STORM DRAIN MASTER PLAN

CITY OF MANTECA
2013

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STORM DRAIN MASTER PLAN

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CHAPTER 1 OVERVIEW

Of the many services provided by the City of Manteca (City), the City strives to provide adequate control of stormwater to protect residents and businesses from flooding while meeting all stormwater regulatory requirements. The City accomplishes this essential service by 1) providing and maintaining a system of storm drains, detention basins and pumping facilities; 2) by monitoring and controlling the operations of the storm drainage system, and 3) by enforcing storm drainage regulations established by the Environmental Protection Agency (EPA) and the State of California.

A comprehensive Storm Drain Master Plan (SDMP) supports the essential service of storm drainage control, disposal and regulatory compliance by assessing the condition of existing drainage facilities and by identifying additional facilities needed to accommodate runoff from future development.

The backbone of the SDMP is the City's dynamic computer model of its storm drainage system. The model was formulated as an XP-SWMM model, a model originally developed by the EPA, and the present version was advanced by a private sector firm, XP Software, Inc. A dynamic model allows analysis over time and provides the ability to maximize the efficiency of detention basin and pump operation along with ability to monitor and control downstream water levels to minimize flooding problems with a minimum of new capital improvements.

SDMP Organization

- Chapter 2 – *Policies* presents a summary of policies for the existing and future storm drainage systems.
- Chapter 3 – *Planning & Design Criteria* presents criteria used to evaluate existing drainage facilities and to plan for future drainage facilities. This criterion is also intended to serve as a guide for the design of on-site drainage systems.
- Chapter 4 – *Existing Conditions* evaluates the existing storm drainage system, identifies existing deficiencies and identifies needed drainage improvements.
- Chapter 5 – *Buildout Conditions* provides the analysis results from future growth modeling scenarios, and identifies the capital improvement projects necessary to accommodate new development in the City's 2023 General Plan Boundary.
- Chapter 6 – *Capital Improvement Program* provides cost estimates for the various drainage capital improvements and prioritizes the improvements in terms of greatest need.

CHAPTER 2 POLICIES

The policies outlined in this section were used to develop the 2013 Storm Drain Master Plan (2013 SDMP), and they provide guidance for the planning and design of all site-specific drainage projects within the City.

2.1 Stormwater Disposal Methodology

The South San Joaquin Irrigation District (SSJID) owns a complex network of irrigation Laterals and Drains that run throughout the City's limits. These facilities deliver irrigation water to various farming operations, and they convey excess irrigation water and field runoff to downstream receiving waters, specifically the San Joaquin River. The City relies on SSJID's facilities to convey its stormwater runoff to the San Joaquin River, and will continue to do so through buildout of the City's General Plan.

2.2 City / SSJID Master Drainage Agreement

The City and SSJID have a long-standing agreement that authorizes the City to discharge its stormwater runoff into SSJID facilities for ultimate disposal to the San Joaquin River. In 1975 the City first entered into a storm drainage agreement with SSJID, and in 2006 the City renewed its drainage agreement with SSJID. See Appendix A1 for a copy of the City / SSJID Master Drainage Agreement. Of the many requirements in the 2006 Agreement, the two most significant new requirements are:

1. All stormwater discharges into SSJID facilities must be monitored and controlled such that the capacity of SSJID's facilities is not exceeded.
2. Stormwater quality must be controlled such that it complies with all applicable laws.

The City meets the first requirement by requiring all new development to attenuate its runoff in a storage facility before pumping it into SSJID's facilities. In addition, the City uses real-time water level monitoring stations at critical low points in the conveyance system complete with SCADA (Supervisory Control and Data Acquisition) facilities. Regarding the water quality requirement, the City is classified as a Phase II city by the State Water Resources Control Board. As such, the City, and consequently new development, is required to comply with the State Board's stormwater NPDES permit for Phase II cities.

2.3 Direct Discharge Area

The Direct Discharge Area (DDA) is an area located primarily in the older areas of the City where most of the storm runoff is not attenuated, but instead flows directly to SSJID facilities without any storage or control. The DDA is largely built out with just small-parcel, in-fill areas remaining. See Figure 2-1 for a depiction of the DDA.

Per the City / SSJID Master Drainage Agreement, SSJID now prohibits the direct discharge of stormwater runoff to its facilities. The existing, non-attenuated storm runoff flows are exempt from this requirement, but new development in the DDA is required to install attenuation facilities with positive control for any new storm runoff. There are, however, exceptions to this requirement as specified in Chapter 3.

2.4 Attenuate Peak Stormwater Runoff Flows

Per the City / SSJID Master Drainage Agreement, SSJID prohibits the direct discharge of stormwater runoff to its facilities. Accordingly, the City requires all new development to attenuate its runoff in a storage facility before pumping it into SSJID's facilities. For surface attenuation facilities, the three allowable basin types are described below.

(1) Permanent Detention Basin: Residential Applications

Permanent detention basins in Residential areas are to be sized to hold a 10-year, 48-hour duration storm runoff volume resulting from 3.56 inches of rainfall occurring over the entire contributing area. All basins shall have positive shut-off control. Regional detention basins designed to serve several developments are preferred over smaller, individual basins. Water quality treatment is to be included in detention storage basins to meet stormwater NPDES permit requirements. Detention basins are to be designed as multi-purpose facilities when practical. The volume of detention basins shall be determined with no allowance for percolation or outlet facilities. The full design criteria for permanent detention basins are summarized in Chapter 3.

(2) Interim Percolation Basin: Residential and Non-Residential Applications

Percolation basins may be used as an interim measure for retention and disposal of stormwater runoff in those areas that will not receive storm drainage service from a major storm drain system by the time development occurs. When discharge capability to a major storm drain system becomes available, the basins are to be exchanged for or converted to detention basins with pumped discharge facilities. Interim percolation basins are to be sized to store two (2), 10-year, 48-hour duration storm runoff volumes over the entire contributing area. The full design criteria for interim percolation basins are summarized in Chapter 3.

(3) Permanent Sole Use Percolation Basin: Non-Residential Applications

Sole use percolation basins may be allowed as a permanent measure for retention and disposal of stormwater runoff for large (minimum 80 acres) non-residential applications as approved by the City. Permanent percolation basins shall be designed to empty the entire runoff volume of a 10-year, 48-hour duration storm within 96 hours. The permanent percolation basin shall be oversized by 50%.

The groundwater separation distance between the bottom of the percolation basin and the seasonally high ground water table shall be 10 feet to reduce the risk of groundwater contamination. The City may allow an exception to this separation requirement for situations where the groundwater level is less than 10 feet but greater than 5 fee provided that the runoff is fully treated upstream of the percolation basin by pretreatment BMPs such as a regional CDS unit or other City-approved equivalent treatment device. For situations where the groundwater separation is less than 5 feet, percolation basins are prohibited.

The percolation basins require a minimum soil infiltration rate of 0.5 inch per hour, not exceeding 3 inch per hour.

The percolation basins should not be used as temporary sediment traps during construction.

The percolation basins are not allowed at sites with Hydrologic Soil Type C and D, e.g., clay.

As part of the percolation basin approval process, the developer shall prepare an operations and maintenance plan for the facility documenting all maintenance and rehabilitation activities and costs for approval by the City.

Access to percolation basin shall be provided for regular maintenance activities.

Prior to acceptance of the Basin, the property owner shall provide an approved fully funded mechanism or entity for funding all maintenance and rehabilitation needs of the basin. The maintenance entity shall be permanent and responsible for meeting all stormwater quality requirements of the State Water Resources Control Board.

The full design criteria for permanent percolation basins are summarized in Chapter 3.

2.5 96-hour Detention Basin Pump Station Discharge Rate

Previous master plans required detention basin pump stations be sized to empty the basin within 48 hours after a rain event. The City is relaxing this requirement such that pump stations may empty the detention basin within 96 hours. This pumping rate change reduces the size of local pumping facilities, and significantly reduces the size and scope of downstream infrastructure needed to convey stormwater to SSJID drainage facilities. This rate change also alleviates capacity issues within SSJID Laterals and the French Camp Outlet Canal (FCOC) as described in Chapters 4 and 5. Chapter 3 provides addition detail on the 96-hour discharge rate.

2.6 Expanded Dual-Use of SSJID Laterals for Stormwater Conveyance

As previously described, SSJID owns a complex network of irrigation Laterals and Drains that run throughout the City's limits. The hydraulic connectivity of SSJID's system is as follows: 1) irrigation water is conveyed to farming operations via a vast network of Laterals; 2) the Laterals carry excess irrigation water and field runoff to several Drains; 3) the Drains convey water to a large central drain called the French Camp Outlet Canal (FCOC); and 4) the FCOC conveys water to the San Joaquin River.

A fundamental goal of previous storm drain master plans was to minimize the use of SSJID's Laterals for conveyance of stormwater runoff to SSJID's Drains. Accordingly, previous master plans specified the construction of a separate stormwater conveyance network that by-passed the Laterals and transported storm drainage directly to the Drains. Once the City's stormwater reached the Drains, the Drains would continue to provide conveyance to the FCOC and to the San Joaquin River. In the 2013 SDMP, however, the City recognizes the opportunity to minimize infrastructure costs for all parties by abandoning the concept of separate conveyance systems and instead expanding the use of SSJID's Laterals. Laterals that are targeted to convey both stormwater and irrigation water to Drains are called dual-use facilities.

This concept is viable since SSJID's Laterals are 1) found virtually everywhere in the existing and undeveloped areas of the City, 2) most of these Laterals are already 42-inch diameter pipe, which is a sufficient size for the City's drainage needs, and 3) SSJID already requires new development projects that disturb their Laterals to remove, realign and replace that infrastructure with an equal or larger diameter pipeline. For situations where the existing Lateral is 36-inch diameter, and SSJID does not require a 42-inch or larger pipe to be installed as a replacement, the City will require a minimum 42-inch diameter pipeline to be installed, and the developer will be reimbursed the upsizing cost for the larger diameter pipe via the Public Facilities Implementation Fee (PFIP) program described in Chapter 6.

Chapter 3 summarizes the criteria necessary to satisfy both the City's and SSJID's requirements for the expanded dual-use of the Laterals for stormwater conveyance.

2.7 Simultaneous Irrigation Flow in SSJID Laterals

SSJID's Laterals are surcharged pipelines that deliver irrigation water to various SSJID customers, while the Drains are gravity-based systems that convey irrigation runoff to the FCOC and the San Joaquin River. In several areas throughout the City the Laterals are already used to convey stormwater runoff to the Drains. Since there is a possibility that the Laterals could be full with irrigation water at the same time that stormwater runoff is being discharged to them, there was a concern that flooding could occur from overcharging. To prevent this flooding potential, SSJID requested that the City design its stormwater systems assuming that the Laterals were constantly conveying 25 cubic feet per second (cfs) of irrigation water, and this SSJID flow rate was reflected in hydraulic calculations in the 2006 SDMP.

At the time the above SSJID flow rate was implemented, most of the City's drainage to the Laterals was uncontrolled, and neither the City nor SSJID had real-time water level information on the Laterals. Thus, the SSJID 25 cfs flow was a conservative safeguard against flooding from lateral overcharging. While the 25 cfs flow assumption was effective given the circumstances, it also artificially reduced available capacity in the Laterals, which created barriers to growth in some areas of the City. The City now has controlled discharge to all SSJID Laterals – except in the Direct Discharge Area as previously described – and the City has real-time water level monitoring stations at critical low points in several of the Laterals and FCOC. Therefore, the need for the SSJID 25 cfs flow assumption has become obsolete, and is consequently not used in the 2013 SDMP.

Similar to the policy change of emptying detention basins in 96 hours instead of 48 hours, the elimination of the 25 cfs flow assumption is very beneficial to future development because it frees up capacity in SSJID's Laterals, and it alleviates capacity issues in the FCOC as described in Chapters 4 and 5.

