
Haven PS
Edenglen PS
Colony HS
RP-1
1A
2A
2B
2C
3
4
5
6
7
8
9
10
11A
11B
12A
12B
I-10 FWY
SIXTH ST
FIFTH ST
FOURTH ST
I ST
G ST
HOLT BLVD
STATE ST
MISSION BLVD
PHILLIPS ST
FRANCIS ST
PHILADELPHIA ST
SR-60
WALNUT ST
RIVERSIDE DR
EIGHTH ST
AIRPORT DR
JURUPA ST
ONTARIO MILLS PKWY
BENSON AVE
MOUNTAIN AVE
SAN ANTONIO AVE
EUCLID AVE
CAMPUS AVE
GROVE AVE
BAKER AVE
VINEYARD AVE
ARCHIBALD AVE
TURNER AVE
HAVEN AVE
MILLIKEN AVE
I-15 FWY
ETIWANDA AVE
WINEVILLE AVE
VINTAGE AVE
MAGNOLIA AVE
CYPRESS AVE
FERN ST
SULTANA AVE
BON VIEW AVE
CUCAMONGA AVE
PARCO AVE
WALKER AVE
CUCAMONGA CREEK
N
0
1,700
3,400
6,800
Feet
ONTARIO
CITY OF ONTARIO
SEWER MASTER PLAN UPDATE
PROJECT NO: 1000952.00
DATE: October 2011
Flow Monitoring Locations
Figure 4-1

**Table 4-1**
**Flow Monitoring Results**

| Site ID | Pipe ID | Manhole ID | Location | Pipe Size (in) | Reason | Depth (in) | | | Velocity (ft/s) | | | Flow (mgd) | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | Min | Max | Avg | Min | Max | Avg | Min | Max | Avg |
| 1A | L131087 | L13120 | East California St & South Euclid Ave (east of Laurel Ave) | 10 | Flow Split | 2.72 | 6.85 | 4.52 | 1.68 | 3.37 | 2.79 | 0.136 | 0.755 | 0.452 |
| 2A | K121041 | K12125 | West of Cypress Ave located in parking lot | 12 | Flow Split | 2.64 | 9.60 | 6.13 | 1.83 | 3.90 | 2.62 | 0.163 | 0.781 | 0.492 |
| 2B | K121051 | K12121 | Main St, West of San Antonio Ave | 8 | Flow Split | 2.57 | 9.97 | 6.13 | 1.70 | 5.50 | 2.62 | 0.148 | 1.199 | 0.464 |
| 2C | K121030 | K12127 | West of Cypress Ave near train tracks | 12 | Flow Split | 1.28 | 3.32 | 2.37 | 2.45 | 7.20 | 5.39 | 0.078 | 0.798 | 0.416 |
| 3 | O111023 | O11121 | Mountain Ave, north of Spruce Ct | 24 | Pump Station Flows / Calibration | 10.87 | 16.49 | 13.36 | 0.34 | 0.98 | 0.69 | 0.308 | 1.328 | 0.821 |
| 4 | J141084 | J14175 | Campus Ave, south of Holt Blvd | 12 | Unit Flow Factors | 1.54 | 3.88 | 2.69 | 2.29 | 6.28 | 5.01 | 0.093 | 0.902 | 0.466 |
| 5 | Q171003 | Q17147 | Intersection of Walnut St & Baker Ave | 18 | Unit Flow Factors | 3.72 | 9.20 | 6.42 | 1.13 | 2.40 | 1.85 | 0.212 | 1.333 | 0.715 |
| 6 | P171043 | P17116 | Philadelphia St, East of Vineyard Ave | 42 | Calibration | 7.71 | 20.11 | 14.14 | 2.21 | 3.17 | 2.76 | 1.867 | 9.038 | 5.159 |
| 7 | K151010 | K15116 | Cucamonga Blvd & North of Ontario Blvd | 18 | Calibration | 2.81 | 7.59 | 5.02 | 2.99 | 5.52 | 4.54 | 0.358 | 2.464 | 1.250 |
| 8 | K171003 | K17102 | North Vineyard Ave, 50 yards north of Terminal Wy | 15 | Unit Flow Factors / Calibration | 2.95 | 6.72 | 4.72 | 2.45 | 5.86 | 4.84 | 0.320 | 1.794 | 1.069 |
| 9 | J211003 | K21100 | Haven St, North of Airport Dr | 27 | Unit Flow Factors / Calibration | 1.90 | 4.85 | 3.42 | 1.21 | 3.03 | 2.17 | 0.098 | 0.952 | 0.440 |
| 10 | N251013 | N25109 | Vintage Ave, North of Francis St | 24 | Unit Flow Factors / Calibration | 0.43 | 2.87 | 1.75 | 0.18 | 2.45 | 1.31 | 0.002 | 0.307 | 0.088 |
| 11A | J151017 | J15116 | D St, West of Cucamonga Ave | 8 | Flow Split | 0.78 | 3.76 | 2.22 | 0.35 | 1.73 | 1.15 | 0.004 | 0.176 | 0.067 |
| 11B | I151073 | I15180 | E St, West of Virginia Ave | 8 | Flow Split | 1.23 | 2.88 | 1.96 | 3.70 | 7.54 | 5.73 | 0.085 | 0.548 | 0.255 |
| 12A | J151042 | J15122 | South west of the Intersection of Holmes Ave and D St | 8 | Flow Split | 0.05 | 5.62 | 2.90 | 0.00 | 3.02 | 1.18 | 0.000 | 0.269 | 0.090 |
| 12B | J151047 | J15127 | Elma St, West of Virginia Ave | 8 | Flow Split | 0.29 | 1.99 | 1.03 | 0.17 | 2.31 | 1.00 | 0.001 | 0.090 | 0.020 |

