



**City of Franklin, TN  
Water Management Department  
Design Criteria for  
Wastewater Infrastructure**

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**City of Franklin, TN Delegated Authority Design  
Criteria for Sanitary Sewer**

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## City of Franklin, TN Delegated Authority Design Criteria for Sanitary Sewer

### 1.1 Standard of Performance for Review of Collection System Construction Documentation

This chapter of the Design Criteria for the Review of Sewage Works Construction Plans and Documents (“*Criteria*”) provides guidance for meeting the following performance standards for collection system projects. This includes methodology for estimating wastewater design flows and minimum design standards for collection system gravity lines, manholes, pump stations, and force mains.

The construction documents, including final plans, specifications, engineering reports, and calculations should be prepared in accordance with Chapter 1 of the *Criteria*. The documents should incorporate the technical standards in this chapter and be sufficient for construction of a system capable of transporting the peak flows, including infiltration and inflow (I/I), as efficiently as possible while minimizing I/I and providing a system that is manageable and cost-effective for the owner to maintain and operate.

Inflow which does not include infiltration has been defined as water, other than wastewater, that enters a sewer system from sources such as roof leaders, cellar drains, yard drains, area drains, foundation drains, drains from springs and swampy areas, manhole covers, cross connections between storm drains and sanitary sewers, catch basins, cooling towers, stormwaters, surface runoff, street wash waters, or other drainage. (EPA (2004); Water Environment Federation (2011).)

Infiltration has been defined as stormwater and groundwater that enter a sewer system through such means a defective pipe, pipe joints, connections, or manholes. (Infiltration does not include inflow). Appendix 2-C contains guidance to determine the quantity of annual I/I. This



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### 1.2 General Requirements for Collection Systems

#### 1.2.1 Construction Approval

In general, construction of new sewer systems, extensions of existing systems, or upgrades to pumping systems must ensure that the downstream conveyance system and the receiving wastewater treatment plant are either:

- Capable of adequately conveying and/or processing the added hydraulic and organic load, or
- Capable of providing adequate conveyance and/or treatment facilities on a timely schedule acceptable to the City of Franklin Water Management Dept.

#### 1.2.2 Operation and Maintenance

“The supervision, operation, and maintenance of sewerage systems shall be of such character as in the opinion of the City of Franklin Water Management Department will produce satisfactory results as judged by current standards of the Department.

#### 1.2.3 Infiltration and Inflow Considerations

Design and construct new sewer systems to achieve total containment of sanitary wastes and exclusion of I/I. This includes installing pipe with watertight joints, watertight connections to manholes, and watertight connections to service laterals or service lateral stubs and using trench design that minimizes the potential for migration of water along the trench. Additionally, the new sewer system and appurtenances must be able to convey the wastewater load, including existing I/I and rainfall derived I/I (RDI/I) peak flow rates, from upstream areas as appropriate. An analysis of existing I/I should be submitted (and may be required) where I/I is known to be a problem in the existing sewer, and extensions are proposed.

#### 1.2.4 Erosion Prevention and Sediment Control (EPSC)

Construction projects resulting in one or more acres of soil disturbance or less than one acre as part of a larger common plan of development comprising at least one acre of cumulative land disturbance are required to obtain authorization under the National Pollutant Discharge Elimination System (NPDES) General Permit for Discharges of Stormwater Associated with Construction Activities (CGP) or an individual NPDES permit. Construction activities include clearing, grading, filling and excavating. Site operators must maintain coverage under this permit for all portions of a site that have not been permanently stabilized.

CGP Notice of Intent (NOI) and supporting documentation are required at the time of construction document submittal. The NOI is processed at the local field office and a copy of the NOI or the permit tracking number shall be included with the plan’s submittal.



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EPSC measures shall be designed in accordance with the current version of the TDEC Erosion and Sediment Control Handbook and meet all City of Franklin Stormwater Requirements as implemented by its MS4 requirements. With regard to EPSC involving sewer lines:

- Clearing, grubbing, and other disturbance to riparian vegetation and wetland shall be kept at the minimum necessary for slope construction and equipment operations.
- Sediment shall be prevented from entering waters of the state.
- All spoil materials from trench excavation, bore pits, and other earth disturbing activities shall be deposited in an upland location and stabilized within seven days in order to prevent erosion into waters of the state.
- All dewatering activities shall be conducted in such a manner as to prevent the discharge of sediment-laden water into waters of the state.
- To minimize wildlife entanglement and plastic debris pollution, temporary erosion and sediment control products that either do not contain netting, or that contain netting manufactured from 100 percent biodegradable non-plastic materials such as jute, sisal, or coir fiber shall be specified. Netting used in these products should have a loose-weave wildlife-safe design with movable joints between the horizontal and vertical twines, allowing the twines to move independently. Degradable, photodegradable, UV-degradable, oxo-degradable, or oxo-biodegradable plastic netting (including polypropylene, nylon, polyethylene, and polyester) are not acceptable alternatives.

### 1.2.5 Overflows and Releases

The City of Franklin Water Management Dept. will not allow or knowingly approve designs that create overflows or releases in separate sanitary sewers or new overflows in existing combined sewers. In existing combined sewer systems, overflows are only permitted at locations noted in the NPDES individual permit. An alarm system to signal existing overflow conditions and procedures for reporting overflows may be required.

### 1.2.6 Fat, Oil and Grease

Facilities with commercial-grade kitchen facilities should be equipped with an effective grease and oil interceptor. Other potential sites of grease/oil production should be investigated by the design engineer.

One or more interceptors in series are required where grease or oil waste is produced that could hinder sewage disposal or treatment, and/or create line stoppages. Interceptors must be located to provide easy access for inspection, cleaning and maintenance. In commercial-grade kitchen facilities, the dishwasher(s) must not be connected to the primary grease trap and/or separator. A separate device may be required to allow for cooling of the dishwasher discharge prior to primary treatment. Oils will not solidify until approximately 70° F or less. Therefore, the minimum interceptor design should be a baffled, three compartment,



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elongated chamber to allow for cooling with a capacity of at least 1,500 gallons. The design should be in accordance with accepted engineering practice. Tanks must also be sized in accordance with local requirements. The tank should be buried, with manhole accesses to all compartments. Tanks should be

manufactured and furnished with access openings having a minimum diameter of twenty-one (21) inches. The tank top should be able to support a minimum of 2500 lb. wheel load. Inlet plumbing should be designed to penetrate 18 inches or more below the discharge invert elevation. In order to demonstrate water-tightness, tanks (including all risers and lids) must be tested prior to acceptance by filling with either air or water in accordance with ASTM standard C1227-05.

### 1.2.7 Calculations

All computations and data used for design of the sewer system shall be submitted to the City of Franklin Water Management Department for approval. The recommended format to present calculations for small pump stations is provided in Appendix 1-D-5 Small Non-Treatment Project Engineering Calculations.

### 1.2.8 Final Plans Submission

Section 1.3 outlines the content expected in contract drawings and specifications.

## 1.3 General Collection System Design Considerations

### 1.3.1 Design Factors

#### 1.3.1.1 Collection sewers (Laterals and Submains)

Design collection sewers for the ultimate development of the area served.

#### 1.3.1.2 Main, Trunk, and Interceptor

Sewers Design shall consider:

- Possible solids deposition, odor, and pipe corrosion that might occur at initial flows;
- Population and economic growth projections and the accuracy of the projections;
- Comparative costs of staged construction alternatives;
- Effect of sewer sizing on land use and development; and



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- Pipe slope to provide enough velocity of wastewater for self-cleansing.

### 1.3.1.3 General Considerations

Designs shall consider:

- Peak wastewater flows from residential, commercial, institutional, and industrial sources;
- Potential for groundwater infiltration from existing upstream sewers;
- Potential for sewer back-ups into homes;
- Topography and depth of excavation;
- Treatment plant location;
- Soils conditions;
- Pumping requirements;
- Maintenance, including manpower and budget;
- Existing sewers;
- Existing and future surface improvements;
- Controlling service connection elevations;
- Buoyancy in locations where high groundwater is anticipated;
- Proximity to surface streams, including minimizing the potential for draining or diversion of stream water into the pipe trench; and
- Water-tightness to exclude groundwater and surface water.

### 1.3.2 Basis of Design Flow Rates

Designs shall consider base design requirements for new sewer systems on per capita flows or by alternative methods. Flow data from similar portions of the same system or very similar systems is generally more credible than generic data from tables.

#### 1.3.2.1 Per Capita Flow

For systems using watertight collection, the recommended design flow is 100 gallons per capita per day or 350 gallons per residential unit per day. For developments that include larger homes, higher flows should be considered. For projects dealing with commercial or very large residential developments, design flows should reflect expected variations from conventional systems and be evaluated and approved by the dept. based upon site-specific evaluation.

Appendix 1-A includes the design basis for wastewater flow and loadings for commercial and



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industrial sources. Substitutions or additions to the information presented in this table are acceptable if better or more accurate data is available. Peak flow:

- Lateral and Submains: Minimum peak design flow should not be less than 400 percent of the average design flow.
  - “Lateral” - a sewer that has no other common sewers discharging into it.
  - “Submain” - a sewer that receives flow from one or more lateral sewers.
- Main, Trunk, and Interceptor sewers: Minimum peak design flow should be not less than 250 percent of the average design flow.
  - “Main” or “trunk” - a sewer that receives flow from one or more submains.
  - “Interceptor” - a sewer that receives flow from a number of main or trunk sewers, force mains, etc.