2.8 Storm Drain Zone 39

There is a region within the City's General Plan Boundary that is outside of SSJID's service area, which is known as Storm Drain Zone 39. Figure 2-2 provides a depiction of Storm Drain Zone 39. Per SSJID policy, stormwater runoff from this out-of-service-area region is prohibited from entering SSJID facilities. As such, the City has devised a separate stormwater disposal strategy for Storm Drain Zone 39. This disposal strategy involves the installation of separate storm drain basins with controlled pumping and monitoring systems per the design criteria in Chapter 3. Stormwater runoff from the developable area within Zone 39 will be conveyed to one regional pump station sized to pump all stormwater from this developable area into Walthall Slough via an outfall and swale.

While SSJID does not provide drainage service to areas outside their service area, an exception was made for the Dutra Estates subdivision project, which is located in the northeast corner of Zone 39. Per the terms of an agreement between the City and SSJID, drainage from the Dutra Estates subdivision project is discharged into SSJID's Drain 8, and SSJID charges the City an annual fee for this service. The City recovers this cost from the Dutra Estates residents via a Benefit Assessment District fee.

Upon completion of the Zone 39 facilities, the City plans to discontinue discharge of this subdivision's drainage into Drain 8 by rerouting Dutra Estates' drainage into the Zone 39 facilities. The existing Benefit Assessment District fee will be used to pay Dutra Estates' share of the Zone 39 facilities, as well as any local piping improvements needed to accomplish the rerouting.

2.9 Stormwater Quality

The City is classified as a Phase II city by the State Water Resources Control Board (State Water Board). Accordingly, the City, and consequently new development, is required to comply with the water quality limitations specified in the State Water Board's statewide general stormwater NPDES permit for Phase II cities. All stormwater quality and treatment criteria in the NPDES permit also apply to the Direct Discharge Area (DDA) described in Section 2.3 above.

To achieve water quality compliance, the City encourages developers and engineers to use the water quality treatment principles in the California Stormwater Quality Association (CASQA) Best Management Practice Handbook in the design of stormwater facilities.

2.10 Low Impact Development (LID) Recommendations

LID involves the principles and techniques used to design and construct sites that disturb only the smallest area necessary; minimize soil compaction and imperviousness; preserve natural drainages, vegetation, and buffer zones; and utilize onsite, lot-sized storm water treatment techniques. LID sites reduce and compensate for development's impact(s) on hydrology and water quality.

LID is considered as "Best Management Practice" (BMP), and will result in improved water quality and reduced peak discharges to SSJID's laterals. It is also a design tool that will help meet the New Development and Redevelopment of the City's Stormwater NPDES Permit, and is recommended in the Manteca General Plan.

The objectives of LID are as follows:

(1) Minimize increases in storm water runoff from any development projects in order to reduce flooding, siltation, increases in stream temperature, and streambank erosion and maintain the integrity of stream channels;

(2) Minimize increases in nonpoint source pollution caused by storm water runoff from development which would otherwise degrade local water quality;

(3) Minimize the total annual volume of surface water runoff which flows from any specific site during and following development projects to not exceed the pre-development hydrologic regime to the maximum extent practicable;

(4) Reduce storm water runoff rates and volumes, soil erosion and nonpoint source pollution, wherever possible, through storm water management controls and ensure that these management controls are properly maintained and pose no threat to public safety.

(5) Meet the requirements of the State Water Board's statewide general stormwater NPDES permit for Phase II cities.

While the use of LID is only recommended at this time, the City anticipates that pending updates to the statewide stormwater NPDES Phase II permit will eventually require the use of LID principles in all new development.

2.11 General Stormwater Recommendations

- Where feasible, drainage facilities should be utilized to accomplish open space requirements of the Manteca General Plan and the San Joaquin County Multi-species Habitat Conservation and Open Space Plan, and to provide opportunities for public education.
- Design of storm drain systems within new developments should consider the use of open drainage corridors where feasible as an alternative to an underground pipe drainage system. Open drainage corridors may provide short-term detention storage and storm water quality treatment. The use of open drainage corridor is subject to approval by SSJID and City Council.
- Where feasible, drainage channels should be integrated into the City's bicycle and pedestrian trail system.
- Buffers, grassy swales and landscaped areas may all be used to break the hydraulic connectivity of runoff across paved surfaces and reduce downstream peak flows.
- Where feasible, drainage channels should be designed as multi-use facilities, incorporating a "natural" appearance, providing habitat, open space and a range of mixed use.
- Where feasible, drainage facilities should incorporate interpretive and educational trails.

CHAPTER 3 PLANNING AND DESIGN CRITERIA

The Planning and Design Criteria presented in this chapter are guidelines and minimum standards for the design of storm drainage conveyance, attenuation facilities and drainage pump stations within the City. To prepare this chapter, the City relied on its current Drainage Design Criteria, as well as the South San Joaquin Irrigation District (SSJID)'s requirements. In addition, the City relied on information from the San Joaquin County's Improvement Standards and the County's Hydrology Manual. Moreover, the guidelines and standards presented in this chapter were also derived from design practices used by other cities and counties in the region and from the California Stormwater Quality Association Stormwater (CASQA) Best Management Practice Handbook.

The City acknowledges that these planning and design standards are not all encompassing for every situation, and the City anticipates that there will be site-specific issues that will require unique solutions not specifically covered by this chapter. This information is intended to be used as design guidelines, and the appropriate review of this information and its application to specific situations is the sole responsibility of the design engineer.

3.1 Direct Discharge Area Design Criteria

The Direct Discharge Area (DDA) is an area located primarily in the older areas of the City where most of the storm runoff is not attenuated, but instead flows directly to SSJID facilities without any storage or control. The DDA is largely built out with just small-parcel, in-fill areas remaining. See Figure 2-1 for a depiction of the DDA.

Per the City / SSJID Master Drainage Agreement, SSJID now prohibits the direct discharge of stormwater runoff to its facilities. SSJID considers the City's existing, non-attenuated storm runoff flows as the City's base flow and is consequently exempt from the attenuation requirement. New development in the DDA is required to install attenuation systems with positive control for any new storm runoff. However, it is possible for new development projects within the DDA to directly discharge to SSJID facilities without attenuation provided that certain criteria are met as specified below.

<u>No Attenuation Required if CA < 0.75 or Mitigated CA < 0.75</u>

C is the composite runoff coefficient and A is the area of the parcel in acres. For projects where the composite runoff coefficient times the area of the parcel (CA) is equal to or less than 0.75, the runoff will be considered part of the City base flow (i.e. exempt from the attenuation requirement), and no attenuation or control system will be required. Alternatively, if a project can demonstrate to the City's satisfaction that CA is effectively equal to or less than 0.75 through the use of LID principles, the runoff will be considered part of the City base flow and therefore the project will not require an attenuation or a control system. Any project in the

DDA where the final CA (mitigated or not) is greater than 0.75 shall install attenuation and control systems.

DDA Infiltration System Design Criteria

For projects in the DDA that will implement LID principles to achieve CA < 0.75, the following infiltration system design criteria apply:

Infiltration systems shall not be installed unless approved by the Director of Public Works. Infiltration systems shall not be installed where the bottom of the infiltration system is less than 10 feet above the groundwater, or where dewatering systems are used to lower the groundwater level. Infiltration systems shall be installed in the property they are serving, with the exception of residential infiltration systems which may be linked together. Infiltration systems shall not receive discharge from other properties or the following water shed types:

- Parking lots, roadways, driveways, or any other areas intended for vehicle access or parking.
- Industrial or commercial sites where hazardous spills may occur or where a high risk of pollutants exists.
- City right-of-way.

Commercial property owners need to be aware that a change in use to a high pollutant risk business (outdoor material handling and storage, nurseries, garden centers, etc.) may require abandonment of the infiltration system and installation of a detention system. For this reason it is advised to only install infiltration systems where business use will not be changed to a higher risk business.

Runoff Coefficient Reduction

When determining the composite runoff coefficient for the total project site, the runoff coefficient for the area served by an infiltration system meeting these requirements may be reduced to 0.10 upon review and approval by the City.

Infiltration System Design

Infiltration systems shall be designed according to the following criteria:

- The capacity of the system shall be designed to store the runoff from one (1), 10-year, 48-hour storm, utilizing the volume formula for detention basins. Surface type runoff coefficients shall be used. The volume of the storage system may include collection conduits. The volume of the system shall be calculated with no allowance for infiltration.

- The high water level in the storage system shall be a minimum of one (1) foot below the lowest grade elevation of the property served.

- The area draining to the infiltration system shall be graded such that ponding will not exceed six (6) inches before overland flow to the City storm drain system will occur. Building pads shall be a minimum of 12" above top of curb. In no area shall ponding be less than six (6) inches below the finished floor level before overland flow occurs.

- The infiltration surface of the system, defined as the area within the boundaries of the maximum water surface, must be able to infiltrate the design volume within 96 hours.

- The infiltration rate shall be determined by conducting infiltration tests per the Table 3-1 below. The average infiltration rate shall be calculated from the test results as described below. The infiltration rate as recommended in the soils report shall be used. Where multiple systems are being designed in a single parcel, the aggregate area of all systems shall be used to determine the number of tests required. Where multiple small parcels (less than 14,520 sq. ft. each) are being developed as one project, as in a residential subdivision, one test per parcel will be acceptable provided there are a minimum of 2 adjacent parcels that will be used for the average infiltration rate.

Table 3-1 infiltration test

Minimum	Maximum	Requirement
0 Sq. Ft.	21,780 Sq. Ft.	2 tests
21,781 Sq. Ft.	43, 560 Sq. Ft.	3 tests
43,561 Sq. Ft.	or greater	3 tests per acre

- Open drain inlets shall not be allowed. Rainwater shall enter the storage system through infiltration, or through filtered inlets. Drain inlets other than landscaped infiltration areas shall be marked "Infiltration System – No Dumping". Filters shall be marked "WARNING – failure to filter runoff may result in damage to the Infiltration System". The City of Manteca will maintain a list of approved inlet manufacturers.

- Minimum ten (10) foot setback from single story structures or as required by a structural or geotechnical engineer. Minimum setbacks from multi story structures shall be determined by a structural or geotechnical engineer.

- Minimum ten (10) foot setback from property lines, except for residential projects where the infiltration system is located within the front yard building setback area, provided that a ten (10) foot setback is maintained from all structures.

- Minimum one hundred and fifty (150) foot setback from any drinking water supply well.

- Maintenance of the infiltration system and inlets are the responsibility of the owner or developer. An agreement is required between the owner or developer and the City which gives the City the authority to inspect the system, and to cause repairs or maintenance to be done on the system, in the event that the owner or developer does not complete said work at the City's request. The owner or developer will be responsible for the cost of any such work.

- Piping systems from the inlets to the infiltration storage area shall be a minimum of four (4) inches in diameter for residential and eight (8) inches in diameter for commercial systems, and shall have cleanouts installed meeting the requirements of the Uniform Plumbing Code.

- One or more simple observation wells shall be installed in each storage system. The observation well may be made of PVC perforated pipe, minimum 4 inch diameter, extending to within 4 inches of the lowest point in the storage system and shall have a locking cover. The cover shall be labeled "Infiltration System Monitoring Well". (See attached detail sheet)

- Both personnel and vehicular access to the infiltration system is required for maintenance purposes.

- Where a commercial kitchen hood exhausts onto a roof, the roof drainage shall be treated with an approved treatment system to remove oil and grease before entering the infiltration system. (See City approved list of devices)

- The infiltration system shall not be put into use until the drainage area contributing to it has been stabilized with landscape, or other means, to prevent sediment from being washed into it.