**Figure 4-2**
**Measured Flow Data**



Site 10 was selected with the intent of developing a unit flow factor for industrial land uses. The flow monitoring indicated in extremely low flows. Through the assistance of City staff, an undocumented connection to an IEUA trunk sewer was found in Vintage Avenue, just south of Jurupa Street.

Based on the flow monitoring data conducted in Old Model Colony and the existing land uses obtained from the City's GIS, calibrated unit flow factors as shown in Table 4-2 were developed. Water use records, aerial photographs and field reviews supplemented this information. The flow factors were developed in units of gallons per day per acre. The multiple family flow factor was found to vary throughout the City from 2,800 gpd/Ac to 6,800 gpd/Ac.

**Table 4-2**
**Calibrated Unit Flow Factors**

| Land Use | | Density (du/Ac) | Average Dry Weather Unit Flow Factor | Units |
|---|---|---|---|---|
| Rural Residential | RR | 0 - 2 | 500 | gpd/Ac |
| Single Family Residential | SFR | 2 - 5 | 1,200 | gpd/Ac |
| Multi-Family Residential | MFR | 11 - 25 | *2,800 | gpd/Ac |
| Commercial | COM | - | 1,000 | gpd/Ac |
| Industrial | IND | - | 400 | gpd/Ac |
| Open Spaces | OPEN | - | 200 | gpd/Ac |
| Public Facilities | PUBLIC | - | 1,000 | gpd/Ac |
| Schools | SCH | - | 25 | gpd/student |
| *Note that unit flow factors based on flow monitoring of sewage generated by existing users* | | | | |
| ** Minimum - unit flow factor for MFR found to vary throughout City* | | | | |

***Edenglen Lift Station Capacity Study***

Edenglen is the City's newest residential community located south of Riverside Drive and east of Hamner Avenue. The community is the first development of NMC. There is an existing sewage lift station serving the currently occupied 201 dwelling units.

The City completed a study of the Edenglen Lift Station capacity on May 18, 2010. Flow monitoring data as well as water consumption from monthly billing data was evaluated. Water consumption records showed that some units had relatively low water use compared to the average, suggesting "under-occupied" units and low contributions to the sewer system. Ultimately, a factor of safety was recommended to account for the limited number of dwelling units, accuracy of measuring low flows, and uncertainties related to physical occupancies and lifestyle habits of existing and future residents. The recommended sewer flow factor was therefore 240 gpd/du.

***Sewer Unit Flow Factors***

The sewer unit flow factors shown in Table 4-3 were used for this study.

The residential unit flow factors in gpd/du are primarily based upon the City's Edenglen Lift Station Capacity Study and the calibrated unit flow factors developed for OMC, which were based on flow monitoring data and water use records. The projected population densities for each type of residential land use were also taken into consideration.