### 1.3.2.2 Alternative Methods

Alternative methods, other than on the basis of per capita flow rates, may include the use of peaking factors of the contributing area, allowances for future commercial and industrial areas, separation of I/I from the normal sanitary flow (for new sewers serving existing upstream sewers), and modification of per capita flow rates (based on specific data). There should be no specific allowance for I/I above and beyond the existing amount into newly constructed or proposed sewers.

## 1.4 Gravity Sewer Design

### 1.4.1 Capacity

Design gravity sewers to be approximately one-half full when conveying the anticipated peak daily dry weather flow and to not surcharge when conveying the anticipated peak wet weather flow.

### 1.4.2 Minimum Size

The minimum size of new public sewers should be eight inches (nominal) in diameter.

### 1.4.3 Depth

Generally, sewers should not be less than 3.0 feet deep from the crown of the pipe but should be sufficiently deep to prevent freezing and physical damage.



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### 1.4.4 Roughness Coefficient

Use a roughness coefficient “n” value of 0.013 in Manning’s formula for the design of all sewer facilities unless a roughness coefficient specific to the given pipe material is available. The roughness coefficient selected must consider the long-term condition of the sewer. However, the “n” value shall be equal to or greater than 0.011.

### 1.4.5 Slope

Sewers must be self-cleansing and capable of transporting most solids to the desired point, usually a treatment facility. One method is approved for design by the City of Franklin and is preferred is The Traditional (Ten-State Standards)

**Table 1-2 Minimum Slope from Traditional Method**

Sewer Size (inches)	Minimum Slope (feet per 100 feet)
8	0.40
10	0.28
12	0.22
15	0.15
18	0.12
21	0.10
24	0.08
27	0.067
30	0.058
36	0.046 **
42	0.037***
48 & greater	As per engineer design. And approved by WMD.

“The pipe diameter and slope shall be selected to obtain the greatest practical velocities so as to minimize settling problems. Flatter slopes shall not be justified with oversize sewers. If the proposed slope is less than the minimum slope of the smallest pipe that can accommodate the design peak hourly flow, the actual depths and velocities at minimum, average, and design maximum day and peak hourly flow for each design section of the sewer shall be calculated by the design engineer and be included with the plans.” (Ten States Standards, 2014).

Under special conditions, using the Traditional Method, slopes slightly less than those required



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for the 2.0 feet-per-second velocity when flowing full may be permitted. Such decreased slopes will only be considered where the depth of flow will be 30% of the diameter or greater for design average flow. Whenever such decreased slopes are proposed, the design engineer should furnish computations of the depths of flow in such pipes at minimum, average, and daily or hourly rates of flow. The maintaining wastewater agency must recognize and accept in writing the problems of additional maintenance that may result from extended periods of minimum flow in decreased slope designs. Uniform slope between manholes is required.

Anchors are required for sewers on 20 percent slope or greater. Secure anchors will have a minimum two-foot thick concrete collar or equal. Suggested minimum anchorage spacing is as follows:

- Not over 36 feet center to center on grades 20 percent and up to 35 percent.
- Not over 24 feet center to center on grades 35 percent and up to 50 percent.
- Not over 16 feet center to center on grades 50 percent and over.

### 1.4.6 Alignment

Normally, straight alignment between manholes is required for gravity sewers. Topographic features may favor curved segments to avoid excessive depth. In such rare cases, the use of fuse-welded flexible segments is preferred to the use of maximum deflection joint values in rigid straight pipes, which tend to promote joint leakage. Curved sewers may be approved where circumstances warrant. A minimum of five feet of horizontal separation between the sewer pipe and gas mains is required, measured edge to edge.

### 1.4.7 Increasing Size

When a smaller sewer joins a larger one, maintain the same energy gradient. An approximate method for securing these results is to match the crowns of the sewers entering/ exiting the manhole or junction structure.

### 1.4.8 High-Velocity Protection

Where velocities greater than 15 feet per second are expected, provide protective measures against internal erosion or displacement by shock, such as drop inlets and off-center inlets and outlets.

### 1.4.9 Inverted Siphons

Inverted siphons are not allowed under normal conditions. However, if necessary due to site characteristics, provide the following:

- Minimum of two barrels;
- Minimum pipe size of six inches--provided with necessary appurtenances for



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- convenient flushing and maintenance;
- Manholes with adequate clearances for rodding;
- Sufficient head and pipe sizes to secure velocities of at least 3.0 feet per second for average flows;
- Inlet and outlet details arranged so that the normal flow is diverted to one barrel, and so that either barrel may be cut out of service for cleaning;
- Proper access maintained; and
- Hydraulic calculations.

### 1.4.10 Manholes

#### 1.4.10.1 Location

Manholes are required at the end of each 8-inch diameter sewer or greater. This requirement may be waived if a stub-out is installed (assuming the line will be extended in the near future).

Manholes are required at all changes in grade, size, or alignment, and at all intersections. Large manholes need to be placed strategically with due regard to topographical landforms or respect to other structures.

Manholes are required at distances not greater than 400 feet for sewers 15 inches or less and at distances not greater than 500 feet for sewer 18 inches to 30 inches. Greater spacing may be allowed in larger sewers and in those carrying a settled effluent. Ensure adequate trench backfill to avoid sagging in long segments. Avoid placing manholes in an open water, permanently inundated wetland.

Access to manholes must be considered in the design phase. Manholes should be located in areas where manned access is optimal and not hindered by permanent or constant obstructions (e.g. parking spaces, under guardrails, structures, etc.)

#### 1.4.10.2 Drop Connection

Provide a drop connection for a sewer entering a manhole at an elevation of 24 inches or more above the manhole invert. Where the difference in elevation between the incoming sewer and the manhole invert is less than 24 inches, a filleted invert will prevent solids deposition.

#### 1.4.10.3 Diameter

The minimum diameter of manholes should be 48 inches. The minimum clear opening in the



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manhole frame shall meet current Occupational Safety and Health Administration (OSHA) standards.

### 1.4.10.4 Flow Channels

Flow channels in manholes should be of such shape and slope to provide smooth transition between inlet and outlet sewers and to minimize turbulence. Channeling height should be to the crowns of the sewers. Benches should be sloped from the manhole wall toward the channel to prevent accumulation of solids.

### 1.4.10.5 Water tightness

Wherever the manhole tops may be flooded, watertight manhole covers are required. In pre-cast concrete manholes, plastic gaskets, pre-molded rubber gaskets or flexible, plastic gaskets are required. Vent manholes every 1,000 feet or every other manhole, whichever is greater. Vent height and construction must consider flood conditions. Protect manholes from the "100-year flood" with manhole rims or watertight manholes vented 24 inches above the 100-year flood elevation.

### 1.4.10.6 Connections

Pay special attention to the connection between the manhole wall and the sewer pipe in order to minimize infiltration into the system. Flexible joints are required for line connections directly to the manholes, or to short stubs integral with the manholes. Flexible joints are joints that permit the manholes to settle without destroying the watertight integrity of the line connections.

### 1.4.10.7 Frames, Covers, and Steps

Frames, covers, and steps, if utilized, should be of suitable material and designed to accommodate prevailing site conditions and to provide for a safe installation. Manhole covers shall not have vent holes or a type of pick hole which allows entry of surface water. When venting is required, provide in accordance with paragraph 2.4.10.5. Materials used for manhole steps should be highly corrosion-resistant aluminum or plastic with reinforcing bar.

### 1.4.10.8 Vacuum Testing

New manholes should be vacuum tested after construction to verify they will not be new sources of I/I. The test shall include the manhole frame. The test is considered acceptable if the vacuum remains at 10 inches of mercury or drops to no less than nine inches of mercury



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within one minute. Alternative testing methods may be allowed if demonstrated to be equal or better than vacuum testing.

### 1.5 Force Main Design

#### 1.5.1 Size

Minimum size force mains shall not be less than 4 inches in diameter except for grinder pumps or septic tank effluent applications.

#### 1.5.2 Velocity

At pumping capacity, maintain a minimum self-scouring velocity of 3 feet per second (fps) unless flushing facilities are provided. Velocity should not exceed 8 fps.

#### 1.5.3 Air/Vacuum Relief Valve

Air/vacuum relief valves are required to all high points in the force main to relieve air locking and to protect against pipe collapse. Specifications should require valves designed specifically for wastewater service. Traditional valves have a history of maintenance issues. Designs should avoid the use of high points to the maximum extent possible and when necessary ensure that they are as conveniently located as possible. Lightweight valves with some non-metallic working components have increased valve reliability and reduced maintenance and repair expenses. When hydraulic transient analyses or risks exist in the system (see paragraph 2.5.6) the slow-closing type valves should be considered. Accessibility of the valves to promote exercising installed valves is an important consideration.

#### 1.5.4 Termination

The force main should enter the receiving manhole with its centerline horizontal and with an invert elevation that will ensure a smooth flow transition to the gravity flow section. In no case should the force main enter the gravity sewer system at a point more than one foot above the flow line of the receiving manhole. The design should minimize turbulence at the point of discharge.

Inert materials or protective coatings are required for the receiving manhole to prevent deterioration due to hydrogen sulfide or other chemicals where such chemicals are present or suspected to be present because of industrial discharges or long force mains.

#### 1.5.5 Friction Losses – Hazen-Williams Equation

A "C" factor that will take into consideration the conditions of the force main at its design usage is required. For example, a grease-coated pipe after several years will not have the same C factor as a new pipe. Unless more specific information is available, system curves with



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ductile iron discharge pipe should be shown for C=110 and C= 130. For PVC and HDPE discharge pipe, curves should be shown for C-130 and C=150.