<u>Infiltration System Performance</u>

Infiltration systems may be checked in order to verify that the system is working. If it is found that water levels in the system are not dropping at a sufficient rate to infiltrate the volume in 96 hours the system shall be rehabilitated.

<u>Infiltration System Operation, Maintenance and Rehabilitation</u>

The design of the infiltration system shall include provisions for operation, maintenance and rehabilitation. An Operation and Maintenance manual and a Rehabilitation plan shall be submitted to the City for approval, and the owner shall enter into a maintenance and rehabilitation agreement with the City of Manteca.

<u>Infiltration Test Standard For Underground Systems</u>

These standards are a modification of the Falling Head Percolation Test Procedure, USEPA, Onsite Wastewater and Disposal Systems, 1980.

Step A. Initial Screening

Initial screening identifies the potential for using infiltration methods at a site and identifies the potential location of the site for infiltration devices. The purpose of the initial screening is to determine if installation of an infiltration system is feasible on the site and to determine where field work may be needed for subsequent field verification. The initial screening shall determine the following:

1. Relevant land use information for the site and adjacent parcels.

2. Property boundary lines and setback distances.

3. Existing and proposed buildings and setback distances.

4. Presence of areas with potential vulnerable groundwater.

5. Drinking water well locations where within 150 feet and setback requirements.

6. Historical depth to groundwater (this will be verified in Step B).

7. Presence of flood plain.

8. Presence of soil and/or groundwater contamination.

Step B. Infiltration Testing

Before any infiltration test is made, the water table elevation shall be determined, and a bore log with soil types shall be created.

Each test shall be conducted according to the following steps:

1. Each test shall be made in a pit terminating a minimum of ten feet above the water table and in undisturbed soil. The bottom of the pit shall be at the proposed bottom elevation of the infiltration system.

2. The diameter of the bore pit shall be six (6) inches, dug or bored. Two (2) inches of gravel shall be placed in the bottom of the hole to protect the bottom from scouring action when water is added.

3. The test hole shall be refilled at least twice after the water surface elevation drops to one foot above the bottom of the pit and then allowed to soak overnight. Infiltration rate measurements shall be taken on the day following the saturation process.

4. Water level readings shall be taken at 30-minute intervals beginning at five feet above the bottom of the test hole to one foot above the bottom before refilling. Design shall be based on rates taken at three feet from the bottom, and the test shall be repeated until successive rates do not vary more than twenty percent. The slowest rate measured within ±6" of the three foot level shall be considered as the rate of infiltration for that test hole.

Step C. Design of the Infiltration System

1. The infiltration system shall be designed to meet the above Standards.

Step D. Report and Approval

1. A report and the proposed infiltration system design calculations and plans shall be submitted to the City of Manteca. The report and plans shall include all of the information that was recorded in steps A through C above.

2. Approval of the infiltration system proposal shall be obtained from the City of Manteca Public Works Department.

Required Qualifications

Individuals performing and completing the above testing shall be a licensed civil or geotechnical engineer acceptable to the City of Manteca.

3.2 96-hour Pump Station Flow Rate

Per Chapter 2, pump stations will be sized to empty a detention basin (or other storage facility) within 96 hours after a 10-year Rain Event. The pump station will be a duplex station with two vertical turbine pumps (one duty and one standby).

3.3 Expanded Dual-Use SSJID Laterals for Stormwater Conveyance

Per Chapter 2, the 2013 SDMP will maximize the use of existing SSJID Laterals to serve as stormwater conveyance pipelines. The design and construction for dual-use laterals are subject to approval by SSJID. Figure 3-1 depicts the existing SSJID Laterals that will serve as dual-use facilities. The City's design requirements for these dual-use facilities are:

(1) Pipelines and manholes for dual-use laterals shall comply with SSJID standards and specifications.

(2) Dual-Use SSJID Laterals shall be re-constructed in compliance with SSJID Standards.

(3) Pipelines and manholes will withstand up to 10 psi pressure.

(4) Minimum pipe diameter is 42 inches.

(5) Pipelines will be constructed at a constant grade where possible.

(6) New development projects will replace the SSJID Laterals on a project-by-project basis to the extent of the individual project's boundaries.

(7) Inverted siphons shall be minimized (a.k.a., depressed sewers) and shall be spaced not less than 0.5 miles apart.

(8) A maintenance manhole shall be provided at every inverted siphon location.

3.4 General Design Criteria

3.4.1 Master Plan

Storm drain design shall conform to the City's current Master Plan and the City's Standard Plans and Specifications. Any deviation from the design criteria requires the approval of the Director of Public Works.

3.4.2 Drainage Studies

At the stage of a project when a tentative map is submitted, a schematic-level storm drainage plan shall be submitted for approval by the Public Works Department. At a minimum, this schematic-level drainage plan shall include: storm tributary area, storm basin capacity, and storm drainage discharge location and discharge rate.

When the first set of improvement plans is submitted for a project or if a vesting tentative map is submitted for a project in lieu of a tentative map, a more detailed drainage master plan shall be submitted for review and approval by the Public Works Department. Under these circumstances, the drainage plan submittal shall include the following:

(1) Topographic map of the drainage shed and adjacent area showing existing and proposed ground elevations and sub-shed areas

(2) Preliminary pipe sizes and typical drainage channel geometry with hydraulic grade lines, inverts proposed ground elevations, and all calculations

(3) Map showing analysis points, proposed street grades, and storm drain facilities

(4) Configuration and elevations of proposed retention basins including a preliminary grading plan

(5) Information on proposed pumps and stage, storage and discharge information for retention basins under design conditions

(6) Requirements for stormwater quality treatment BMPs, and Low Impact Development (LID) applications

(7) Preliminary site plan for each basin and site and equipment layout for the pump station

The proposed drainage plan may be evaluated by the City using its master XP-SWMM model to ensure that the proposed facilities conform to the City's storm drainage master plan. The drainage master plan at this phase of the project shall also include Best Management Practices (BMPs) to address stormwater quality issues.

3.4.3 Geotechnical / Groundwater Information

Geotechnical and groundwater information must be submitted with the drainage study. At a minimum, the following information should be provided.

- Preliminary geotechnical analysis
- Description of groundwater conditions
- Limitations on depth of retention basins

3.4.4 Subsurface Percolation

The use of subsurface percolation of runoff will not be permitted except for DDA.

3.4.5 Phasing of Drainage Facilities

If phasing of drainage facilities is proposed, a phasing plan should be submitted prior to approval of improvement plans or map recordation. Design criteria should provide required protection at each phase. Triggering mechanisms should be clearly identified for constructing subsequent phases of drainage facilities.

3.5 Design Runoff

Criteria for computing design runoff are consistent with established City design criteria and with the San Joaquin County Hydrology Manual. The two methods accepted by the City are discussed below; other methods may be acceptable subject to prior approval by the City Public Works Department.

- Rational method: A basic methodology appropriate for drainage systems upstream of basins.
- SWMM modeling: A dynamic computer model used in the formulation of the master plan and recommended for the sizing of detention basin discharge facilities.

For analysis of on-site storm drain systems that do not include storage or pumping, methodologies such as spreadsheet analysis, StormCAD, HYDRA and Rational Method based solutions may be used. The use of any hydraulic method is subject to approval by the Department of Public Works.

3.5.1 Rational Method

The Rational Method was developed to estimate runoff from small urban and developed areas. The Rational formula relates rainfall intensity, a runoff coefficient and drainage area to the direct peak runoff from the drainage area. The relationship can be expressed as:

$$Q = CIA$$

Where, Q = the runoff rate from a drainage area in cubic feet per second (cfs)

C = a runoff coefficient that represents the ratio of runoff to rainfall

I = the time averaged rainfall intensity corresponding to the time of concentration expressed in inches per hour

A = tributary drainage area in acres

The values of the runoff coefficient and the rainfall intensity are determined based on study of drainage area characteristics. The drainage area is determined by computing the area tributary to the point where flow is to be determined based on a topographic map.

- **Runoff Coefficient, C** – The runoff coefficient is the ratio of the rate of runoff to the rate of rainfall at average rainfall intensity (I).

  The value of the runoff coefficient is dependent on rainfall intensity, drainage area slope, vegetative cover, infiltration and other factors.
  The runoff coefficients that follow are derived from the San Joaquin County Hydrology Manual (Hydrology Manual). The runoff coefficients shown in Table 3-2 are by surface type and they are provided for design and hydraulic analysis purposes.

  The runoff coefficients shown in Table 3-3 are by land use type, and they are provided for planning and hydrology purposes. Also, when Low Impact Development (LID) techniques are used for flow attenuation and water quality, the design engineer can adjust these “C” values.

Table 3-2 Typical Runoff Coefficients for Various Surface Types (For Design)

Surface Type	Runoff Coefficient (C)
Basin	1.00
Pavement	0.95
Roof	0.80
Compacted Earth	0.75
Lawns and Open Area	0.15

Table 3-3 Runoff Coefficients for the City of Manteca (For Planning)

Land Use Type	Runoff Coefficient (C)	Minimum Overland Flow Time (Minutes)
Residential (1)		
Very Low Density Residential (VLDR)	0.30	25
Low Density Residential (LDR)	0.30	25
Medium Density Residential (MDR)	0.50	20
High Density Residential	0.65	15
Business and Commercial (1)		
Business Professional (BP)	0.90	15
Commercial Mixed Use (CMU)		
General Commercial (GC)		
Neighborhood Commercial (NC)		
Industrial (1)		
Heavy Industrial (HI)	0.70	15
Light Industrial (LI)		
Business Industrial Park		
Other (1)		
Parks	0.10	30
Schools	0.25	28
Agriculture	0.30	25
(1) In lieu of using this table, composite coefficients may be determined for land uses within a development by using the typical coefficients in Table 3-2.		

- **Rainfall Intensity** – Rainfall intensity is determined by using the intensity-duration-frequency curves in Figure 3-2. The critical duration of the storm rainfall used to enter the intensity curves is based on the time of concentration of the drainage area under study. The time of concentration, T_c, is defined as the interval of time required for the flow at a given point to reach a peak with uniform rainfall intensity. It is common to define the time of concentration as the time from the beginning of rainfall for runoff from the most remote part of the drainage area to reach the point where flow is to be determined.

The time of concentration is computed as the initial overland flow time (t_I) plus the travel time (t_t) in conveyance facilities such as pipes and channels. Minimum values for this initial time are shown in Table 3-3.

$$\mathbf{T_c = t_I + t_t}$$

- **Drainage Area** – The area of the drainage basin to the point of interest is measured using the capability of AutoCAD with the shed map or manually using a planimeter and a map. The drainage area is expressed in acres.

3.5.2 XP-SWMM Model

An XP-SWMM model can be used in the hydrologic analysis when there is a retention or detention basin or when the designer wants to utilize the benefits of dynamic modeling.

3.5.3 Other Methodologies

Additions to the storm drain system can be analyzed and designed using models such as StormCAD and HYDRA.

3.5.4 Stormwater Runoff Quality

All stormwater projects shall be consistent with the City's NPDES permit and shall be designed using the CASQA Best Management Practices Handbooks for new development and redevelopment or other acceptable design standards.