**Table 4-3**
**Ultimate Unit Flow Factors**

| Landuse | | Density (du/ac) | Density (people/du) | FAR | Aveage Dry Weather Unit Flow Factor[1] | | | |
|---|---|---|---|---|---|---|---|---|
| **Residential** | | | | | | | | |
| Rural Residential | RR | 0 - 2 | 4.0 | | 250 | gpd/du | 500 | gpd/ac |
| Low Density Residential | LDR | 2 - 5 | 4.0 | | 240 | gpd/du | 1,200 | gpd/ac |
| Low Medium Density Residential | LMDR | 5 - 11 | 4.0 | | 240 | gpd/du | 2,000 | gpd/ac |
| Medium Density Residential (OMC) | MDR | 11 - 25 | 3.8 | | 210 | gpd/du | 4,200 | gpd/ac |
| Medium Density Residential (NMC) | MDR | 11 - 25 | 3.3 | | 182 | gpd/du | 4,200 | gpd/ac |
| High Density Residential (OMC) | HDR | 25 - 45 | 3.3 | | 180 | gpd/du | 6,300 | gpd/ac |
| High Density Residential (MU Areas) | HDR | 25 - 45 | 2.0 | | 110 | gpd/du | 5,000 | gpd/ac |
| **Commercial** | | | | | | | | |
| Business Park | BP | | | 0.40 | 70 | gpd/tsf | 1,200 | gpd/ac |
| General Commercial | GC | | | 0.30 | 70 | gpd/tsf | 900 | gpd/ac |
| Hospitality[2] | HOS | | | 1.00 | 100 | gpd/tsf | 140 | gpd/room |
| Neighborhood Commercial | NC | | | 0.30 | 100 | gpd/tsf | 1,300 | gpd/ac |
| Office Commercial | OC | | | 0.75 | 90 | gpd/tsf | 3,000 | gpd/ac |
| Restaurant[3] | | | | | 1,000 | gpd/tsf | | |
| **Industrial** | | | | | | | | |
| Industrial | IND | | | 0.55 | 70 | gpd/tsf | 1,600 | gpd/ac |
| **Mixed Use** | | | | | | | | |
| Mixed Use | MU | | | | Use various unit flow factors for | | | |
| **Open Space** | | | | | | | | |
| Open Space Non-Recreational | OS-NR | | | | | | 200 | gpd/ac |
| Open Space Recreational | OS-R | | | | | | 200 | gpd/ac |
| **Public** | | | | | | | | |
| Public Facility | PF | | | | | | 1,500 | gpd/ac |
| Public School - Elementary[4] | PS | | | | 15 | gpd/stu | | |
| Public School - Junior High or High School[4] | PS | | | | 20 | gpd/stu | | |

*[1] Unit Flow Factor Abbreviations:*
*ac = acre*
*du = dwelling unit*
*gpd = gallons per day*
*room = hotel/motel room*
*stu = student*
*tsf = thousand square feet*

*[2] For future hospitality areas, sewage loads can be estimated based on the number of projected rooms. It is not recommended to estimate the load based on acreage.*

*[3] For future restaurants, sewage loads can be estimated based on the building square footage.*

*[4] For future schools, sewage loads should be estimated based on the number of students. It is not recommended to estimate the load based on acreage.*

Retail/service and employment water use was estimated by using a factor of 43 gpd/person (see Technical Memorandum “Ultimate Citywide Water Demand Estimate”, dated June 2011). It is estimated that the sewage generation for retail/service and employment will be about 90 percent of the water use. This results in a factor of 39 gpd/person. The commercial sewer unit flow factors in gpd/tsf are primarily based on this factor of 39 gpd/person and the employment population per tsf. A minimum of 70 gpd/tsf is recommended for commercial uses.