### 1.5.6 Water Hammer

Force main systems are at greater risk from damaging water hammer (hydraulic transients) than potable water distribution systems due to the lack of branching to dissipate the pressure waves and storage tanks to act as damping reservoirs. Large diameter and high lift systems generally receive hydraulic transient analyses in the design phase while small diameter and low total dynamic head (TDH) systems usually do not generate severe conditions that compromise piping systems in the short or long run. Between these extremes, there are probably a number of systems that would benefit from mitigation of transient hydraulic effects to extend the life of system components due to metal fatigue. Equipment start-up or inspection requirements should include measurement and evaluation of hydraulic transients. The provision of adjustable cushioned check valves on pump discharges, soft-start or variable-speed pump drives, easily maintained and conveniently installed air/vacuum/combo release valves in the force main, and surge valves, as well as the avoidance of quick-closing automatic control valves upon loss of power can reduce the adverse effects of transients if the risk is classified as significant. Computer analysis is usually required to avoid problems at start-up. Two references are recommended for consultation:

- Pipeline Design for Water and Wastewater, American Society of Civil Engineers, New York (1975); and
- Robert L. Sanks et al., Pumping Station Design, 151 (2nd ed. 1998).

The latter reference gives the following checklist to determine system risk for hydraulic transients. "If two or more conditions exist, a surge will probably occur with a severity proportional to the number of conditions met:

- There are high spots in the pipe profile;
- There is a steep gradient;
- Length of force main is less than 20xTDH;
- Flow velocity is in excess of 4 ft/sec;
- Factor of safety (based on ultimate strength) of pipe (and valve and pump casings) is less than 3.5 for normal operating pressure;
- There can be slowdown and reversal of flow in less than  $t_c$  (critical time where  $t_c = 2L/a$  where L=length of pipeline in feet and a=elastic wave speed in water contained in the pipe in ft/sec);
- There is check valve closure in less than  $t_c$ ;
- There is any valve closure in less than  $t_c$ ;
- There can be damage to pump and motor if allowed to run backward at full speed;
- Pump can stop or speed can be reduced to the point where the shut-off head is less



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than static head before the discharge valve is fully closed;

- Pump can be started with discharge valve open;
- There are booster stations that depend on operation of the main pumping station; OR
- There are quick-closing automatic valves that become inoperative if power fails or pumping system pressure fails.”

### 1.5.7 Isolation and Valves

Installation of isolation valves at strategic locations along the force main to facilitate maintenance of the system is recommended. Accessibility of the valves to promote exercising installed valves is an important consideration.

### 1.6 Trenchless Technologies

Construction drawings for horizontal directional drilling and jack and bore must show the entire bore including:

- Sending and receiving pits.
- Distance from the base of the road or the bottom of the stream to the top of the casing pipe.
- Distance from the base of the ditch (if any) to the top of the casing pipe.
- Carrier pipe diameter, material, wall thickness and type of seams or joints.
- Casing pipe diameter, material, wall thickness and type of seams or joints.
- Means and methods of dewatering.

Entry and exit locations must be at least 50 feet from a stream bank or wetland margin. The depth of bore below the streambed must be sufficient to reasonably prevent release of drilling fluid, based on the soil or rock material as identified in the site characterization report. A site-specific contingency and containment plan for inadvertent release of drilling fluid must be established prior to commencement of work. This plan must include notification to the COF WMD upon release to surface waters.

#### 1.6.1 Casing Pipe Diameter

The inside diameter of the casing pipe shall be large enough to allow the carrier pipe to be removed without disturbing the casing. For pipe casings with a carrier pipe diameter less than 6 inches, the inside diameter of the casing pipe shall be at least 2 inches greater than the largest outside diameter of the carrier pipe joints or couplings. For steel pipe casings with a carrier pipe 6 inches in diameter or more, the inside diameter of the casing pipe shall be at least 4 inches greater than the largest outside diameter of the carrier pipe joints or couplings.



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### 1.6.2 Casing Pipe Thickness

The casing pipe must have sufficient thickness to withstand both earth loads and live loads imposed from traffic above. Coated steel pipe that is bored or jacked into place shall conform to the wall thickness requirements for uncoated steel pipe since the coating may be damaged during installation.

### 1.6.3 Casing Pipe Spacers

Casing pipe spacers shall be used as needed to offset the carrier pipe within the casing.

### 1.7 Construction Materials, Installation, and Testing

The COF WMD will consider any generally accepted material for sewers. The material selected should be adapted to local conditions such as character of industrial wastes, possibility of septicity, soil characteristics, exceptionally heavy external loadings, internal erosion, corrosion, abrasion, and similar problems. Carefully consider pipe and compression joint materials subjected to corrosive or solvent wastes. Chemical/stress failure and stability in the presence of common household chemicals such as cooking oils, detergents, and drain cleaners are factors.

The specifications should contain appropriate requirements based on the criteria, standards, and requirements established by the industry in its technical publications and stipulate the need to keep the pipe interior, sealing surfaces, fittings, and other accessories clean.

Protection of stored pipe is required. Pipe bundles should be stored on flat surfaces with uniform support. Pipe with prolonged exposure (six months or more) to sunlight requires a suitable covering (canvas or other opaque material). Ensure that gaskets are not exposed to oil, grease, ozone (produced by electric motors), excessive heat and direct sunlight. Consult with the manufacturers for specific storage and handling.

Requirements should be set forth in the specifications for the pipe and methods of bedding and backfilling thereof so as not to damage the pipe or its joints, impede cleaning operations, not create excessive side fill pressures or ovality of the pipe, nor seriously impair flow capacity.

Design all pipes to prevent damage from superimposed loads. Proper design allowance for loads on the pipe because of the width and depth of trench is required.



## City of Franklin, TN Delegated Authority Design Criteria for Sanitary Sewer

### 1.7.1 Pipe Standards

#### 1.7.1.1 Rigid Pipe

Rigid pipe includes, but is not be limited to, concrete pipe. Any rigid pipe should have a minimum crushing strength of 2000 pounds per linear foot. All pipes should meet the appropriate ASTM and/or ANSI specifications.

#### 1.7.1.2 Semi-rigid Pipe

Semi-rigid pipe includes, but is not be limited to, ductile iron pipe. All pipes should meet the appropriate ASTM and/or ANSI specifications.

#### 1.7.1.3 Flexible Pipe

Flexible pipe includes, but is not be limited to, ABS solid wall pipe, polyvinyl chloride pipe (PVC), polyethylene pipe (PE), fiberglass composite pipe, reinforced plastic mortar pipe (RPM) and reinforced thermosetting resin pipe (RTR). PVC pipe should have a Standard Dimension Ratio (SDR) number of 35 or lower (i.e. SDR 26) for gravity lines. All flexible pipe shall have a minimum calculated deflection under identical conditions similar to the SDR 35 PVC pipe. The force main pipe and fittings, including all restrained joint fittings and thrust blocking, shall be designed to withstand pump operating pressures and pressure surges, but not less than 150 psi. All pipes should meet appropriate ASTM and/or ANSI specifications. ASTM D-3033 and D-3034 PVC pipes differ in wall thickness and have non-interchangeable fittings.

### 1.7.2 Pipe Bedding and Backfilling

Provide protection from damage due to superimposed loads. The width and depth of the trench require allowances to be made for loads on the sewer. Backfill material up to three feet above the top of the pipe should not exceed 6 inches in diameter at its greater dimension.

Ductile iron pipe is required in roadways where cover is less than 4 feet. In such cases, a minimum cover of six inches is required. Ductile iron pipe is required when the top of the sewer is less than 18 inches below the bottom of a conveyance. Specifications should require specific installation procedures to ensure pipe manufacturer's installation recommendations or other industry standards are adhered to in order promote proper installation, which may be the single most important factor in pipe longevity and reduction in leak development. Pipe susceptibility to French drain effects happens at places other than where the main concern is stream capture; the scouring of pipe bedding when check dams are not provided where appropriate may hasten pipeline failures.



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### 1.7.2.1 Rigid Pipe

Bedding Classes A, B, or C as described in ASTM C-12 or WPCF MOP No. 9 (ASCE MOP No. 37) should be used for all rigid pipe, provided the proper strength pipe is used with the specified bedding to support the anticipated load.

### 1.7.2.2 Semi-Rigid Pipe

Utilize Bedding Classes I, II, III, or IV (ML and CL only) as described in ASTM D-2321 for all semi-rigid pipe provided with the specified bedding to support the anticipated load. Utilize ASTM-A-746 to install ductile iron pipe.

### 1.7.2.3 Flexible Pipe

Utilize Bedding Classes I, II, or III as described in ASTM D-2321 for all flexible pipe for bedding, haunching, initial backfill, and backfill. Provide the proper strength pipe with the specified bedding to support the anticipated load. Alternatively utilize Class I bedding material for bedding, haunching, and initial backfill as described in 2.7.2.4. (polyethylene pipe).

### 1.7.2.4 Alternate Bedding Option

Sewers may be bedded and backfilled with a minimum of 12 inches of Class I material over the top and below the invert of the pipe as an alternative to sub-sections 2.7.2.1, 2.7.2.2 and 2.7.2.3.

### 1.7.2.5 Deflection Testing

All flexible pipes require deflection testing after backfill has been in place at least 24 hours. No pipe should exceed a deflection of 5 percent.

Run the test with a rigid ball or an engineer approved 9-arm mandrel having a diameter equal to 95 percent of the inside diameter of the pipe and manually pulling the test device through the line.