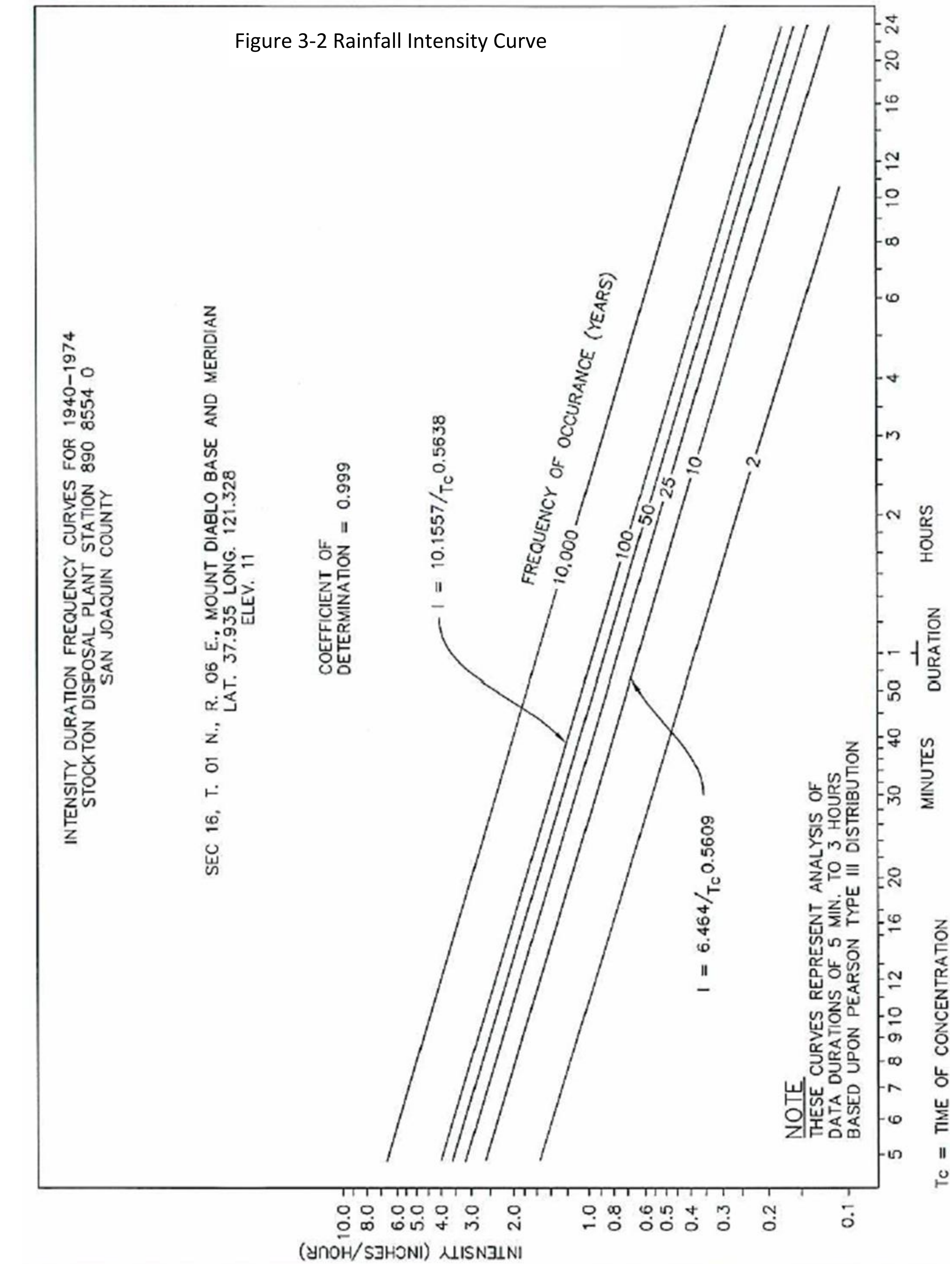


Figure 3-2 Rainfall Intensity Curve

3.6 Design of Conveyance Facilities

This section presents design criteria for stormwater conveyance facilities including:

- Piped storm drains
- Pump discharge lines
- Street flow
- Overland release
- Open drainage channels

Storm drain facilities should be constructed in accordance with the City's Standard Specifications and Standard Plans.

3.6.1 Piped Storm Drains

(1) Size

The minimum pipe diameter of storm drains shall be 12 inches. Laterals that connect catch basins to manholes shall have a minimum diameter of 12 inches.

(2) Pipe Flow

A. All Storm Pipes Upstream of Detention Basins:

- The Hydraulic Grade Line (HGL) shall be a minimum six (6) inches below the elevation of inlet grates and manhole covers for a design storm of 10 years.

B. All Storm Pipes Downstream of Detention Basins:

- Pipes shall be designed to handle pressurized flow that discharge to SSJID Laterals.
- On a case-by-case basis, pipes may be designed as gravity flow systems that discharge directly to the South Drain.

The starting elevation in the basin shall be the result of 2.65 inches of runoff. This criterion is based on a 10-year, 12-hour storm plus one-half of a 10-year, 6-hour storm as being representative of the water surface in the detention basin or interim percolation basin when peak flows occurred in the storm drains.

(3) Roughness Coefficients

Storm drains are primarily reinforced concrete pipe (RCP) with rubber gaskets. Polyvinyl Chloride (PVC) plastic pipe may also be used, especially for on-site drainage and where high groundwater is not a problem. The following design values for the Manning's "n" coefficient shall be used:

- Reinforced concrete pipe (RCP) 0.013
- Polyvinyl Chloride (PVC) 0.013
- Corrugated Polyethylene HDPE (corrugated interior) 0.02
- Corrugated Polyethylene HDPE (smooth interior) 0.012

(4) Minor Losses

Minor losses in storm drains are defined as local losses of energy other than friction losses and include losses at transitions, junctions, manholes and control devices. Minor losses are computed as a percentage of the change in velocity head upstream and downstream of the manhole or other feature.

$$H_L = K (V_1^2/2g - V_2^2/2g)$$

The losses by definition are minor, but accumulated throughout the drainage system can become substantial. The following K values are recommended for common design conditions with pipe velocities about 2.5 feet per second. The design engineer shall modify these recommendations as appropriate.

Table 3-4 Minor Losses in Storm Drain Pipes

Minor Losses	K Values
Expansion Losses: $H_L = K (V_1^2/2g - V_2^2/2g)$	
(1) Increase in one pipe size	0.11
(2) Increase in two pipe sizes	0.18
Manhole Losses: $H_L = K (V^2/2g)$	
(3) Straight throughout, no change in pipe size	0.05
Contraction Losses where:	
(4) $H_L = K (V_1^2/2g - V_2^2/2g)$	0.03

(5) Flow Velocity

The minimum velocity at design flow in pipes shall be 2.5 feet per second (fps) in free flow condition. The maximum velocity at design flow in pipes shall be 10.0 feet per second (fps) in free flow condition.

(6) Storm Drains that Discharge through Drain Levee Embankments

A storm drain that discharges to a Drain through a levee designed to protect adjacent land from flooding shall be constructed in accordance with the criteria of Section 3.8.

(7) Installation

A. Placement

All storm drains shall be placed within rights of way dedicated for public streets unless use of easements is specifically approved by the Department of Public Works. Storm drain lines shall be placed north and east of the centerline of streets in areas of new development. The distance that drain lines are placed from the street centerline varies according to the width of the street right of way and is shown in the Table 3-5.

Table 3-5 Drain Line Placements

Width of Street Right-of-Way (Feet)	Distance from Centerline (Feet)	Width of Median (Feet)
46 & 50	5	N/A
60, 70 & 84	5	N/A
84-110	10	12

B. Cover

Storm drains shall have a minimum cover of 30 inches. Exceptions must be approved by Public Works.

C. Manholes

- Location

Manholes shall be located at junction points, changes in gradient, changes in alignment and changes in conduit size. Manholes shall not be spaced more than 300 feet apart on lines 21 inches or smaller and 500 feet on lines larger than 21 inches without prior approval.

- Types

City Standard Plan D-1 shall be used for drain lines smaller than 33 inches in diameter Saddle manholes shall be used on drain lines 33 inches or larger in diameter. A saddle manhole may be required at intersections of more than three pipes regardless of size. Eccentric cones shall be used on manholes greater than 8 feet in depth. A special structure may be required for confluences of pipelines 42 inches in diameter and larger.

D. Catch Basins

- Location

Catch basins shall be connected by laterals to storm drains at manholes only. Catch basin laterals shall have a minimum diameter of 12 inches and a slope of 0.005 ft/ft to connect the catch basin to a manhole. In certain circumstances, City may allow, on a case-by-case basis, a series of inlet-to-inlet connection. The inlet-to-inlet connection shall be approved by the Director of Public Works.
Taps to the back of curb inlets for private connections shall be allowed only if adjacent property has sufficient grade to be above any street flooding should the drop inlet become blocked. Also, the curb inlet lead to the main drain line must have adequate capacity for the flows from private property. Calculations must be provided to support the use of taps into the back of the drop inlet box.

E. Types
Curb inlets shall be constructed as shown on D-4 and D-5 of the Standard Plans.

F. Access Control
Trash racks at inlets and access control structures are required and shall be located in areas with public access. Structures are to be constructed consistent with City Standards or as approved by the Department of Public Works.

G. Utility Conflicts
Conflicts with existing water lines should be avoided by alignment selections where possible. Where such conflicts cannot be avoided, relocation of the water line will be required. When conflicts between drainage and sewer lines are unavoidable, separation should be obtained by means of an inverted siphon in the drain line. A manhole should be provided to separate pipes when one drain pipe passes through a larger pipe.

H. Private Non-Residential Connections
Parcel sizes of one acre or larger that do not drain to a regional basin are required to drain to a basin prior to connecting to the street drainage system.

I. Design Hydraulic Grade Line
The starting hydraulic grade line in the detention basin shall be the result from a rainfall of 2.65 inches from duration of 15 hours.
This criterion was based on a 10-year 12-hour storm plus a 10-year 6-hour storm as being representative of the water surface in the detention basin or interim percolation basin when peak flows occurred in the storm drains.

3.6.2 Pump Stations

Pump design shall be undertaken with close coordination between the City staff and the design engineer and they are expected to meet prior to initiating design work. City review of design and meetings with the design engineers are required at the 10 percent completion, 50 percent completion and 100 percent completion.

(1) Site Design

The pump station site shall provide adequate space to accommodate the pump structure, support structures, maintenance activities and a minimum 20 feet wide maintenance vehicle access. For pump station sites subject to a 1% annual chance flood within the Base Flood Elevation (BFE), the finished pad area at the pump station site shall be at least two feet above the 100-year flood elevation.

(2) Pump Discharge

Detention basin pump stations shall be designed to discharge the 10-year, 48-hour storm volume from the basin during a period of not less than 96 hours. Pumps will be alternated to balance operating hours.

(3) Standby Pump

Pump stations shall be designed with a minimum of one standby pump

(4) Telemetry

Pumps shall be monitored and controlled by the City telemetry system

(5) Trash Racks

Pump stations shall be designed with trash racks and a sediment dam.

(6) In-line Lift Station

When a storm drain requires a lift station other than for a detention basin, the pump shall be designed for a 10-year storm.

3.6.3 Pump Discharge Lines

(1) Hydraulic Grade Line

The flow line of the pump discharge pipe shall be installed above the 100-year frequency flood elevation or a flap gate shall be installed.

(2) Pipe Material

Discharge pipe shall be steel pipe or High Density Polyethylene Pipe (HDPE). The joints of the steel pipe shall be welded. Joints in the HDPE shall be made by the thermal butt fusion process or other processes where the joints develop the full tensile capability of the pipe.

(3) Pipe Wall Thickness

The minimum pipe wall thickness for steel pipe shall be ¼-inch. Steel pipe shall conform to AWWA C-200. The pipe shall be double wrapped, tape coated, 50 mm minimum thickness. Linings shall be a coal tar epoxy, 16 mm minimum dry film thickness.

(4) HDPE Pipe

HDPE pipe shall be DR 32.5 (51 psi), and shall comply to AWWA C906. Special inspection is required for installation and bedding.

(5) Cover

The minimum cover for the pipeline shall be 24 inches under a roadway and 19 inches where there is no road over the pipe.