The City’s 2010 General Plan defines an area along Vineyard Avenue, south of the I-10 Freeway and north of the Ontario International Airport, as the hospitality area. It consists of numerous hotels and restaurants that provide service for patrons of the airport. In developing the sewer unit flow factor for this area, the water use information was examined. In 2008, the total average water use for this area was about 544,000 gpd. Per the City’s 2010 General Plan, the estimated floor area of these buildings is 6,312 tsf. The equivalent water unit flow factor is therefore about 86 gpd/tsf (544,000/6,312). There is some uncertainty about the level of occupancy for the period of water use data used and there are a couple of undeveloped lots. The general plan square footage is the ultimate estimate. Therefore, it is recommended to use a factor of safety when estimating the water use and sewer loads. For planning purposes, the hospitality unit flow factor recommended is 100 gpd/tsf. For future hospitality developments, a unit flow factor of 140 gpd/room can also be utilized. This factor was developed from examination of water records and the number of associated rooms of hotels located in the Southern California area.

For future restaurants, if the building square footage is known, it is recommended to estimate the sewage load based on a unit flow factor of 1000 gpd/tsf. This factor was developed from examination of water records and the building square footage for restaurants located in the Southern California area.

The industrial unit flow factor is estimated at 70 gpd/tsf. Depending on the type of industrial processes used at certain facilities, this factor may be low. The water use records were utilized to identify any high water users that may potentially produce more sewage. The sewage load representing high water users were increased in the hydraulic model on a case by case basis.

The open space unit flow factor recommended is 200 gpd/ac. The public facility unit flow factor recommended is 1,500 gpd/ac. The public elementary school unit flow factor recommended is 15 gpd/student. The public junior high school and high school unit flow factor recommended is 20 gpd/student. These are typical factors used for planning purposes, based upon review of water use records and accounting for irrigation. For this study, the available water use records for each school in Ontario was looked at along with the acreage of the school parcel and the latest student enrollment numbers. The recommended sewage unit flow factors for schools is based on a percentage (about 40-45%) of the water use. The remainder of the water use is assumed to be utilized for irrigation.

The development of the sewer unit flow factors is documented in more detail in the Technical Memorandum, Sewer Load Estimates (see Appendix C).

## 4-3 Peaking Factors

***Peak Dry Weather***

The wastewater unit flow factors discussed in Sub-section 4-2 are used to generate average dry weather flows (ADWF) entering the collection system. However, the adequacy of a sewage collection system is based upon its ability to convey the peak flows. At any individual point in the system, peak dry weather flow (PDWF) is estimated by converting the total average flow upstream of the point in question to peak dry weather flow by an empirical peak-to-average relationship.

The peaking formula commonly used in sewerage studies is of the following form:

$$PDWF = a \times ADWF^{b}$$

where PDWF = Peak Dry Weather Flow
ADWF = Average Dry Weather Flow
a, b = Peaking Formula Coefficients

The temporary flow monitoring data was reviewed to develop peaking relationships at each site. As expected, these relationships varied from site to site depending upon the makeup and size of the tributary land use. Coefficient “b” is typically found to be in the range of 0.91 to 0.92 based on empirical studies. Using a coefficient “b” of 0.92, the resulting coefficient “a” can be calculated from the measured flow data. The calculated coefficient “a” for each flow monitoring site is shown graphically on Figure 4-3. The coefficient “a” selected for this study is based on the information shown on Figure 4-3. It was determined that a coefficient “a” of 2.0 would cover most situations in the system without being overly conservative. If the coefficient selected is too conservative, hydraulic deficiencies would be unecessarily identified.

Based on the information shown in Figure 4-3, the following peaking relationship was selected for this study:

$$PDWF\ (mgd) = 2.0 \times ADWF\ (mgd)^{0.92}$$

Please note that the units of the peaking formula above are in million gallons per day (mgd).

***Peak Wet Weather***

The peak wet weather flow (PWWF) has two components: peak dry weather flow (PDWF) and rainfall dependent inflow/infiltration (I/I) as expressed by the following equation:

$$PWWF = PDWF + I/I$$

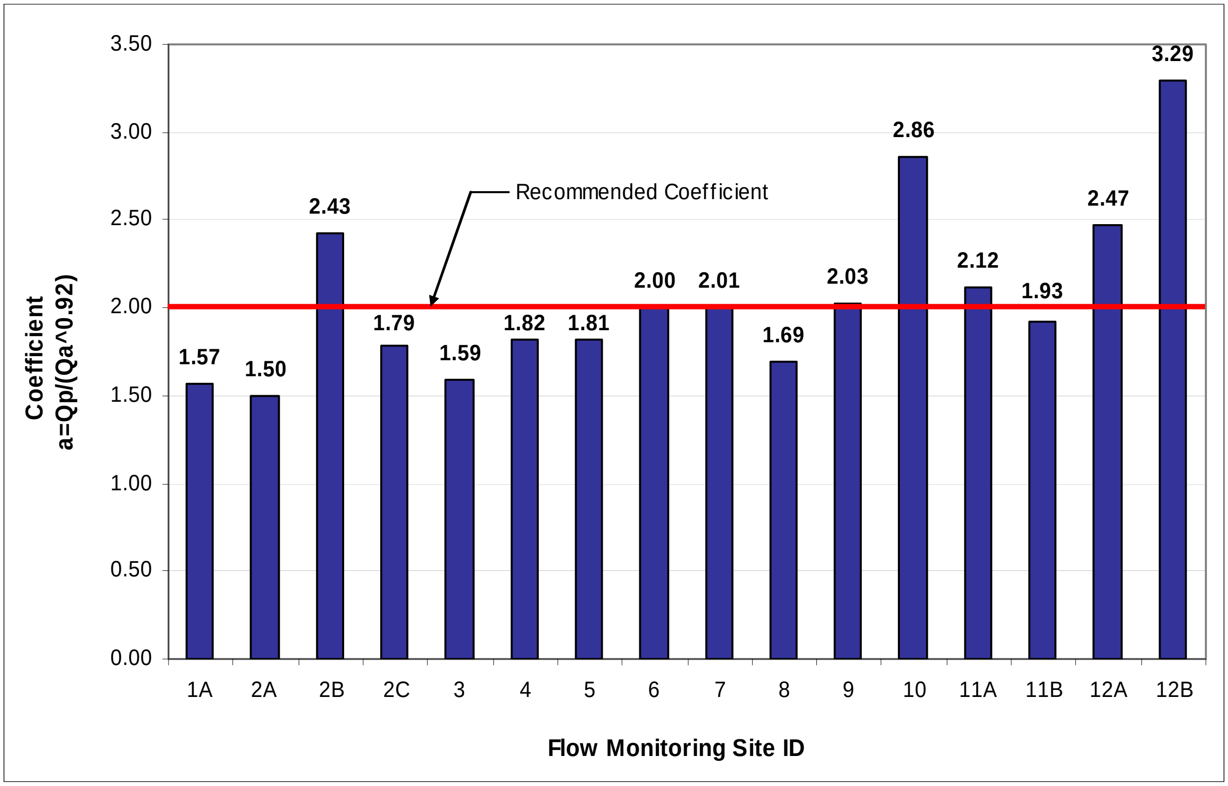
Inflow and infiltration is discussed further in Sub-section 4-4.

The flow monitoring effort for this study did not cover a wet weather period. Until wet weather flow data can be collected, it is recommended that the peak wet weather flow be estimated as the following:

Peak Wet Weather Flow (PWWF) = 1.34 x Peak Dry Weather Flow (PDWF)

Although the PWWF/PDWF factor of 1.34 may not cover all situations, it is not reasonable or feasible to design the sewer system to carry the flows that would result from the use of a larger ratio. Instead, it is recommended that the City concentrate on projects such as replacing manhole covers, installing plugs in manhole covers, and replacing or relining cracked pipes to reduce inflow and infiltration.

**Figure 4-3**
**Peaking Formula Coefficient “a”**




## 4-4 Inflow and Infiltration

Inflow is the surface water that typically gains entry to the sewer system through perforated or unsealed manhole covers during rainfall events. Infiltration is defined as water entering the collection system from the ground through defective pipes, pipe joint connections, or manhole walls. The sewer system design capacity must include allowances for these extraneous flow components, which inevitably become a part of the total flow. The amount of inflow and infiltration (I/I) that enters the system typically depends upon the availability, adequacy, and location of the storm water drainage facilities; age and condition of structures; materials and methods of construction; the location of the groundwater table; and the characteristics of the soil. In the absence of flow monitoring data, many regulating agencies utilize commonly accepted practices for estimating I/I. For example, I/I is often estimated based on the diameter and length of pipeline (100 to 400 gpd/ in. dia/ mile) or as a percentage of the peak flow or pipeline capacity.