### 1.7.3 Joints

Include the method of making joints and the materials used in the specifications. Sewer joints shall impede infiltration and prevent the entrance of roots.

Elastomeric gaskets, other types of pre-molded (factory made) joints, and ABS solvent-cement



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welded joints are required. Utilize ASTM-F2620 for butt-fusion joining technique with polyethylene pipe. Remove internal beads for butt-fusion joints on pipelines with slopes less than one percent. Cement mortar joints are not acceptable. Field solvent welds for PVC and PE pipe and fittings are not acceptable.

### 1.7.4 Leakage Testing for Gravity Lines

Utilize ASTM-C-828 or F1417 for low-pressure air testing for all gravity pipes. The time required for the pressure to drop from the stabilized 3.5 pounds per square inch gauge (psig) to 2.5 psig should be greater than or equal to the minimum calculated test time (air loss rate shall be part of the test criteria).

The testing method should take into consideration the range in groundwater elevations projected and the situation during the test. The height of the groundwater should be measured from the top of the invert (one foot of H<sub>2</sub>O = 0.433 psi).

Table 2-3 provides the minimum test times and allowable air loss values for various pipe size per 100 ft.

**Table 2-3 Leakage Test Parameters**

<b>Pipe Size</b> (inches)	<b>Time, T</b> (sec/100 ft)	<b>Allowable Air Loss, Q</b> (ft <sup>3</sup> /min)
6	42	2.0
8	72	2.0
10	90	2.5
12	108	3.0
15	126	4.0
18	144	5.0
21	180	5.5
24	216	6.0
27	252	6.5
30	288	7.0

### 1.7.5 Visual Inspection of Gravity Lines

Video inspection is required to confirm proper installation and to provide a visual record of the condition of the newly constructed sewer for future reference.

### 1.7.6 Pressure Tests for Force Mains

Test all force mains before backfilling at a minimum pressure of at least 50 percent above the design operating pressure for at least 30 minutes. Test in accordance with the applicable



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provisions of American Water Works Association (AWWA) Standard C605-94, Section 7. Leakage should not exceed the amount given by the following formula:

$$L = ND (P)^{.5} / 7,400$$

Where:

**L** is allowable leakage in gallons per hour

**N** is the number of pipe joints

**D** is the pipe diameter in inches

**P** is the test pressure in psi



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### 1.7.7 Anchorage for Force Mains

Anchor force mains within the pumping station and throughout the line length to include, thrust blocks, restrained joints, and/or tie rods.

### 1.8 Protection of Water Supplies

#### 1.8.1 Water Supply Interconnections

There shall be no physical connection between a public or private potable water supply system and a sewer or appurtenance thereto.

#### 1.8.2 Relation to Water Mains

**Horizontal Separation:** Maintain at least 10 feet of horizontal separation between the sewer and any existing or proposed water main measured edge to edge. Should local conditions prevent a lateral separation of 10 feet, sewers closer than 10 feet to a water main will be considered if in a separate trench and if the elevation of the top (crown) of the sewer is at least 18 inches below the bottom (invert) of the water main.

**Vertical Separation:** Whenever sewers must cross under water mains, install the sewer at such elevation that the top of the sewer is at least 18 inches below the bottom of the water main. Alternatives will be considered if the sewer elevation cannot be varied.

When it is impossible to obtain proper horizontal and vertical separation as stipulated above,

- The sewer should be designed and constructed equal to the water main pipe and should be pressure-tested to assure water-tightness (see drinking water criteria), or
- Either the water main or the sewer line may be encased in a water-tight carrier pipe which extends 10 feet on both sides of the crossing, measured perpendicular to the water main.

#### 1.8.3 Backflow Preventers

State approved reduced pressure backflow prevention devices are required on all potable water mains serving the wastewater treatment plant or pumping station. The COF WMD can provide a list of approved backflow preventers.



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### 1.9 Collection System Elements in Relation to Streams

#### 1.9.1 General

It is the owner's responsibility to obtain all necessary permits along streams, rivers or wetlands (i.e., United States Army Corps of Engineers, the Tennessee Valley Authority [TVA], and/or the Tennessee Department of Environment and Conservation [TDEC] of Water Resources.). The ARAP must be issued prior to the approval of final construction plans.

#### 1.9.2 Site Evaluations

Once a project site is identified, two evaluations are required in order to proceed in the Aquatic Resources Alteration Permit (ARAP) process, the Water Resource Inventory and the Site Characterization Report.

##### 1.9.2.1 Water Resource Inventory

A Water Resource Inventory (WRI) identifies streams, wetlands, ponds, lakes and wet weather conveyances on the site. A wetland delineation report and/or Hydrologic Determination report (per rule 0400-40-17-.04) will be generated by the consultant and submitted to the TDEC at the local field office for concurrence. The number of stream and wetland crossings, trenching techniques, installation and stabilization methods, and permanent erosion protection determines the permitting scenario for the project.

##### 1.9.2.2 Site Characterization Report

A Site Characterization Report (SCR), submitted by the applicant, evaluates the potential of a French drain effect of the utility trenches in the proximity of streams or wetland. Flows from shallow groundwater table, wetlands and interflows from valleys to the streams must be identified as well as the type of and the depth to the restrictive layer (aquitar) and/or bedrock with respect to the stream bottom and trench depth and utility alignment. Areas with karst geology should be evaluated for the possibility of jointing, solution development, and creation of preferential flow paths.

The two areas of vulnerability for capture of the flow in the trench are at stream/wetland crossings and at areas where the trench parallels the inflow of shallow groundwater to the stream/wetland. The site characterization report documents the potential, or lack thereof, for flows entering the pipeline trench. The site characterization report also documents the depth of parent material below the stream bed is sufficient to reasonably prevent the release of drilling fluid.



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### 1.9.3 Permitting Scenarios for Aquatic Resource Alterations

Three scenarios apply to the ARAP process:

#### **Scenario 1: No ARAP application and no written authorization required**

Activities involving verified wet weather conveyances do not require ARAP application and must be conducted in accordance with conditions found in Tenn. Code Ann. § 69-3-108(q).

Some types of crossings are covered under a General Permit for Utility Line Crossings without the submittal of an application or written authorization from the Division. There is no permit processing period for this permitting scenario. Note that the General Permit is re-issued every five years and requirements of the most recent permit should be followed. The types of such activities identified in the General Permit (in effect April 7, 2015) include:

- Activities involving streams or wetlands utilizing pipe bursting, slip lining, or other non-invasive technologies conducted in accordance with permit conditions for utility line crossings.
- Activities involving streams or wetlands where lines are suspended from culverts and bridges or similar structures conducted in accordance with permit conditions for utility line crossings.
- Activities involving streams or wetlands utilizing up to three horizontal directional drilling crossings conducted in accordance with permit conditions for utility line crossings, provided no aquatic species listed as threatened, endangered, or deemed in need of management by State or Federal authorities is located within one mile of the project location. The following conditions must be met:
  - Entry and exit locations must be at least 50 feet from the stream bank or wetland margin.
  - The depth of bore below the stream bed must be sufficient to reasonably prevent the release of drilling fluid, based on parent material.
  - A site-specific contingency and containment plan for the inadvertent release of drilling fluid must be established prior to commencement of work. This plan must include notification to the Division upon release to surface waters.

Where non-invasive techniques are specified for stream or wetland crossing, no ARAP application is required; design considerations listed in Section 2.9.4.1 must be specified on the plans.

Where horizontal directional drilling is specified and the number of stream or wetland crossings does not exceed the number eligible under the no-notification provisions of the current General Permit, design considerations listed in Section 2.9.4.1 and 2.9.4.2 must be



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specified on the plans.

For trenches parallel to and within 50 feet of a stream or wetland where potential for the French drain effect has been identified in the site characterization report, design considerations in Section 2.9.4.4 must be specified on the plans.

### **Scenario 2: General ARAP Permit for Utility Line Crossings**

The following types of crossings are covered under a General Permit for Utility Line Crossings when a complete application is submitted to the local field office and a written authorization from the Division NOC is issued. Note that the General Permit is re-issued every five years and requirements of the most recent permit should be followed:

- Horizontal directional drilling – no limit on the number of crossings.
- Open trenching, jack and bore and auger boring – maximum of 5 crossings.

For projects requiring a general ARAP, complete application, approved water resource inventory report, and a site characterization report must be submitted with the construction documents. The typical permit processing period is 30 days or fewer for this permitting scenario. Construction documents will be reviewed and approved once the ARAP general coverage is issued. The typical plans review period is 30 days or fewer.

Where horizontal directional drilling is specified and the number of stream or wetland crossings exceeds the no-notification threshold of the current General Permit, ARAP application is required. Notices of coverage are issued by the environmental field office and shall be included in the plans submittal. Applicable specifications listed in section 2.9.6 must be specified on the plans.

Where open trenching, jack and bore and auger boring is specified, the number of stream and/or wetland crossings determines which permitting scenario applies. For up to five crossings, the general permit applies and a complete ARAP application is required. A NOC may be processed by the regional environmental field office and shall be included in the plans submittal. Design considerations listed in section 2.9.4 must be specified on the plans.

For trenches parallel to and within 50 feet of a stream or wetland where potential for the French drain effect has been identified in the site characterization report, design considerations in Section 2.9.4.4 must be specified on the plans.