(6) Relief Valves

Adequate air and vacuum relief valves shall be installed at the crown of the discharge pipe. Air and vacuum relief valves shall be enclosed in a reinforced concrete vault located outside any road travel way. A galvanized steel cover that is hinged, and can be locked, shall be installed on the vault. The top of the vault shall be extend six (6) inches about the travel way. The vault cover shall be vented.

(7) Outfall

The discharge pipe to an open channel shall exit through a reinforced concrete outfall structure. Adequate stone protection or concrete apron shall be provided as needed to protect ground slope from erosion.

(8) Flap Gate

The discharge pipe shall be flap gated. The flap gate shall be mounted to the concrete outfall structure.

(9) Gravity Discharge

Gravity discharge from basins shall be provided with a positive shut-off valve as required by SSJID.

(10) Erosion Protection

Erosion protection consisting of angular stone, reinforced shotcrete or other types of revetment shall be placed on the adjacent and opposite canal bank. The erosion protection shall be extended at least ten feet upstream and downstream of the outfall structure. Angular stone shall be facing class as specified in Section 72 of the State Standard (CALTRANS) specifications or an approved equal. A suitable geotextile fabric shall be placed beneath the angular stone. The minimum thickness of angular stone shall be 12 inches.

(11) Approval of Drawings by SSJID

Drawings of any proposed crossing of storm drains discharging to or passing under SSJID Drains or Laterals must be submitted for approval by SSJID prior to construction.

3.6.4 Street Flow

(1) New Storm Drain System

The 10-year design HGL in new storm drains shall be at least 6 inches below the adjacent catch basin grate.

(2) Gutters

Minimum gutter slope is 0.0025 ft/ft. Minimum gutter slope around radius corners is 0.005 ft/ft.

(3) Valley Gutters

Valley gutters will not be permitted for street drainage.

(4) Inverted siphons

Inverted siphons will not be permitted for street drainage.

(5) Dry Well

Dry well for disposal of stormwater runoff will not be permitted.

(6) Inlet Spacing and Design Water Spread
Storm drain inlet spacing shall be based on the maximum water spread on the street. Design water spread on the pavement shall not exceed the travel lane adjacent to the gutter.

3.6.5 Open Channels

Open channels are sometimes the most feasible and preferred stormwater conveyance. Two approaches to open channels may be used:

- Open Drainage Corridors are the preferred approach to drainage channel design when feasible and appropriate for the situation. The open drainage corridors are a multi-use and more natural design incorporating the principles of stream restoration with conveyance, habitat, and recreation in a "natural" setting.
- Open Drainage Channels are used with permission and when land availability and other constraints make a drainage corridor infeasible. They are usually unlined and trapezoidal in shape.

The basic open channel criteria are included Section (1) below. Criteria specific to the traditional trapezoidal drainage channel are included in Section (2) below.

(1) Open Drainage Corridors

A. General

When space allows and where feasible and appropriate, open drainage channel corridors are preferred. Open drainage channel corridors provide an opportunity to provide open space, vegetation and habitat while providing recreational opportunities with pedestrian and bicycle trails. Interpretive and educational experiences will also be included in the corridors as possible.

The City supports open drainage corridors where feasible because open corridors provide a more natural solution to the conveyance of stream flows. The open drainage corridors will have aesthetic values and increase environmental benefits. If designed correctly, the corridor will improve stormwater treatment and replace lost riparian and wetland habitat. They are an amenity that sparks environmental awareness of the citizens. And, with good design, they can reduce channel maintenance costs.

Channel storage and water quality treatment within the corridor will help to reduce the land area devoted to detention basins in some areas.

These multi-use or multi-functional drainage corridors require a more thoughtful design process than the traditional flood control channel. The City encourages the effort because of the myriad benefits that will accrue to landowners, residents and the City. Typical sections for drainage corridors are shown in Figure 3-3.

Design of open drainage corridors should also meet the technical criteria for open drainage channels in Section (2) below. Open channels in lieu of piped storm drains are appropriate in situations where stormwater runoff exceeds the capacity of a 72 inch diameter pipe.

Open drainage corridors shall be designed with adequate width to accommodate the channel and parkland, trails, habitat, vegetation and maintenance needs. Open drainage corridors could be up to 200 feet wide.

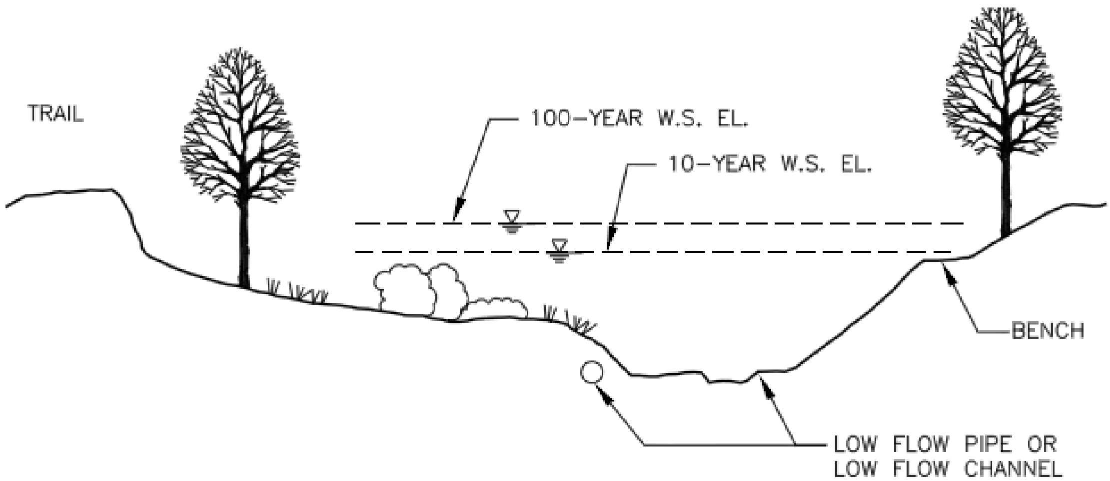
In keeping with the spirit of the open drainage corridors, concrete lined channels will be permitted with special permission only in those areas where there is not sufficient space for an open drainage corridor. In areas of limited right of way, a proposed concrete lined channel requires the approval of the Director of Public Works.

The conveyance portion of a drainage corridor shall be located in excavation so that the design water surface is below ground level. Above ground water surface elevations shall not be used except in special situations when a levee is specifically approved by the Director of Public Works.

B. Design Discharge

Channels shall be designed to safely confine the peak 100-year discharge.

Figure 3-3 Typical Cross Section for Open Drainage Corridor



C. Configuration

Open drainage corridors shall have varying side slopes to emphasize the appearance of a “natural” channel. The preferred configuration is a meandering channel when possible.

D. Hydraulic Design

Open drainage corridors can be simple trapezoidal conveyance channels or more complex meandering streams that are an amenity to the property and the City. The engineer is encouraged to recommend concepts that will provide the best solution for the individual site. Minimum criteria for open drainage corridors are presented for guidance.

Hydraulic design includes the corridor shape and size, slope, depth, roughness coefficients compatible with the design and planned corridor vegetation.

- Shape - Side slopes must be laid back to accommodate vegetation, maintenance and safety. A slope of 3 horizontal to 1 vertical (3:1) is the steepest side slope allowed unless otherwise approved by the Director of Public Works. This slope is usually used with shrubbery and with erosion control measures. Generally, slopes of 8:1 should be used where there will be public access, passive recreation and large turf areas. Benches will be used to break up long slopes and reduce the maximum vertical distances to about four feet. Low flow channels may have steeper side slopes.
  Channel bottoms shall be a minimum of 8 feet wide to provide maintenance vehicle access.

- Slope - Channel slopes will be limited by the elevation difference available within the length of open channel. In general, drainage corridors should have a minimum velocity at design discharge of 2 feet per second. The maximum permissible velocity is 6 feet per second without providing special protection. When high velocities occur in low flow channels, protection should be provided.

- Depth - Maximum water depth shall be 5 feet at design storm (100-year), not including the low flow channel. The total channel depth shall be equal to the depth of water at design discharge plus two feet of freeboard.

- Roughness Coefficient - The hydraulic design of drainage corridors should, to the extent practical, be based on a roughness coefficient, “n” value, of 0.10 or higher, Table 3-6. Designing for a higher “n” will provide the flexibility needed for location and selection of vegetation. The design roughness coefficient is computed as a composite value across the channel section taking into account changes in channel roughness and vegetation.

Table 3-6. Open Drainage Corridor Roughness Coefficients

Channel Vegetation	Roughness Coefficient
Earth	0.035
Tules	0.08
Cottonwood / Willows	0.10
Low growing perennials	0.05
Scrubs	0.08
Grasses	0.03 – 0.05

- Vegetation - Vegetation planted and or allowed to grow in an open drainage corridor shall take into account the objectives of the corridor, the available corridor width, the highest potential “n” value and the adopted maintenance plan. Planting can range from mowed turf to scrubs and trees.

E. Other Design Considerations
- Freeboard - For channels that are located with their design water surface below ground level, one feet of freeboard shall be provided from the water surface to the ground surface.

- Trails - Pedestrian and bicycle trails shall be built into the drainage corridor to the extent feasible. Trails should be constructed to Caltrans Class I bicycle trail standards.

- Retaining Walls - When space is severely limited, retaining walls may be needed to provide a physical corridor boundary. When used, retaining walls shall be located on one side of the open drainage corridor to allow ease of access on the side of the drainage corridor opposite the wall. Corridors should be designed to minimize the need for retailing walls.

- Maintenance Access - A maintenance access road shall be provided along one side of the open drainage corridor. Bike paths may be used as maintenance access roads.

- Fencing - Open drainage corridors will not generally be fenced but will remain open for public access. Low flow, “wet”, areas may be fenced where potential safety concerns exist. Fencing shall be provided along the top of retaining walls.

- Right-of-Way (RW) - Open drainage corridors shall have adequate right of way to accommodate the multi-functions and multi-uses of the corridor including the conveyance section, maintenance road and maintenance activities, plantings and trails.

F. Low Flow

Provision shall be made for conveying low flows in the open drainage corridor by construction of a low flow channel. The low flow channel within the corridor provides water quality and habitat benefits but will cause a continuously “wet” channel throughout the open corridor. A low flow pipe may be allowed for all or a part of the low flow channel flow.

- Low Flow Channel - Two types of low flow channels may be used within the open drainage corridors. The use of low flow channels or pipe should be discussed with City staff and the choice shall be approved by the Director of Public Works. When approved for use, low flow channels may be concrete lined or unlined.

  - Concrete lined low flow channel - A lined low flow channel should be located at the low point of the open drainage corridor. The lined channel shall be capable of conveying low flow, “nuisance” flows. Low flow channels should also have capacity for SSJID flows when applicable.

  - Unlined low flow channel - The channel should have a depth of 1 to 1.5 feet and a width that provides capacity for the nuisance flows. Channels should have bottom widths in the range of two to eight feet. Low flow channels should also have capacity for SSJID flows when applicable.

- Low Flow Pipe - Low flow pipe may be used within an open drainage corridor with the approval of Public Works. When a low flow pipe is proposed, provisions must be made for access and maintenance.

- Bridges - The open drainage corridor can be narrowed and flow transitioned under bridges and road crossings to reduce the length of bridge or culvert structure required. Bridges, culverts and utility crossings shall have a minimum clearance from soffit to water surface of one foot and not cause backwater.

(2) Open Drainage Channels

These criteria apply to major drainage channels and not to roads, ditches or drainage swales.

Unlined trapezoidal channels, while not the preferred drainage conveyance solution, are sometimes required because of space limitations or other reasons. Design of channels will follow the general open channel criteria in Section (1) above supplemented by the criteria in this section.