AKM’s experience from other master planning studies and review of limited flow monitoring information available during severe rainfall events indicate that the peak wet weather flow can vary from 10 percent of average dry weather flows in steeper areas with adequate drainage facilities, to over 400 percent of average dry weather flows in flat areas that lack significant drainage facilities.

For this study, extraneous flow due to inflow and infiltration is included in the peak wet weather flow formula described above. If better data becomes available subsequently for specific areas, the analysis shall be updated based upon that information.

### 4-5 Sewer Design Criteria

Design criteria are established to ensure that the wastewater collection system can operate effectively under all flow conditions. Each pipe segment must be capable of carrying peak wet weather flows in the hydraulically stable zone of the pipe. Low flows must be conveyed at a velocity that will prevent solids from settling and blocking the system.

The design capacity of a gravity pipeline is the calculated capacity of the pipeline based on the Manning formula:

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

where, **Q** = flow in cubic feet per second
**R** = hydraulic radius in feet = A / P
**A** = cross-sectional area of the pipe in square feet
**P** = wetted perimeter in feet
**S** = slope of pipe in feet of rise per foot of length
**n** = Manning's friction factor

Sewer system capacity is established using a Manning's friction factor of 0.013 for vitrified clay pipe.

The design and analysis of sewer pipes is typically based upon the depth to diameter ratio (d/D). In this study, **existing** pipes are considered capacity deficient if the d/D is above 0.64 at peak dry weather flows. This d/D ratio was arrived at by taking 75 percent of a pipe's maximum stable flow capacity, which is at a d/D of 0.82. The area above a d/D of 0.82 is considered hydraulically unstable. This provides capacity for 25 percent of peak dry weather flow for inflow and infiltration. Calculated capacity deficiencies shall be verified through flow monitoring prior to replacing facilities.

The extra pipeline capacity allows for the possibility that actual wastewater flows may be slightly higher than anticipated, especially during the hours when instantaneous or intermittent peaks may occur. These peaks are generally observed between the hours of 6:00 a.m. and 9:00 a.m. and 7:00 p.m. and 9:00 p.m. during weekdays and somewhat later in the morning hours during weekends in the predominantly residential areas. They may also be observed during rainfall events due to inflow and infiltration. Additionally, the area above the water surface helps to keep the sewage aerated, reducing the possibility of septic conditions and odors.

For **new construction**, the design and analysis of gravity sewer pipes shall be based on the following depth to diameter ratios:

- Pipes **12-inches and smaller** in diameter shall be designed to flow at a maximum **d/D of 0.50** under peak dry weather flows

- Pipes **15-inches and greater** in diameter shall be designed to flow at a maximum **d/D of 0.64** under peak dry weather flows
- For either group, the depth of flow to diameter ratio shall not exceed **0.82 with peak wet weather flows**

At a minimum, all pipes shall be 8 inches or larger in diameter and the velocity of flow in the pipe shall be greater than 2 feet per second at average dry weather flow (ADWF). This velocity will prevent deposition of solids in the sewer and help to resuspend any materials that may have already settled in the pipe. The minimum corresponding slopes for various pipe sizes are shown in Table 4-4.

It is important to note that the slopes listed in Table 4-4 assume the depth of flow in the pipe is 50 or 64 percent full depending on the size. If there is insufficient flow to create this condition, greater slopes than those shown may be required.

The peak flow velocity shall be less than 10 feet per second in vitrified clay pipe.

The City recognizes that minimum slopes and velocities are sometimes not achievable under certain circumstances. On a case by case basis, the City may approve sewer designs that do not meet these criteria.

**Table 4-4**
**Minimum Sewer Slopes**

| Sewer Size | 2 ft/s Velocity Slope |
|---|---|
| 8" | 0.0057 |
| 10" | 0.0042 |
| 12" | 0.0033 |
| 15" | 0.0019 |
| 18" | 0.0014 |
| 21" | 0.0011 |
| 24" | 0.0008 |
| 27" | 0.0008 |
| 30" | 0.0007 |
| 33" | 0.0006 |
| 36" & larger | 0.0005 |

## 4-6 Pump Station Design Criteria

It is desirable to develop a sewer collection system with as few pump stations as possible due to the associated cost and maintenance required. The City's policy does not allow new pump stations. If a pump station is absolutely necessary, the following criteria shall be minimum standards.