### **Scenario 3: Individual ARAP Permit**

Crossings outside the scope of Scenario 1 and 2 or part of cumulative impacts that would require coverage under an Individual ARAP permit are eligible to submit a completed application for an individual permit specific to aquatic alteration to TDEC's central office at William R. Snodgrass Tennessee Tower 312 Rosa L. Parks Avenue, 11th Floor, Nashville, Tennessee 37243. The types of such activities are described below:



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- Activity exceeds the scope of Scenario 1 and 2 or is part of cumulative impacts;
- Activity does not meet conditions of the general permit for utility line crossings;
- Activity requires blasting; and/or
- Activity impacts waters designated as National Wild and Scenic Rivers, Outstanding Natural Resource Waters or Federal or State listed deemed in need of management, threatened, or endangered aquatic species or habitat are located within one mile of the project location.

For crossings requiring authorization under an individual ARAP, a complete application, water resource inventory report, and a site characterization report must be included with the submittal to the central office. ARAPS are issued or denied within 90 days of the date the Division

determines an application is complete. If a public hearing is scheduled, an additional 90 days is added to the time limit. Due to the longer permit processing period, it is advised to submit preliminary plans with the application in advance of submitting construction documents for approval. Construction documents will be reviewed and approved once an ARAP is in on public notice. The typical plans review period is 30 days or less.

Where crossings exceed or are outside of the scope of general permit for utility line crossing or are part of cumulative impacts, submit a complete application for an individual permit to the Nashville central office. Submittal of preliminary plans is advised in order to process the individual ARAP. Applicable specifications must be specified in the preliminary plans and the final construction documents.

Where open trenching, jack and bore and auger boring is specified, design considerations in Section 1.9.4.1, 1.9.4.3, and 1.9.4.4 must be specified on the plans.

For trenches parallel to and within 50 feet of a stream/wetland where potential for French drain effect has been identified in the site characterization report, design considerations in Section 1.9.4.4 must be specified on the plans.

Where blasting is specified for stream/wetland crossing or for trench excavation within 50 feet of a stream/wetland specifications in Section 2.9.4 must be clearly specified on the plans and in the permit application narrative.

1.9.4 Design considerations involving sewer line stream crossings.

1.9.4.1 Design considerations for all sewer lines crossing streams or wetland:

- Sewer systems shall be designed to minimize the number of stream crossings. Sewer lines shall intersect the stream channel as close to perpendicular as possible and at no less than



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- 45 degrees to minimize length and degree of disturbance of the stream bed. Sewers crossing streams shall be free from changes in grade. Maximize the manhole distance from the stream's edge (minimum of 50 feet is recommended). Avoid placing manholes in an open water, permanently inundated wetland. Where practicable, sewer line replacements should utilize the existing trench.
- Latitude and longitude of each crossing should be included on the cover sheet and on the plans. Provide detail drawings of all stream crossings with elevations of the streambed and of normal and extreme high and low water levels to the 100-year flood plain, as established by FEMA. Plans should clearly mark each crossing with pipe profiles, elevation of adjacent streams, the extent of streamside vegetation where the trench parallels stream within 50 feet, and where flowable fill/plugs/check dams/anti-seep collars/manholes/etc. are proposed. Identify the number and location of all stream crossings in the cover letter when submitting construction documents for review.
  - The crossing shall be designed to prevent the impoundment or loss of normal or base flows. Aquatic organism passage must be preserved.
  - The excavation and fill activities associated with utility line crossing of non-navigable streams shall be kept to a minimum and shall be separated from flowing waters. The crossing shall be constructed in the dry to the maximum extent practicable, by diverting flow utilizing cofferdams, berms, temporary channels or pipes.
  - Temporary diversion channels shall be protected by non-erodible material and lined to the expected high-water level.
  - All contours must be returned to pre-project conditions to the extent practicable, and not disrupt or impound stream flow.
  - Temporary impacts to wetlands shall be mitigated by the removal and stockpiling of the first 12 inches of topsoil prior to construction. Upon completion of construction activities, all temporary wetland impact areas are to be restored to pre-construction elevations. Other side-cast material shall not be placed within the temporary impact locations. Permanent vegetative stabilization using native species of all disturbed areas in or near the wetland must be initiated within 14 days of project completion (see also *Landscaping with Native Plants*, SOUTHEAST EXOTIC PEST PLANT COUNCIL (<https://www.se-eppc.org/pubs/landscape.html>)). Non-native, non-invasive annuals may be used as cover crops until native species can be established.
  - Material:
    - Gravity sewers entering or crossing streams should be ductile iron pipe from manhole to manhole, wrapped in plastic, and encased in flowable fill. This provision is subject to a case-by-case review. The best practice depends on stream flow, local soils, topography, and geology.
    - Force mains entering or crossing streams should be HDPE pipe, fusible PVC or DIP with mechanical joints or restrained mechanical joints with approval of the utility.
  - Depth:



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- The top of all sewers entering or crossing streams shall be at a sufficient depth below the natural bottom of the stream bed to protect the sewer line. In general, provide the following cover requirements:
  - One foot of cover where the sewer is located in rock;
  - Three feet of cover in other material. In major streams, more than 3 feet of cover may be required; and
  - In paved stream channels, the top of the sewer line should be placed below the bottom of the channel pavement.
  - Less cover may be approved only if the proposed sewer crossing will not interfere with future modifications to the stream channel. Justification for requesting less cover shall be provided to the COF WMD.
- Flotation:
  - Evaluate the possibility of pipe flotation when the pipeline is constructed in areas which will be inundated, such as stream crossings, flood plains, and high groundwater areas.

### 1.9.4.2 Design considerations for crossings involving horizontal directional drilling

Locate entry and exit bore pits with adequate depth to prevent frac-out and reduce over-excavation. Location of bore pits and bore pit spill piles must be identified on the plans.

The depth of bore below the stream bed must be sufficient to reasonably prevent the release of drilling fluid, based on the parent material as identified in the site characterization report.

A site-specific contingency and containment plan for the inadvertent release of drilling fluid must be established prior to commencement of work. This plan must include notification to the division upon release to surface waters.

### 1.9.4.3 Design considerations for crossings involving bedrock streams and karst geology

Provisions shall be made to prevent the loss of stream flow due to fracturing of the bedrock. Sewer line crossing streams with bedrock streambeds must provide non-erodible fill and cover, such as concrete or controlled low strength materials (flowable fill), and trench plugs at each end of the crossing.

### 1.9.4.4 Design considerations for crossings involving trenches parallel to streams and wetlands

- Trench Plugs:
  - Trench plugs are barriers placed within an open pipeline excavation in order to slow



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flow and reduce erosion in the trench and to prevent the trench from becoming a subsurface drainage path. Since the bedding and embedment are constructed using cohesionless, free-draining soils, a path is created for water to flow easily (French drain effect) alongside the pipe. In areas where there is high groundwater, where the pipeline crosses streams or aquifers, or where the natural groundwater flow would be affected or even diverted by the select material, trench plugs of compacted, cohesive, soils or impervious materials should be constructed at intervals along the pipeline.

- The trench plug area will have a bedding of compacted, cohesive soils or impervious materials, whereas the bedding on both sides of the trench plug will have a bedding of uncompacted, cohesionless soil.
- Location and spacing of trench plugs: Minimum of one trench plug between manholes, and one trench plug at each end of the stream crossing or wetland. The trench plugs between manholes shall be located near the upstream manhole.
- Aerial Crossings:
  - Sewers that lay on piers across ravines or streams may be allowed if no other practical alternative exists or, in the design engineer's judgment, other methods will not be as reliable.
  - Support shall be provided for all joints in pipes utilized for aerial crossings. The supports shall be designed to prevent frost heave, overturning, and settlement.
  - Provide precautions against freezing, such as insulation and increased slope.
  - Provide expansion joints between above ground and below ground sewers. Where buried sewers change to aerial sewers, use special construction techniques to minimize frost heaving.
  - For aerial stream crossings, the impact of floodwaters and debris shall be considered. The bottom of the pipe should be placed no lower than the elevation of the 50-year flood. Ductile iron pipe with mechanical joints is recommended.
  - The design should consider maintenance of an adequate waterway for the 100-year flood flows and analyze the impact of the proposed aerial crossing(s) on flooding, including hydraulic modeling, such as Hydrologic Engineering Center-River Analysis System (HEC-RAS) modeling, as necessary.

### 1.10 General Requirements for Wastewater Pumping Stations

#### 1.10.1 Location and Flood Protection

Locate wastewater pumping stations as far as practicable from present or proposed built-up residential areas, with an all-weather road and noise control, odor control, and station architectural design taken into consideration. Sites for stations should be of sufficient size for future expansion or addition, if applicable. Provide security for the pumping station and controls. Protect the station's operational components from the 100-year flood. Where high groundwater conditions are anticipated, buoyancy of the wastewater pumping station structures shall be considered and, if



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necessary, adequate provisions shall be made for protection. Connect to the pumping station at an elevation higher than the maximum water table elevation, where possible.

### 1.10.2 Pumping Rate and Number of Units

Provide at least two pump units, each capable of handling the expected maximum flow. For three or more units, design the station for actual flow conditions and to be of such capacity that, with any one unit out of service, the remaining units will have capacity to handle the maximum wastewater flow. A pump station expected to operate at a flow rate less than one-half the average design flow for an extended period may create septic conditions due to long holding times in the wet well. The design should consider the need for additional measures to prevent the formation of odors.

The design should consider the use of variable-speed or multiple staged pumps, particularly when the pumping station delivers flow directly to a treatment plant. The design should allow delivery of the wastewater at approximately the same rate as received at the pumping station.

### 1.10.3 Grit and Clogging Protection

Where it may be necessary to pump wastewater prior to grit removal, the design of the wet well should receive special attention, and the design of the discharge piping should prevent grit settling in pump discharge lines of pumps not operating.