An alternative of a totally concrete lined channel will be permitted with special permission only especially in areas of limited right of way. Earth channels with concrete bottoms are permitted where drainage channels are approved for use.

A. Design Flow

Drainage channels shall be designed to convey the 100-year peak flow with one foot of freeboard.

B. Side Slopes

Drainage channel side slopes should be 3:1 or flatter. Slopes of 2:1 may be used in special situations with the approval of the Director of Public Works.

C. Levees

Levees will be allowed only in specific situations with the permission of the Director of Public Works. Leveed channels will be designed to convey the 100-year peak flow with three feet of freeboard.

D. Roughness Coefficients

Channel roughness coefficients, Manning's "n", are shown in Table 3-7.

Table 3-7 Channel Roughness

Type of Channel	Design Manning's "n"
Open channel with gunite lining	0.018
Open channel with paved bottom	0.030
Earth channel	0.035
Stream Restoration Channel	0.06 - 0.1

E. Bottom Widths

Channel bottom widths shall be at least eight feet wide, not including the low flow channel, to permit maintenance activities.

F. Curvature

The center line radius shall be a minimum of 35 feet or at least twice the bottom width, whichever is greater.

G. Fencing
Single purpose drainage channels shall be fenced when safety conditions dictate.

H. Access
A 12-foot all weather maintenance road will be required on one side of the channel. The access road may be combined with a bike trail and/or pedestrian path. When an access road will also be used for a bike trail, the road will be constructed to Caltrans standards for Class 1 bike paths. A maintenance road is not required adjacent to a City street.

I. Road Crossings
Crossing of South San Joaquin Irrigation District facilities shall conform to requirements of the District.

3.7 Stormwater Storage Basins

3.7.1 Introduction

Stormwater storage is needed to reduce runoff to meet limitations imposed by SSJID on the use of SSJID's Drains and Laterals and to provide treatment of stormwater runoff. Two primary types of stormwater storage basins are used in the City:

- Detention Basins
- Interim Percolation Basins

A detention basin is designed to detain all or a portion of the volume of a storm thereby reducing the peak outflow. After a period of time, the stored runoff is discharged to downstream drains.
An interim percolation basin is designed to hold the entire volume of a storm with no provision for discharge. Percolation basins are used only as an interim measure until discharge capability to a Drain is available and require the approval of the Public Works Department.

Basins are also used as a best management practice in the treatment of stormwater quality in conformance with NPDES permit requirements. When basins are used as a BMP in conformance with the CASQA handbooks, they are usually constructed in combination with stormwater storage facilities.

3.7.2 General Criteria

The design of stormwater storage basins shall include considerations for the following attributes:

- Sustainable dual and multi-uses including parks, ponds, open space, recreation and nature areas
- Water quality enhancement
- Low maintenance
- Accessibility

3.7.3 Detention Basins

(1) Capacity

The design capacity of a stormwater detention basin shall be determined based on the following relationship.

$$V = \frac{C\ A\ R}{12}$$

Where:

V = The basin volume in acre feet
C = Runoff coefficient for the basin tributary shed
A = Tributary shed area in acres
R = Total rainfall in inches for a design storm
Urbanized shed: 3.56 inches (10-year, 48-hour duration storm)
Rural shed: 2.63 inches (10-year, 24-hour duration storm)

The volume of detention shall be computed with no allowance for percolation or outlet facilities. The maximum water surface elevation in the basin shall be 12 inches below the lowest drain inlet elevation in the tributary shed.

(2) Outflow

Detention basins shall empty, either by gravity or by pumps, over a 96-hour period. Positive control by pumps or valves is required via the City telemetry system.

(3) Basin Side Slopes

Detention basins shall have maximum side slopes of 6 horizontal to 1 vertical with turf and up to 3:1 with other materials.

(4) Bottom Elevation

The bottom elevation of a basin shall be a minimum of two feet above the groundwater elevation except the Director of Public Works may approve less if the basin is lined.

(5) Maximum Depth

The maximum depth of water for an unfenced basin shall be five feet.
The detention basin shall be fenced with a minimum six feet high chain link fence or equal when the maximum depth of water in the basin is greater than five feet. An eighteen foot double gate shall provide access within the fenced area.

(6) Hydraulic Grade Line

The starting hydraulic grade line in the detention basin shall be the result from a rainfall of 2.65 inches from duration of 15 hours as discussed in Chapter 3, Design of Conveyance Facilities.. This criterion was based on a 10-year 12-hour storm plus a 10-year 6-hour storm as being representative of the water surface in the detention basin or interim percolation basin when peak flows occurred in the storm drains.

(7) Lift station

When storm drain requires a lift station other than for a detention basin, the pump shall be designed for a 10-year storm.

(8) Completion

Upon completion of all testing and after streets are paved, the contractor shall clean the overall storm drain system in such a manner as to insure that all foreign matter and debris shall be removed and disposed of in a manner acceptable to the Engineer.

3.7.4 Interim Percolation Basins

Percolation basins are only to be used as an interim facility.
In those areas that will not receive drainage service from a major drain with sufficient capacity for additional retention basin discharge by the time development occurs, percolation basins may be used as an interim measure for retention and disposal of urban storm waters.

When discharge capability to a major drain becomes possible, the basins should be exchanged for or converted to retention basins with discharge facilities. Therefore, percolation basins should be designed to facilitate this transition, as well as in accordance with the following recommended design criteria:

(1) Design Criteria

- The basin should be designed to store the runoff volume from two (2) 10-year, 48-hour storms, using the storage volume formula of Section 3.8.1 (1). The volume of the basin should be calculated with no allowance for percolation or outlet facilities.

- The ground surface of the basin, defined as the area within the boundaries of the maximum water surface, must be able to percolate the design volume within 96 hours.

- The percolation rate should be determined by the procedure described below. At least one percolation test per acre of ground surface, equally spaced, but not less than three borings for any basin shall be conducted. The average percolation rate shall be calculated from the test results as described below.

- The basin should consist of two distinct storage areas, one for low flow runoff and one for high flow runoff. The low flow region should be designed lower than the high flow region such that it receives the first runoff from a storm event.

- The low flow runoff area should be designed to store and percolate the runoff resulting from 0.5-inch of rainfall over the entire drainage area, without overflowing into the high flow runoff area.

- The low flow area should be fenced to restrict public access. Landscaping may be used in lieu of a fence to impede access with the approval of the Director of Public Works.

- Where the distance from the bottom of the basin to groundwater is less than 10 feet, the length of time that a percolation basin is in use shall not exceed 10 years in commercial and industrial areas, and 15 years in residential areas, or, the stormwater shall be treated prior to entering the basin to reduce the possible containment load.

- If at any time the Director of Public Works determines that the stormwater discharge into the percolation basin is a threat to groundwater quality, a treatment system shall be installed upstream of the basin.

- The bottom of the basin should be turf.

- The hydraulic grade line in storm drains upstream of the detention basin resulting from a rainfall of 2.65 inches from duration of 15 hours shall be not less than six inches below any drain inlet in the drainage area as discussed in Chapter 3, Design of Conveyance Facilities. This criterion was based on a 10-year, 12-hour storm plus a 10-year, 6-hour storm as being representative of the water surface in the detention basin or interim percolation basin when peak flows occurred in the storm drains.

- Side slopes are to be designed to maintain stability under saturated conditions, but in no case shall the slopes be steeper than 5:1.

- For basins that are intended to be used as parks, the low flow area shall be fenced or separated by landscaping to restrict public access, and the entire basin shall be turf.

- For basins that are not intended to be used as parks, the entire basin shall be fenced to restrict public access, and the ground surface of the basin, defined as the area within the boundaries of the maximum water surface, shall be seeded with an annual-type ground cover.

- For private basins, maintenance of the basin is the responsibility of the owner. An agreement is required between the developer and the City that gives the City the authority to cause repairs or maintenance to be done on the basin in the event that the developer does not complete said work at the City's request. The developer will be responsible for the cost of any such work.

- Both personnel and vehicular access to the fenced areas is required for maintenance purposes.

- As this basin is temporary until such time as access to a positive drainage system becomes available, the developer shall either construct the pump station and appurtenances at the time of construction of this basin or set up the funding mechanism for future construction.

- When a positive drainage system to serve this basin becomes available, the basin shall be converted to a retention basin with discharge facilities designed in accordance with City Standards.

- If infiltration exceeds 2.4 inches per hour, pretreatment to protect groundwater is required.

- The basin bottom should be scarified by 12 inches (beyond the finish grades indicated on storm basin plans).

- The basin bottom should be deep-ripped at least 24 inches deep at no less than 36 inches O.C.

- Necessary piping and structures shall be installed in the basin bottom.

- An 85% relative compaction of engineered fill should be provided as indicated on subdivision improvement plans.

- Once engineered fill is created no further construction work, other than park improvements may be made in the basin bottom.

(2) Percolation Testing

- The number of percolation tests to be performed should be determined as described above. Before any percolation test is made, the water table elevation shall be determined. Each test shall be conducted according to the following steps:

- Each test shall be made in a one-foot diameter test hole terminating five feet above the water table and in undisturbed soil.

- The test hole shall be refilled at least twice after dropping to five feet above the bottom of the pit and then allowed to soak overnight. Percolation rate measurements shall be taken on the day following the saturation process.

- Water level readings shall be taken at 30-minute intervals from fifteen feet above the bottom to five feet above the bottom before refilling. Design shall be based on rates taken at seven feet from the bottom, and the test shall be repeated until successive rates do not vary more than twenty percent. The slowest rate measured within 6 inches of the seven-foot level shall be considered as the average rate of percolation for that test pit.

3.8 Stormwater Quality

This section addresses criteria for integrating Best Management Practices (BMP) into regional stormwater solutions. Stormwater source control and treatment control measures shall be designed consistent with principles set forth in the California Stormwater Quality Association Stormwater BMP Handbook (CASQA Handbook), January 2009 or later edition.

3.8.1 Dry Extended Basin

The Dry Extended Basin, (CASQA. Handbook TC22), or a variant thereof, is the BMP that appears to have the potential as part of a regional solution in Manteca, Figure 3-4. When this BMP is selected, it is recommended that the water quality storage act in concert with operational detention storage. There are other potential facilities, regional and local, that will provide treatment of stormwater runoff. These should be developed by the design engineer when appropriate in a close working relationship with City staff.

(1) Dry Extended Detention Basin

Dry extended detention basins are likely to be selected for most regional applications. These basins will be designed with an outlet that will detain the water quality volume for 48 hours to allow adequate settling time. Dry extended basins should be designed together with needed stormwater detention basins to have a single coordinated basin that will accomplish the dual objectives.

Basins used as a BMP in the treatment of stormwater quality shall be designed using the methodology and procedures of the CASQA Handbook. Volume-Based BMP design treats 85 percent of annual runoff as presented in the CASQA Handbook. The 48-hour drain down Capture/Treatment Analysis curves for the Fresno Yosemite International Airport included in Appendix D of the CASQA Handbook should be used to confirm the size of the basin.

(2) Multi-functional Basins

Functionally, basins will have water quality function and a stormwater detention function to reduce peak flows. Basins will also function as parks with recreation and trails and will provide habitat, vegetation and open space.

Figure 3-3 shows a plan and cross section of a typical basin. Each basin comprises a low flow area that is frequently wet even during non-storm days, a broad turfed area with flatter side slopes that provides storage during large storm events, and recreation facilities including trails and pathways. These multi-use basins should also follow the criteria for Detention Basins. Depending on the design, some of the lower areas of the basin may have to be fenced or protected from intrusion with a vegetation barrier.

3.8.2 Flow-Based Treatment

Flow based treatment including vegetated swales and open drainage corridors may be part of a stormwater quality plan.