The pump station must be designed to be reliable, and sized with sufficient capacity. They must contain redundant equipment, an emergency power supply, bypass pumping capability, sufficient wet well storage, and be able to notify the appropriate personnel in the event of failure.

The primary components of a typical pump station are the wet well, motors, valves, dry well, pumps, ventilation, electrical, controls and the force main. The following general criteria are recommended.

**The wet well** stores the incoming wastewater until a pump is activated to discharge it to a gravity facility for further conveyance. It shall be designed with sufficient capacity to prevent short cycles whereby the pumps frequently start and stop, yet small enough that it will regularly evacuate sewage from the wet well to prevent the wastewater from becoming septic. Generally, the desired number of pump cycles shall be limited to no more that 6 per hour for motors up to 10 horsepower. Motors up to 75 horsepower shall start no more than 4 times per hour. Larger motors shall cycle less frequently. Pump stations shall also have sufficient volume to store sewage in the event of mechanical or electrical failures, until the City can respond to the failure and prevent overflows. The necessary emergency storage is dependent upon how rapidly the City can respond to a failure and mitigate it. A minimum emergency storage of 30 minutes at peak wet weather flow shall be provided.

**The pumps** shall be sized to efficiently handle the peak wet weather flows. A minimum of two pumps sized at the peak wet weather flow to the station shall be provided so that sufficient standby capacity is available when one pump is removed for repairs or experiences a mechanical failure. The pumps shall be able to pass a minimum solid size of 3 inches without clogging. The shafts, seals and impellers shall be constructed of wear resistant material to provide long life. Tungsten Carbide seals, Ni-Hard impellers, and 316 stainless steel pump shafts are recommended. For services where aggressive agents may be found in the sewage, such as at golf courses, complete stainless steel construction is recommended. This includes the pump bowl, shaft, impeller, and motor housing.

**The dry well** houses the valves, pumps, motors and electrical equipment and controls. It must be well ventilated and provide unobstructed access to all equipment. A minimum 3-foot clearance from all obstructions shall be provided. Greater clearances may be required for equipment with special maintenance needs. Provisions for equipment removal including hatches, large door openings, and hoists shall also be provided.

**The force mains** shall be selected to operate within a 3 feet per second to 5 feet per second velocity range, but shall not be smaller than 4-inches in diameter.

While submersible pump stations may be utilized for the small flows, the larger pump stations shall be the wet well/dry well type. They shall be designed with easy access to all equipment. The National Electric Code classifies the wet wells of wastewater pumping stations as Class I, Group D, Division 1 facilities if ventilated at less than 12 air changes per hour, and Division 2 if continuously ventilated at 12 or more air changes per hour. Dry wells, which are physically separated from wet wells, if ventilated at less than 12 air changes per hour, are classified as Class I, Group D, Division 2 locations. Wet wells, and under certain circumstances dry wells, are considered confined spaces and shall be entered in accordance with the corresponding requirements of Occupational Safety and Health Administration (OSHA).

All pump stations shall incorporate redundant control systems for operation of the pumps. A float system shall be used as a backup for a primary control system that utilizes an ultrasonic device or a bubbler system for level measurement and pump operation.

Full SCADA telemetry equipment which includes a telephone dialer as a backup, must be provided at all sewer pump stations. When an alarm or failed condition occurs, the dialer calls pre-programmed telephone numbers in sequence until the call is acknowledged, indicating response will be provided by City staff. If the alarm or failed condition is not corrected within a set time, the dialer will call the pre-programmed numbers again. The dialer can also be used to remotely check the status of the station if desired.

A summary of sewer system design criteria is listed in Table 4-5.