Design of the pumping station should consider the protection of the pump from damage caused by grit and debris, where warranted. To accomplish this, maintain minimum pump operational speeds through the installation of bar screens with a grinder or comminutor, or similar devices. For larger or deeper stations, duplicate protection units, each sized at full capacity, are preferred.

### 1.10.4 Pumping Units

#### 1.10.4.1 Pump Openings

Pumps shall be capable of passing a 3-inch compressible solid. Pump suction and discharge openings shall be at least 4 inches in diameter unless the pump has chopping or grinding capabilities.

#### 1.10.4.2 Priming

Locate pumps so that under normal operating conditions they will operate under a positive suction head.



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### 1.10.4.3 Intake

Each pump should have an individual intake. Wet well design should seek to avoid turbulence near the intake.

### 1.10.4.4 Valves and Piping

Provide suitable shutoff valves on suction and discharge lines of each pump for normal pump isolation and a check valve on each discharge line between the shutoff valve and the pump. Pump suction and discharge piping should not be less than four inches in diameter except where design of special equipment allows. The velocity should not exceed six feet per second in the suction line and eight feet per second in the discharge piping. A separate shutoff valve is desirable on the common line leaving the pumping station.

### 1.10.4.5 Controls

Locate the controls to ensure that they will not be affected by flows entering the wet well, by the suction of the pumps, or by proximity to wet well walls. Controls must be able to activate additional pumps if the water in the wet well continues to rise. Controls can be float switches, air-operated pneumatic, radar, ultrasonic or capacitance probe types. Provisions should be made to automatically alternate the pumps in use. Pumping stations with motors and/or controls below grade should be equipped with a secure external disconnect switch. Consider an “intrinsically safe” power source if float switches are used. Consider redundant controls and/or remote monitoring to assist in preventing overflows.

### 1.10.4.6 Ventilation

Provide ventilation for all pumping stations during all periods when the station is manned. Portable ventilation equipment is acceptable for small pumping stations. Mechanical ventilation is required if screens or mechanical equipment, which might require periodic maintenance and inspection, are located in the wet well. In pits over 15 feet deep, multiple inlets and outlets are desirable. Dampers are not be used on exhaust or fresh air ducts, and fine screens or other obstructions in air ducts should be avoided to prevent clogging.

### 1.10.5 Flow Measurement

Provide flow measuring instrumentation suitable for measuring flow at pumping stations with flow capacity greater than 0.5 million gallons per day (MGD) to determine pump operation and calculate I/I. Include provisions to record discharge flow rate and change in wet well storage volume as well as a rain gauge to measure rainfall at the station. The system should have the capability to utilize the recorded values to calculate I/I.



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### 1.10.6 Alarm System

An alarm system is recommended for all pumping stations such as telemetry alarm to 24-hour monitoring stations or telephone alarms to duty personnel (when reliability classification or property damage warrants it). An audiovisual device is required at the station for external observation when telemetry is not used.

Alarms for high wet well and power failure are required, as a minimum, for all pumping stations. For larger stations, alarms signaling pump and other component failures or malfunctions are required.

A backup power supply is required, such as a battery pack with an automatic switchover feature for the alarm system, such that a failure of the primary power source will not disable the alarm system. The alarm system must be tested and verified that it is in good working order.

### 1.10.7 Overflows

Design and build pumping stations without any type of overflow or bypass structure.

### 1.10.8 Materials

The use of concrete additives or protective coatings to prevent deterioration caused by corrosive gases is recommended.

### 1.10.9 Electrical Equipment

Electrical systems and components (e.g., motors, lights, cables, conduits, switch boxes, and control circuits) in enclosed or partially enclosed spaces where flammable mixtures occasionally may be present (including raw wastewater wet wells) should comply with the National Electrical Code requirements for Class I Division 1 locations.

### 1.10.10 Water Supply

Pump stations should include a frost-free hose bibb and a reduced pressure backflow preventer with double-check valves or other means of washing down the station (i.e. wash truck). Maintain no physical connection between any potable water supply and a wastewater pumping station that under any conditions might cause contamination of the potable water supply. A potable water supply must comply with conditions stipulated in Section 2.8.

### 1.10.11 Lighting

Adequate lighting is required for the entire pumping station.

### 1.10.12 Pump and Motor Removal



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Make provisions for the removal of pumps, motors, and other equipment, without interruption of system service.

### 1.10.13 By-Pass Pump Facilities

All new pump stations or modifications should include by-pass pump facilities capable of passing the pumping station design capacity.

### 1.10.14 Safety

Provide suitable and safe means of access to equipment requiring inspection or maintenance and stairways and ladders that satisfy all OSHA requirements.

## 1.11 Requirements for Specific Types of Wastewater Pumping Stations

### 1.11.1 Wet Well - Dry Well Stations

#### 1.11.1.1 Separation

Separate wet and dry wells, including their superstructures. Divide the wet well into two sections, properly interconnected, to facilitate repairs and cleaning where continuity of pumping station operation is necessary.

#### 1.11.1.2 Wet Well Size and Design

Provide an evaluation of the effective capacity of the wet well based on pumping requirements and reliability classifications. Wet well design should consider approaches for minimizing solids deposition.

#### 1.11.1.3 Dry Well Dewatering

Provide a separate sump pump in the dry wells, discharging to the wet well, to remove leakage or drainage with the discharge above the high-water level of the wet well. Water ejectors connected to a potable water supply will not be approved. All floor and walkway surfaces should have an adequate slope to a point of drainage.

### 1.11.2 Suction Lift Stations



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### 1.11.2.1 Priming

Conventional suction-lift pumps should be of the self-priming type, as demonstrated by a reliable record of satisfactory operation. The maximum recommended lift for a suction lift pumping station is 15 feet, using pumps of 200 gallons per minute (gpm) capacity or less.

### 1.11.2.2 Capacity

The capacity of suction lift pumping stations is limited by the net positive suction head and specific speed requirements, as stated on the manufacturer's pump curve, for the most severe operating conditions.

### 1.11.2.3 Air Relief

An air relief line on the pump discharge piping is required for all suction lift pumps. This line should be located at the maximum elevation between the pump discharge flange and the discharge check valve to ensure the maximum bleed-off of entrapped air. The air relief line should terminate in the wet well or suitable sump and be open to the atmosphere.

### 1.11.2.4 Pump Location

For standard designs, suction lift pumps are mounted on the wet well but not within the wet well. If the pumps are not located above the wet well, the pump manufacturer should be consulted with respect to the maximum length of the suction line.

### 1.11.2.5 Access to Wet Well

Access to the wet well should not be through the dry well, and the dry well should have a gastight seal when mounted directly above the wet well.

## 1.11.3 Submersible Pumps

### 1.11.3.1 Pump Removal

Submersible pumps should be readily removable and replaceable without dewatering the wet well or requiring personnel to enter the wet well. A hoist or crane system for removing the pumps from the wet well either through a permanent installation at the site or a mobile system for use at multiple sites is recommended.



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### 1.11.3.2 Controls

The control panel should be located outside the wet well and suitably protected from weather, humidity, vandalism, and gases migrating from the wet well.

### 1.11.3.3 Valves

Control valves are recommended on the discharge line of each pump, in a convenient location outside the wet well, in separate pits protected from weather and vandalism.

### 1.11.3.4 Submergence

Positive provision, such as backup controls, is required to assure submergence of the pumping units.

### 1.11.4 Grinder and Effluent Pumps

The requirements for grinder and effluent pumps are included in Section 2.14.

## 1.12 Operability and Reliability for Wastewater Pumping Stations

### 1.12.1 Objective

The objective of reliability is to prevent the discharge of raw or partially treated wastewater to any waters and to protect public health by preventing backup of wastewater and subsequent discharge to basements, streets, and other public and private property.

### 1.12.2 Backup Units

A minimum of two pumps or pneumatic ejectors are required in each station in accordance with Section 2.11.2.

### 1.12.3 Power Outages

An emergency power source or auxiliary power is required for all pumping stations larger than 1 MGD to ensure continuous operability unless experience has shown the frequency and duration of outages to be low and the pumping station and/or sewers provide storage sufficient for expected interruptions in power service.



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### 1.12.4 Emergency Power Supply

#### 1.12.4.1 General

Emergency power supply provisions for pumping stations to at least two independent public utility sources, or by provision of portable or in-place internal combustion engine equipment that will generate electrical or mechanical energy, or by the provision of portable pumping equipment is required. Emergency power must be provided for all stations which are 1 MGD or larger, or as determined by the reliability classification.

Emergency power should be provided that, alone or combined with storage, will prevent overflows from occurring during any power outage that is equal to the maximum outage in the immediate area during the last 10 years. If data were available for fewer than 10 years, an evaluation of a similar area served by the power utility for 10 years would be appropriate.

#### 1.12.4.2 In-Place Equipment

The utilization of in-place internal combustion equipment requires the following guidelines:

- Placement: bolted in place with facilities for unit removal for purposes of major repair or routine maintenance.
- Controls: automatic and manual startup and cut-in.
- Size: adequate to provide power for pumps as well as lighting and ventilation systems and such further systems that affect capability and safety.
- Engine Location: located above grade, with suitable and adequate ventilation of exhaust gases.
- Underground Fuel Storage Tank: design and construction must conform to the applicable requirements of Federal Regulations 40 CFR parts 280 and 281. Contact the Division of Underground Storage Tanks, for guidance: <https://www.tn.gov/environment/program-areas/ust-underground-storage-tanks.html>.