Design of vegetated swales is discussed in the CASQA Handbook as BMP TC-30. When using a vegetated swale as a part of stormwater treatment, the swale should be designed with a minimum travel time of 10 minutes with side slopes of 3:1 or flatter.

Swales as BMPs are designed to be shallow with design depths that do not exceed two thirds of the grass height. Grasses selected should be dense and close growing varieties to maximize treatment.

Other flow based treatments include underground vaults and vortex separators and other proprietary devices. Many of these devices have not been thoroughly tested and the design engineer has the burden of installing facilities that will achieve the City’s quality objectives. There are several wet vault systems available, BMP MP-50. Vortex Separators, BMP MP-51, are a specific configuration of wet vault that uses the centrifugal movement of water to increase the removal of suspended sediment and attached pollutants.

3.8.3 NPDES Permit Attainment Requirements

Manteca’s NPDES permit requires implementation of certain education, public involvement, ordinances, development standards and implementation designed to reduce pollutants from stormwater discharge. Attachment 4 to the permit outlines design standards and the City’s Stormwater Management Program presents a plan through which the City can attain the water quality goals.

The CASQA Handbook provides information on the selection and design of treatment measures. Chapter 13.28 of Title 13 Public Services establishes minimum stormwater management requirements for development in the City. Development applications must be responsive to each of these directives.

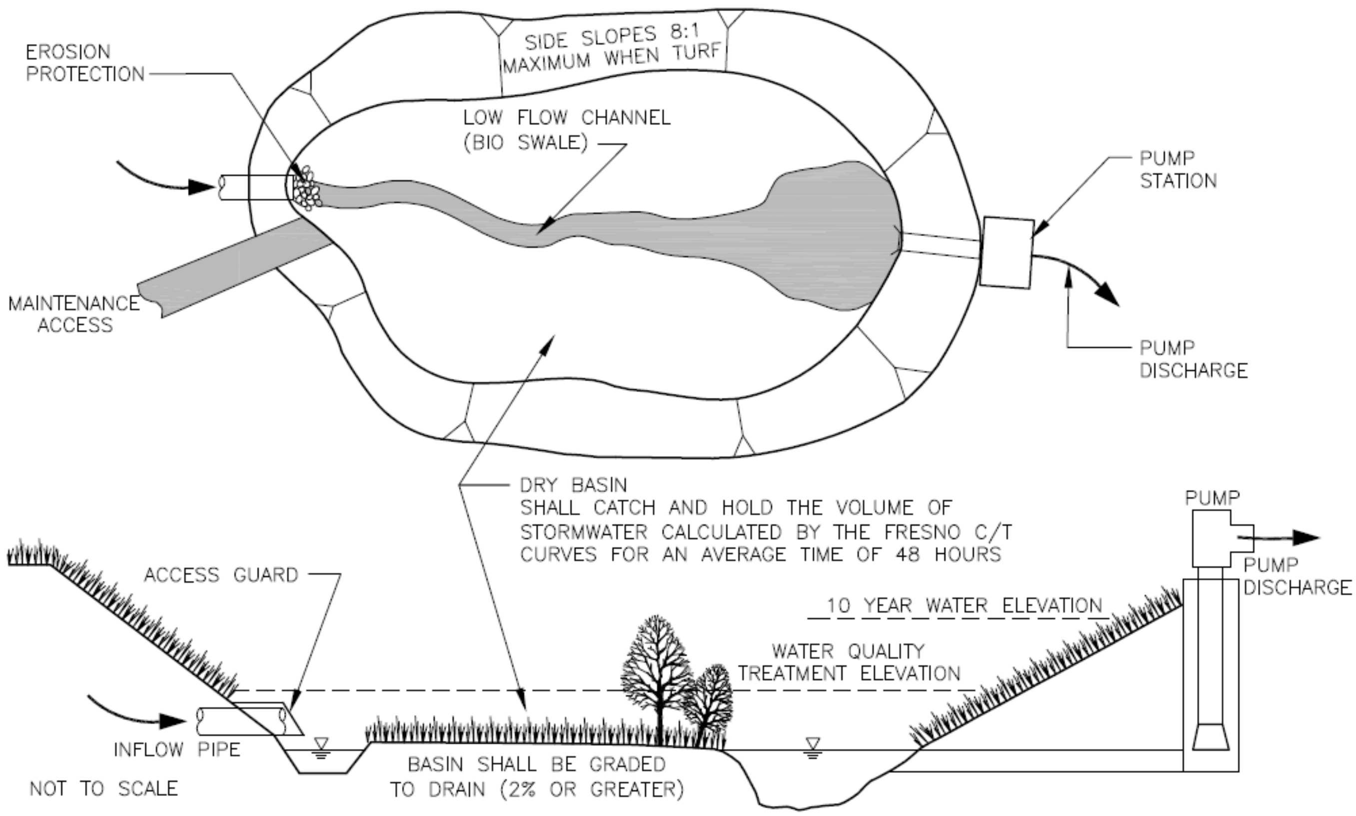


Figure 3-4 Dry Extended Detention Basin

CHAPTER 4 EXISTING CONDITIONS

The purpose of this chapter is to provide a detailed description of the City's existing storm drainage system; to describe how the existing system was evaluated to identify any existing deficiencies; and to identify improvements needed to remedy existing deficiencies.

4.1 2013 Existing Storm Drainage Systems

(1) Detention Basins

SSJID requires that storm drainage flows do not exceed the capacity of their facilities. As such, the City requires detention basins to help satisfy this requirement as they provide storage to attenuate peak flows before drainage flows are pumped into SSJID's facilities. Some basins also delay releasing water for a longer period to further reduce the potential of downstream flooding. Most detention basins are joint-use facilities providing recreation and other uses when not being used for stormwater detention. Figure 4-1 shows the locations of the existing detention basins.

(2) Stormwater Quality Treatment Systems

Stormwater quality standards imposed and monitored by the EPA and the State Water Resources Board through the City's NPDES permit require treatment of stormwater runoff prior to its release into the sloughs, creeks, rivers or the Delta. Treatment is often provided within detention basins in a separate "wet" area that is part of or adjacent to the main basin. Other treatment may be provided by on-site source control and by site specific facilities such as vortex separators. Stormwater quality is an integral part of the City's stormwater management system.

(3) Pump Stations

Most existing stormwater is pumped into the SSJID Laterals and Drains. Pumps are sized according to City design criteria and their operation is controlled by water levels in downstream drains. Figure 4-1 shows the locations of the existing storm drainage pump stations, and Appendix A2 provides a tabular summary of these pump stations.

(4) Water Level Monitoring Stations

There are 10 existing water level monitoring stations throughout the City's storm drainage systems that are used to obtain real-time water level measurements at critical low points in the system to prevent flooding. Figure 4-1 shows the locations of the existing water level monitoring stations, and Appendix A2 provides a tabular summary of these monitoring stations.

(5) Supervisory Control and Data Acquisition (SCADA) System

The City uses a SCADA system to remotely monitor and control the existing storm drainage pump stations and water level monitoring stations.

(6) SSJID Drains and Laterals

The City currently uses several SSJID Drains, Laterals and the French Camp Outlet Canal to convey stormwater runoff to the San Joaquin River. Drains remove irrigation runoff as well as stormwater from irrigated lands and urban runoff; pressurized laterals systems deliver irrigation water and are also used to convey some drainage. The use of Laterals for City drainage has some limitations because capacity must be maintained for irrigation flows at all times of the year and hydraulic grade lines are maintained higher for irrigation water deliveries. The Drains and Laterals currently used for stormwater conveyance are summarized below and also shown on Figure 4-1.

A. French Camp Outlet Canal (FCOC)

The FCOC flows from south to north along the Union Pacific Rail Road tracks from north of Highway 120 to French Camp Slough in Stockton. The FCOC collects irrigation drainage and stormwater runoff from all SSJID Drains and Laterals, and is the backbone of the City storm drain system.

B. Drains

- Drain 3 - conveys runoff from east to west along the Louise Avenue corridor. Drain 3 is a major drain 24,000 feet long serving 2,200 acres.
- Drain 3A - flows north of Drain 3 and serves 88 acres. At the present time, Drain 3A is pumped at Pump Station 15 to Lateral Rf but the master plan will direct its flow to Drain 3N and to Drain 3.
- Drain 3N - is proposed to serve 1,270 newly developing acres in the north of the City. Drain 3N will terminate at its confluence with Drain 3.
- Drain 4 - serves 885 acres of central Manteca
- Drain 5 - and its Center Street tributary, the Drain 5 interceptor, drain 1,822 acres of central Manteca.
- Drain 7 - drains almost 1,600 acres from the Spreckels complex north of Highway 120 to the southern beginning of the FCOC.
- Drain 8 - drains 236 acres south of Highway 120 north to its confluence with Drain 7 and the FCOC.

- South Drain - is planned to serve 8,680 acres of the growing south area of the City including new industrial land in the southeast. Figure 4-2 shows the runoff shed boundaries for each SSJID drain.

C. Laterals

- Lateral Rf - is the most northerly lateral in the City. It currently receives drainage inflow from Pump Station 15 but with the master plan its use for drainage will be eliminated with the construction of Drain 3N.
- Lateral Re - receives City drainage from Pump Stations 3 and 4. The lateral discharges into Drain 3.
- Lateral T - receives drainage inflow from Pump Stations 1, 2, 10, 14, A, B, C and D.
- Lateral Tb - receives less than one cfs of drainage inflow from Pump Station 13. The lateral discharges into Drain 5.
- Lateral Z - receives drainage inflows from Pump stations 9 and 24. Lateral Z flows into Lateral Y
- Lateral Y - receives drainage inflow from Pump Stations 7 and 8. Pump Station 7 is being diverted into Drain 7. Lateral Y discharges into Drain 7 upstream of the FCOC.

4.2 2013 Existing Condition Model

The City has experienced a significant amount of development since the models for the 2006 Storm Drain Master Plan (2006 SDMP) were created. Several new residential developments and commercial/industrial projects have been approved for development or constructed since the adoption of the 2006 SDMP. Because of these changes, the 2006 SDMP Existing Model has been updated to reflect the current conditions of the stormwater system with approved developments as of April 2011. This updated model is referred to as the 2013 Existing Conditions Model (2013 Existing Model).

The 2013 Existing Model uses many of the same assumptions, data and criteria of the 2006 SDMP model, except for the following major criteria differences described below.

1. 96-hour Pump Rate for Detention Basin Pump Stations

   Previous master plans required detention basin pump stations be sized to empty the basin within 48 hours after a rain event. The 2013 SDMP relaxes this requirement such that pump stations may empty the detention basin within 96 hours after a rain event. As such, for each existing storm drainage pump station, the flow rate needed to empty its corresponding detention basin in 96 hours was calculated and input into the XP-SWMM model as each station's pumping rate.

Exception: The existing “Industrial Park Pump” and “Vanderbilt Pump” will keep the 48-hour pump rate since there is no detention basin for each drainage tributary area.

2. No Simultaneous Irrigation Flow in SSJID Laterals (25 cfs Rule)
   The City now has controlled discharge to all SSJID laterals – except in the Direct Discharge Area as previously described – and the City has real-time water level monitoring stations at critical low points in several of SSJID’s Laterals and in the FCOC. As such, the need for the 25 cfs rule has become obsolete, and is consequently not used in 2013 Existing Model.

4.2.1 XP-SWMM Model

Many computer models are available to simulate hydrologic and hydraulic conditions in the City’s storm drainage system. The existing drainage system model was developed in an XP-SWMM model including all existing drainage systems and the FCOC.

The model includes all detention basins/pump stations and the downtown storm drains and open channels to the FCOC. A schematic representation of the existing model is presented in Figure 4-3.