**Table 4-5**
**Sewer System Criteria**

| | |
|---|---|
| ***Collection System*** | |
| Minimum Pipe Size | 8-inch |
| Minimum Velocity | 2.0 ft/sec at average flow<br>3.0 ft/sec at peak dry weather flow |
| Pipe Depth to Diameter Ratio for *Existing Pipes* | 0.64 for all pipe sizes at peak dry weather flow<br>0.82 for all pipe sizes at peak wet weather flow |
| Pipe Depth to Diameter Ratio for *New Construction* | 0.50 for pipes 12-inches and smaller at peak dry weather flow<br>0.64 for pipes 15-inches and larger at peak dry weather flow<br>0.82 for all pipe sizes at peak wet weather flow |
| ***Pump Stations*** | |
| Pumps | ▪ Minimum 2 each sized at peak wet weather flow<br>▪ Minimum solids handling capacity 3" |
| Wet Wells | ▪ Sized to limit pump cycling to less than 4 to 6 times/hr<br>▪ Provide sufficient storage at peak wet weather flow to allow response to a failure<br>▪ Equipment to be maintained must be accessible without entering structure |
| Ventilation | ▪ 12 -air changes/hour minimum in dry well and as required by NFPA 820<br>▪ 30-air changes/hour minimum in wet well if not operated continuously<br>▪ 12-air changes/hour minimum in wet well if operated continuously |
| Controls | Redundant system. Float operated back-up controls. |
| Emergency Power | Stationary source with automatic transfer switch |
| Telemetry | Full SCADA with dialer system as back up at all pump stations to alert personnel in the event of a station failure. |
| Force Mains | ▪ Minimum velocity 3.0 ft/sec<br>▪ Maximum velocity 5.0 ft/sec<br>▪ Minimum size 4"<br>▪ Air/Vacs installed in vaults<br>▪ Plumb Air/Vacs piping back to wet well to avoid discharges of raw sewage to vaults |

## 4-7 Service Life of Pipe and Lift Station Equipment

In addition to the design criteria discussed in previous sections, the useful lives for which one can expect relatively trouble-free service is also of great importance when assessing an existing or future sewer system. Once the service life of a facility is exceeded, it becomes subject to failure and is often expensive to maintain. The determination of useful life can be difficult and depends on many different considerations including the following:

- Type of materials used and recorded performance of similar installations
- Velocities and flow rates expected in the system
- Chemical and biological conditions of the wastewater
- Construction methods and installation

The values listed in Table 4-6 are generally accepted as prudent planning criteria and are used as benchmarks for replacement recommendations in this study.

**Table 4-6**
**Planning Criteria for Facility Useful Life**

| Facility | Description | Useful Life (Years) |
|---|---|---|
| **Gravity Sewers:** | Cast Iron Pipe (CIP) | 20 |
| | Plastic Pipe | 65 |
| | Vitrified Clay Pipe (VCP) | 75 |
| **Force Mains:** | Asbestos-Cement Pipe (ACP) | 40 |
| | Ductile Iron Pipe (DIP) | 40 |
| | Plastic Pipe | 30 |
| **Pump Stations:** | Structure | 60 |
| | Piping | 30 |
| | Valving | 20 |
| | Mechanical | 15 |
| | Electrical | 15 |

## 4-8 Criteria for Specific Plans and Development Subareas

Each party wishing to pursue development of a tract or area within the City service area shall develop a Sub-Area Master Plan (SAMP). The developer's plans for providing adequate sewer service to all users within the proposed development, how the local sewer system will connect to the backbone and regional system, and the impact of the proposed development to the downstream facilities (to the regional system) shall be fully described in the SAMP. The local sub-area sewers shall meet the sewer design criteria provided in this document and the City Standard Drawings for

Sewer Construction. At a minimum, sewage flow calculations shall be based upon the unit flow factors contained in Table 4-3 or higher factors if specific conditions require it.

Where flow from a new development or redevelopment is proposed to be added to an existing City sewer, the existing sewer shall be flow monitored by a qualified company acceptable to the City at the owner's cost for a minimum period of two weeks to verify the existing minimum, average, and peak dry weather flows. The location(s) of flow monitoring shall be determined by the City. Two copies of the flow monitoring report shall be submitted to the City in the City's required format. The City will determine the adequacy of capacity in all the City facilities that will convey the subject flow to the regional facilities. Service to proposed development or redevelopment shall be subject to availability of capacity in the City sewers and regional sewers.

A typical Sub-Area Sewer Master Plan Report shall include, but not be limited to the following:

- Map showing project boundaries and drainage areas
- Detailed land use description and map
- Average dry weather, peak dry weather, and peak wet weather flow calculations
- Exhibit showing all proposed sewer facilities and connections to the downstream regional system
- Phasing of development and wastewater flows
- Hydraulic calculations for phased and fully developed ultimate conditions, from the development to the regional system, meeting all sewer design criteria
- Results of flow monitoring, if project area is tributary to existing City sewers