#### 1.12.4.3 Portable Equipment

Portable pumping units should have connections to operate between the wet well and the discharge side of the station. The station should be provided with permanent fixtures that will facilitate rapid and easy connection of lines to portable equipment.

#### 1.12.4.4 Explosion Proof Electrical Fittings/Controls

National Fire Protection Association (NFPA) Codes and Standards that may be applicable to wastewater projects are listed in the Bibliography for this chapter. Their applicability varies by



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jurisdiction and project specifics and the design agent must use due diligence and best professional judgment in referencing the standards in specific projects' design or specifications.

### 1.12.5 Storage

Provide wet well and tributary main capacity above the high-level alarm sufficient to hold the peak flow expected during the maximum power outage duration during the last 10 years. If provisions are made for a by-pass pump or portable emergency generator, the storage should be adequate for the operator to have time to put them into operation once they are notified of an outage in order to prevent an overflow or release at the station.

### 1.13 Alternative Sewers

Design of septic tank effluent pump (STEP), septic tank effluent gravity (STEG), or grinder pump systems shall include, at a minimum, determining the peak-hydraulic grade line, matching the peak-hydraulic grade line to the individual pump curve or elevation of the STEG units, sizing the holding vessel based on estimated or actual wastewater flows, and designing system appurtenances required to provide a reliable municipal system. See Section 2.2.3 for ownership requirements.

#### 1.13.1 Pressure Sewer Systems

For systems served by grinder pumps, all raw wastewater should be collected from individual buildings/dwellings and transported to the pressure or gravity system by appropriately sized pumps. For restaurants or facilities with commercial-grade kitchen facilities, grease and oil interceptors should be installed prior to the grinder pump.

All pumps must have adequate operating curves that allow for pumping into the pressurized common line under maximum head conditions. Additionally, each pump must be equipped with properly installed and approved backflow prevention assembly. Furthermore, tanks must be watertight and located above the seasonal groundwater table where possible. Where it is not possible to locate tanks above the seasonal groundwater table, the design engineer must provide anti-buoyancy calculations and specify appropriate anti-flotation devices. Installations should ensure that odors are minimized.

##### 1.13.1.1 Design Flows

To determine the number of pumps operating at any time, the following two methods may be used:

- Probability method – best applies to pumps with vertical or near vertical head/ discharge curves. Many pump suppliers provide charts to determine the number of pumps operating simultaneously. One example is provided below.



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Number of grinder pump cores connected	Maximum daily number of grinder pump cores operating simultaneously
1	1
2 to 3	2
4 to 9	3
10 to 18	4
19 to 30	5
31 to 50	6
51 to 80	7
81 to 113	8
114 to 146	9
147 to 179	10
180 to 212	11
213 to 245	12
246 to 278	13
279 to 311	14
312 to 344	15

(Source: Low Pressure Sewer Systems Using Environment One Grinder Pumps Table 3)



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- Rational method - best applies to centrifugal grinder pumps

$$Q = AN+B$$

Where:

Q = Design flow (gpm)

A = Coefficient (typically 0.5)

N = Number of equivalent dwelling units

B = Minimum flow from the first pump or the total contributing flow from a group of pumps (typically 20)

$$No \frac{p}{d} (p) 1 + \frac{14}{4 \frac{JNo(ot)No(d)}{100}}$$

Where:

$No(\frac{p}{d})$  = number of persons per dwelling

$Q(p)$  = flow per person

$No(lot)$  = Number of lots

$No(d)$  = Number of dwellings

### 1.13.1.2 Peak Flows

Reserve space between the pump on and high-level alarm can be calculated using:

$$Q = (V-S)/t$$

Where:

Q = minimum required pump discharge rate (gpm)

V = volume of peak flow from home = 400 percent of average daily flow (gal)

S = Storage volume between pump on and alarm (gal)

t = time (minutes)



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### 1.13.1.3 Minimum Flow velocities in Pipes

A velocity in alternative sewer systems of 3 ft/sec should occur a minimum of once per day.

### 1.13.1.4 Retention time

For wastewater retained in the pipe for more than 30 minutes, provide protection for the discharge manhole from hydrogen sulfide.

### 1.13.1.5 Pipe Materials

See Section 1.7 Construction materials, Installation, and Testing

### 1.13.1.6 Grinder Pump Components

- Install resilient seated gate valves or fully ported ball valves on the upstream side of intersections and on each side of areas where disruption of the main can be anticipated.
- Install cleanouts at the ends of all pipelines, at changes in diameter and intervals of no more than 500 feet.
- Install air release and combination air release assemblies in the system. Make provisions for odor absorption if odors would be a nuisance or due to conditions at a station.

### 1.13.1.7 Service lines

Install service lines from the pump to the force main, typically 0.75 to 1.5 inches in diameter. Provide two check valves with one at the pumping unit and the other at the street right-of-way or curb. Install one shut off valve between the check valve and the connection to the header pipe.

### 1.13.1.8 Pump Vault

Pump vault material can be fiberglass reinforced plastic, polyethylene, or concrete.



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### 1.13.1.9 Pump Selection

Select pumps which meet the criteria for the maximum hydraulic grade line and are able to meet the pumping requirements of the structure where installed. The system shall be reviewed

as a whole, and a type or characteristic of a pump shall be selected for the entire system that has sufficient head to operate at the maximum hydraulic grade line. All pumps should have operating curves that do not allow backflow under maximum head conditions.

surcharging any of the services. Design zones within the system with different maximum hydraulic grade lines may be utilized.

### 1.13.1.10 Service Laterals

Design laterals to be no larger than the diameter of the collector main,

### 1.13.1.11 Cleanouts

Install cleanouts at upstream end of collector mains, junctions of mains, changes in main diameter, and every 400 feet.

## 1.14 Reclaim Water Distribution System

### 1.14.1 Minimum Pipe Size

The minimum size of pipe for principal reclaim water mains 4-inch diameter.

Size of reclaim water mains shall be justified by hydraulic analysis. 2-inch water mains will only be permanent dead-ends where future growth is not feasible. The length of 2-inch mains shall be restricted to 500 feet in any one direction.

All reclaim water mains shall be sized after a hydraulic analysis based on flow demands and pressure requirements. The system shall be designed to maintain a minimum pressure of 20 psi at ground level at all points in distribution system under various conditions of flow.

Wide variations in pressure above the minimum requirement of 20 psi may be inherent in the design of a reclaim water distribution system but pressures no greater than 150 psi should be delivered to the customer (unless higher pressures are requested.). Main line



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pressure reducing valves can be used to reduce pressures below 150 psi where feasible. Where water pressures over 150 psi are necessary to the operation of the distribution system, customers must have individual pressure reducing valves.

All assumptions and any flow data used must be clearly documented and submitted with the hydraulic analysis. Hazen-Williams friction factor appropriate for type of pipe being used but in no case greater than 130.

### 1.14.2 Flushing Hydrants

The minimum pipe size to which a flushing hydrant may be connected is a 2 inch.

### 1.14.3 Dead Ends

Dead ends shall be minimized.

Where dead-end mains occur they should be provided with a flushing hydrant for blow-off for flushing purposes. The blow-off shall be at least 2 inches in diameter.

No flushing device shall be directly connected to any sewer nor be subject to flooding or plugging.



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**APPENDIX 1-A**

**DESIGN BASIS FOR WASTEWATER FLOW AND LOADING**

<b>Occupancy Group</b>	<b>Description</b>	<b>Occupancy Load Factor (sq-ft/occupant)</b>	<b>Demand Factor (gal/occupant)</b>
A	Assembly	20	3.25 <sup>1</sup>
B	Business	100	25 <sup>2</sup>
E	Educational	50	4 <sup>1</sup>
F	Factory	500	25 <sup>2</sup>
H	High Hazard	200	7 <sup>1</sup>
I	Institutional	100	7 <sup>1</sup>
M	Mercantile	30	7 <sup>1</sup>
S	Storage Group	300	25 <sup>1</sup>
U	Utility	300	12.5 <sup>1</sup>

<b>Occupancy Group</b>	<b>Description</b>	<b>Demand Per Unit (gal/unit)<sup>1</sup></b>
R	Single Family House	350
R2	Residential 2	100
R2B1	Residential 2 (1 Bedroom Apartment)	250
R2B2	Residential 2 (2 Bedroom Apartment)	300
R2B3	Residential 2 (3+ Bedroom Apartment)	350

**1 SFUE= 350 GALLONS PER DAY**

*Example for SFUE & Impact Fee Calculation*

A commercial building was submitted during the initial Plans Review phase by the developer in which the square footage of the two-story building is 20,377 ft<sup>2</sup> per floor. A tap for water has already been created and a sewer tap has not, and a 3” water meter is sized appropriately. The bottom floor is commercial shopping, and the top floor is designated for business offices.



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First determine the following from Appendix A, Comprehensive Fees and Penalties tables for Occupancy Descriptions.

Business:  
Occupancy Load Factor = 100  
Demand Factor = 25

Mercantile:  
Occupancy Load Factor = 30  
Demand Factor = 7

Next calculate the Total demand (TD), and sum both occupancies:

$$TD = \left[ \frac{\text{Bldg. Sq. Ft.}}{\text{OLF}} \right] \times DF$$

$$\text{Business: } TD = \left[ \frac{20,377}{100} \right] \times 25 = 5,094.25$$

$$\text{Mercantile: } TD = \left[ \frac{20,377}{30} \right] \times 7 = 4,754.63$$

Sum the business demand and the mercantile demand to get the Total Demand.

$$\text{Total Demand} = \text{Business TD} + \text{Mercantile TD}$$

$$\text{Total Demand} = 5,094.25 + 4,754.63$$

$$\text{Total Demand} = 9,848.88$$

Determine the SFUE multiple.

$$\text{SFUE} = \frac{9,848.88}{350} = 28.14 \text{ SFUEs}$$



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### *Example 4 for SFUE Impact Fee Calculation*

A 50-unit apartment complex is developing within the City, in which a sewer and water tap has not been made for either utility. A 4" master meter is planned for the service tap off the main distribution line, and there will be one large sewer tap to the main. The building consists of 25 one-bedroom homes and 25 two-bedroom homes. First determine the following from Appendix A, Comprehensive Fees and Penalties tables for Occupancy Descriptions.

Residential (R2B1): 25 units  
Residential (R2B1): 250 gallons/unit

Residential (R2B2): 25 units  
Residential (R2B2): 300 gallons/unit

First, determine the demand from each building occupancy.

Total Demand = R1B1 demand + R2B2 demand

Total Demand = 10 + 12 = 22 gallons/day

Then, determine the total amount of SFUEs.

$$\text{SFUE} = \frac{13,750}{350} = 39.29 \text{ SFUE}$$



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### APPENDIX 1-B SEWER

#### INTEGRITY METRICS

Collection system metrics are included in the Design Criteria for the Review of Sewage Works Construction Plans and Documents since trends for I/I in a system as revealed by integrity metrics are often required in engineering reports (ERs) or preliminary engineering reports (PERs). They are used as justification or rationale to support the need for a project or in the life cycle cost analyses (LCCAs) when expanding a wastewater treatment plant or conveyance systems when compared to sewer rehabilitation to reduce actual flows.

Similar to the adage that “all models are wrong, but some are useful”, every measure for sewer integrity has limitation; each has advantages and disadvantages. Some are more affected by seasonal or annual variations in rainfall; some more by inflow than infiltration and some the opposite; some by significant industrial flows and some by seasonal flows (in tourist areas for instance); almost all can be distorted by significant overflows or releases upstream of the metering point in a system. Rarely does one measurement tell the whole story or one metric provide a completely convincing argument that integrity has been improved. Some metrics can be quickly calculated and others require significant data collection and selection; almost all require insight in interpretation. The following definitions and metrics have proven useful and verifiable by TDEC in analyzing and checking data from Tennessee systems and are commended to the engineering community.

A great deal of time and energy can be spent in an attempt to calculate an “acceptable” amount of I/I or an average amount of I/I per unit length of collection lines or per capita. Examples are:

- Average Daily Dry Weather Flow/person; if greater than 120 gallons per person per day excessive infiltration could be indicated.
- Average Daily Wet Weather Flow/person; if greater than 275 [gpd] then excessive inflow could be indicated
- Average Daily Flows/per inch diameter mile per day (idm): if less than 1500 [gpd/idm] is acceptable and greater than 4000 [gpd/idm] should definitely be “considered for TV inspection program.”

Division analysis suggests that most Tennessee systems exceed these amounts and very few standards are applicable across all systems. There is a point at which a dollar spent on sewer rehabilitation is not justified by the value of the conveyance and treatment of the I/I eliminated. The Division would rather utilities decide to stop sewer rehabilitation when the return of the sewer rehabilitation investment compared to the annual I/I cost is reached rather than attempt to justify I/I by the application of an arbitrary “acceptable” general standard.

Flow measurement devices must be properly sized, installed, maintained, and calibrated whether part of a temporary installation or a permanent installation if the



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data is to be reflective of the actual circumstances.

Definitions:

**ADF:** Average Daily Flow on a weekly, monthly, seasonal, or annual basis equals the arithmetic average of each day's cumulative 24-hour flow recorded at the same time each day. (To be sufficiently accurate for most uses, flows on Saturday and Sunday should be differentiated on the same time basis as weekday flows.) There may be some adjustments similar to those in the ADDWF paragraph below, which apply to ADF calculations.

**ADDWF:** Average Daily Dry Weather Flow or Base Sanitary Flow (BSF) is the seven day ADF during the lowest seven-day cumulative flows over an annual or longer basis. The COF WMD will usually utilize the ADDWF over the most recent five-year period. ADDWF is an important number; it is argued that ADDWF represents baseline flow and every drop above that amount is I/I, which represents unnecessary conveyance and treatment costs or costs for which the utility is not recovering legitimate fees if the system's wastewater utilizes water system metering as the basis for their wastewater billing. There are exceptions for blindly recording the seven-day average low flow that can be expected to occur once in every five years ("7Q5") for the metered flow:

- If there is a significant industrial flow that is not completely uniform, such as a manufacturing plant that operates only five days a week or has extensive downtime with no discharge during a stand-down or holiday period, then the ADDWF may have to be adjusted for those periods of time.
- If there are seasonal periods (e.g., a summer recreational destination), the ADDWF may have to be calculated on a Labor Day to Memorial Day basis and a Memorial Day to Labor Day basis.
- In a high growth scenario, residential or commercial, the "7Q5" may be too long a period and the "7Q5" basis may have to be reduced to an annual calculation (a "7Q1" basis, for instance.)

There may be other mitigating circumstances that need to be considered to accurately reflect a system's base sanitary flow. If precise values are desired, the additional qualifications may be worthwhile; if the annual pattern is consistent and trends alone are the important purpose, the additional calculations may not be worth the effort. However, the ADDWF or BSF is calculated, it must be consistently applied to develop representative trends.

### Annual I/I Rate, Volume, and Cost:

Annual I/I Rate (aQ-I/I) [gpd] = ADF[gpd] – ADDWF [gpd]

Annual I/I Volume (aV-I/I) [gal per annum] = (aQ-I/I) [gpd] x 365 [days/year]

Annual O&M Cost I/I (a\$-I/I) [\$ per annum] = (aV-I/I)[gal per annum] x (Treatment & Conveyance O&M Cost [\$ /1000 gallons]) x 1000 [gallons]

- Limitations: The same considerations outlined above under ADDWF or BSF



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above apply. In addition, all the above calculations are subject to annual variations in rainfall and ambient groundwater conditions.

- Alternative analyses: The most common alternatives are created by dividing Annual I/I Rate, Volume or O&M Cost by unit quantities descriptive of the system such as the number of service connections (taps), length of collection system lines or inch-diameter-miles of collection system lines. This forms values such as Annual I/I Rate per customer ( $aQ\text{-I/I} / \text{customer}$  [gpd/customer]) or Annual I/I Volume/miles of collection lines ( $aV\text{-I/I/mile}$  [gal per annum per mile]) or Annual O&M Cost I/I per inch-diameter mile ( $a\$ \text{-I/I/inch-diameter mile}$  [ $a\$ \text{-I/I/yr idm}$ ]). In a growing community, these system descriptors in the diameter could change from year to year.

### Peaking Factors:

Peaking Factors (PF) must be described by a specific:

- location (for instance: wastewater treatment plant (WWTP) influent flow meter, sewer lift station influent, manhole influent flow, etc., usually chosen on the basis of available information and purpose of the monitoring);
- denominator (usually ADF or ADDWF);
- periodicity (usually annual, seasonal basis); and/or
- reference event(s) (graphically displayed versus incident rainfall, average after significant rainfall events, maximum after reference, or maximum rainfall event during specified period, PF at reference event such as two-year 24-hour storm determined from plot of all significant events).

Usually, the description is chosen to demonstrate improvement in a specific area of the collection system.

- Limitations: These metrics are susceptible to the same considerations as ADF and ADDWF above, as well as seasonal rainfall variations between different periods of evaluation. If this is to be used for trending the projected PF at a specific reference rain event is the most valuable for trend analysis.

### Graphical Running Average:

One way to reduce the impact on I/I metrics due to variations in weather patterns from year-to-year or between different periods of evaluation is to plot the 360-day running average of incident precipitation and flow for a specific collection system area or at the WWTP influent flow meter. This is best performed over multiple years to identify overall trends. Progress is indicated by the differential slope and spacing between the two curves, one representing I/I potential and the other the response of the system to that precipitation trend. It has the benefit of capturing reductions in base flow as well as rainfall derived I/I.

- Limitations: Flows that do not reach the flow measurement device are not accounted for. It is not particularly helpful over short periods of time.



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### **Volume of Rainfall-Induced I/I versus Incident Rainfall Events:**

During wet weather conditions, the volume of I/I added to a collection system is proportional to the incident rainfall, if I/I can be conveyed to the flow measuring device. This calculation is a good indication of sewer integrity, especially during the period of higher water table and major rain events. Because of this linear relationship, the average of qualifying rain events and their flow response can give a proportion of volume of I/I per quantity of incident rainfall. This calculation is relatively independent of variations in rainfall as long as sufficient qualifying rainfall events occur during an evaluation period. A Municipal Annual I/I and RDI/I Analysis Spreadsheet can be provided with certain limitations on its use. The method provides guidelines on selecting qualifying rainfall events and the interpretation of the linear regression results. The program also provides flow-versus-time and assists in the determination of low flow periods, ADDWF, ADFs, PF, as well as the relative cost of I/I and sewer rehabilitation projects to reduce I/I impact.

- Limitation: The regression analysis is dependent on the appropriate selection of qualifying rainfall events; it depends on daily flow values, so Saturday and Sunday flows must be differentiated; and it predominately uses rainfall-derived I/I during the wet six to eight months of the year. BSF or ADDWF reduction is not captured by the regression calculation.



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