The XP-SWMM software uses the hydraulic data to simulate the storm drains, open channels, detention basins and pump stations, and other facilities. In the model these facilities are represented as links and nodes. The model uses the data to calculate flow rates and resulting water surface elevations throughout the storm drain system.

For the open channels, the modeled ground elevation is the surveyed top of the higher of the left or right bank, and the flooding elevation is the top of the lower of the left or right bank. The channel sections are from the FCOC study or from the storm drain surveys.

For piped storm drains, the ground elevation is usually the manhole rim elevation, but could be the top of the detention basin or other relevant elevation. The flooding elevation was assumed to be 0.3 feet below the rim elevation to account for the typical street crown.

The developments and facilities added to the model to create the 2013 Existing Model are discussed in detail below.

(1) Drain 3

Drains 3 and 3A serve the northern portion of the City. Drain 3 tributary area is the largest and is substantially developed. Recent developments in the Drain 3 tributary area include Union Ranch and Rodoni Estates. The Union Ranch development was approved and is currently under construction. It is a residential/commercial development and is located to the north of Lathrop Road and extends from Airport Way on the west to approximately half a mile beyond Union Road on the east. The project completely encloses and realigns the existing Drain 3 open channel from Lathrop Road to Airport Way with 6-foot by 8-foot and 6-foot by 9-foot rectangular box culverts.

Portions of Union Ranch also discharge to Drain 3A that flows westward and connects into Drain 3. Drain 3A will run through Union Ranch as twin 48-inch pipes and its construction is partially complete. When the remaining portion is completed, Drain 3A, which currently discharges to SSJID Lateral Rff via Pump Station 15, will connect to a 48-inch pipe.

The Rodoni Estates development to the east of Highway 99 was completed. The eastern portion of Drain 3 from Cottage Avenue westward to Felice Way was previously an open channel. The open channel was converted into a 48-inch pipe as a part of the Rodoni Estates construction.

(2) Drain 3A

Drain 3A discharges to SSJID Lateral Rff though Pump Station 15 that is located north of Lathrop Road midway between Union Road and Highway 99.

The Drain 3A drainage area west of Highway 99 is mostly developed. Runoff from this area is conveyed west and north to Lathrop Road. Tributary area to Drain 3A that is east of Highway 99, bounded by Lathrop Road on the north, Southland Avenue on the south and Cottage Avenue on the east, has only small amounts of development and is a potential growth area.

(3) Drain 4

Drain 4 tributary area is mostly built out. The Lincoln Estates, a 3.6 acre residential development that drains to Drain 4, was completed. It is located east of north Lincoln Avenue and north of East Edison Avenue. There have been no recent developments in the area except the Lincoln Estate construction.

(4) Drain 5

Drain 5 and the Drain 5 Interceptor serve a large portion of central Manteca (Figure 4.2). The Drain 5 system is mostly piped. Beginning approximately 2,000 feet west of Union Road, it flows westward as an open channel to the FCOC, a distance of about 6,400 feet. The Drain 5 tributary area is mostly developed except for the large parcels in the most western portion of the watershed.

The City has approved the Villa Ticino West project along Drain 5 west of Airport Way. As a part of this project, the Drain 5 open channel to the west of Airport Way will be converted into an open drainage corridor as defined in the SDMP. In addition, the existing twin 48-inch pipes also to the west of the Airport Way will be replaced with an open channel corridor or any equivalent drainage system.

Modeling analysis of the Drain 5 interceptor shows the potential for minor short term ponding in the area of Center Street, from Center Street at Magnolia Ave to Union Rd at Eucalyptus Street. However, the City has not observed actual ponding in this area after a rain event; thus, no capital improvements are provided at this time. Instead, the City will continue to monitor this facility's performance during rain events, and will recommend capital improvements in future master plans based on observations.

(5) Drain 7

Drain 7 serves the area north of Highway 120 and west of Highway 99. Drain 7 begins on the east side of Spreckels Avenue and continues under Spreckels Avenue, Industrial Park Drive, and Daniel Street and discharges to the FCOC. Its tributary area is substantially developed.

The Spreckels Industrial Forcemain from the Spreckels Basin and Pump Station to the discharge into Drain 7 is complete.

The Big League Dreams development west of Airport Way was constructed. Construction of Stadium Center, a commercial development also west of Airport Way, is completed. The open channel between Airport Way and the FCOC was converted to a rectangular box as a part of the Big League Dreams project. The development includes a detention basin and pump station that discharges to Drain 7 and the FCOC.

The current South Drain area developments drain to Lateral Ya south of Highway 120. Lateral Ya then flows north, crosses Highway 120 and discharges into Drain 7.

(6) Drain 8 and 8A

Drain 8 is largely used to drain newly developing residential areas. Drain 8 is an open channel from Highway 120 north to its confluence with the FCOC. Upstream of Highway 120, Drain 8 has been piped.

Currently, Drain 8 conveys runoff from the Dutra Farms South East subdivision. The Bella Vista Subdivision discharges to Drain 8A, which then connects to Drain 8.

(7) South Drain

The South Drain tributary area is bound by the City limits on the west and south, Highway 120 on the north and extends past Highway 99 on the east. The area also includes land east of Highway 99 to Austin Road between Highway 99 and one half mile north of Yosemite Avenue.

The South area is partially developed and is rapidly growing. The land between Main Street and Highway 99 has substantial residential development. The developments in South Drain include: Woodward Park; Tesoro, Paseo Apartments; Antigua, Ken Hill Estates; and Terra Bella. Runoff from these developments currently drains to Lateral Ya. Runoff in Lateral Ya then flows into Drain 7 via Lateral Yb. However, this is an interim arrangement, and all Lateral Ya flows will ultimately be redirected to the new South Drain pipeline in Woodward Avenue in the future.

The Sundance and Oleander subdivisions between Airport Way and Union Road have been approved for development. Runoff from these developments will discharge to Drain 8 in the interim until the South Drain Pipeline and Pump Station are built. Dutra Estates, located west of Airport Way, is also partially developed and discharges to Drain 8.

Some portions of the South Drain Pipeline in Woodward Avenue have been constructed or are under construction. The South Drain Pipeline from Atherton Drive to Main Street has been completed.

(8) French Camp Outlet Canal (FCOC)

The FCOC begins at the confluence of Drains 7 and 8, approximately 1,000 feet north of Highway 120. The channel follows the Union Pacific Railroad north and terminates at the French Camp Slough in Stockton. The FCOC receives flows from the Drains 3, 4, 5, 7, and 8 and SSJID irrigation laterals. Some areas immediately adjacent to the FCOC are proposed to drain directly to the FCOC. The Pacific Business Park (formerly Assieh) Industrial development is one of them and has been approved by the City. This development is located on the east side of the FCOC and Union Pacific Railroad, north of Louise Avenue.

Improvements made to the FCOC since the 2006 SMDP include the culvert crossings at Louise Avenue and Farm Road. In 2009, the culvert crossing at Louise Avenue was upgraded with two 12-foot by 8-foot box culverts. In 2007, the City replaced the Farm Road culvert crossing, just downstream of the confluence of Drains 7 and 8, with a 66-inch CMP pipe.

4.2.2 XP-SWMM Model Results

Several XP-SWMM model scenarios were run to assess the capabilities of the existing facilities to provide adequate level of drainage and flood control protection. In general, the 96-hour pumping rate policy and the “no 25 cfs” policy had significant positive impacts on the existing drainage system. Virtually all of the flooding and capacity issues identified in the 2006 SDMP are no longer issues because of these policies. However, two flooding issues remain as described below.

(1) Monterey Place

Storm runoff from the Monterey Place area flows directly into Drain 3. This area has experienced frequent flooding problems because of the elevation of the local drain inlets are lower than the crown of the Drain 3. Accordingly, when Drain 3 flows full, the water level rises above the top of the drain inlets, flooding the streets in the Monterey Place area. Drain 3 is currently operated to shut down discharge from upstream detention basins when the water level rises in Drain 3 at Monterey Place. This operation, however, limits the drainage capacity of Drain 3 for the rest of the upstream tributary area. To control flooding at Monterey Place and to increase the capacity of Drain 3, the City plans the following: 1) construct an underground storage vault sized for the Monterey Place tributary area; 2) disconnect and cap the Monterey Place storm laterals and reroute them to the underground storage vault, and 3) install a small pump station in the storage vault to discharge the attenuated flows into Drain 3 when capacity is available. See Figure 4-4 for the location of the Monterey Place improvements.

(2) Drain 5

Along Drain 5 in the vicinity of Airport Way, the twin 48-inch pipes immediately downstream of the Airport Way crossing are causing a flow restriction, which is causing upstream flooding problems. Replacing the twin 48-inch pipes with an open channel corridor will alleviate the flooding problems. See Figure 4-4 for the location of the Drain 5 improvement.

CHAPTER 5 BUILDOUT CONDITIONS

The purpose of this chapter is to document the assumptions and criteria used to simulate buildout conditions, and to identify what infrastructure is needed to accommodate buildout development to the City's 2023 General Plan Boundary.

5.1 Buildout Conditions and Buildout XP-SWMM Model

The following major assumptions and criteria were used to simulate future storm drainage flows and to analyze buildout conditions:

- All lands currently undeveloped within the 2023 General Plan boundary were assumed to be developed according to their respective land use designations, and runoff flows from these lands were estimated using the runoff coefficients described in Chapter 3.
- All future runoff is attenuated and discharged to major conveyance facilities at a 96-hour pump rate.
- The 25 cfs Rule is not used. That is, SSJID irrigation flows and City storm runoff are not simultaneously present in the Laterals in any of the analyses. This criterion is used since all future flows are attenuated and since existing and proposed monitoring stations provide real-time water elevations at critical low points in the major conveyance facilities.
- The FCOC is the only facility used to convey stormwater to the San Joaquin River. The exception to this criterion is for Zone 39 storm flows, which will be conveyed to the San Joaquin River via Walthall Slough.
- Drains 10 and 11 are not used as stormwater conveyance facilities as recommended by the 2006 SDMP. Instead, the South Drain Pipeline and Pump Station are used as the main facilities to convey stormwater from Zone 36 to the FCOC as described below.
- The use of SSJID Laterals to convey stormwater to the Drains is significantly expanded. Figure 3-1 in Chapter 3 depicts the various Laterals that are targeted to convey both irrigation water and storm runoff (i.e. dual-use facilities). Additional detail about the proposed dual-use facilities is provided below.

Zone 36 Drainage Conveyance Methodology

The 2006 SDMP proposed to route a significant portion of Zone 36 runoff to Drains 10 and 11 for discharge to the San Joaquin River. However, after researching the environmental impacts and maintenance costs associated with using Drains 10 and 11, the City believes it is more practical to route all Zone 36 flows to the FCOC for discharge to the San Joaquin River. Drain 8 currently provides conveyance of some existing Zone 36 flows to the FCOC, but its existing capacity is incapable of handling buildout flows. Also, it is impractical to enlarge Drain 8 to accommodate all Zone 36 buildout runoff flows. Thus, the City proposes to complete the construction of a drainage pipeline in Woodward Avenue – known as the South Drain Pipeline – to serve as the main stormwater conveyance facility for Zone 36.

Given the various vertical constraints of other existing utilities in Woodward Avenue, the invert of the South Drain Pipeline will be constructed deeper than the FCOC's invert. As such, flows in the South Drain Pipeline need to be pumped into the FCOC. Accordingly, the City proposes to construct a pump station – called the South Drain Pump Station – to accomplish this.

New Laterals Targeted for Dual-Use

(1) North Drain – Drainage Zone 30

The City proposes to use Lateral Q and portions of Lateral R as the main dual-use facilities for Zone 30 as shown in Figure 3-1.

(2) Drain 3 and Drain 3A – Drainage Zone 32

Drains 3 and 3A are the existing dual-use facilities in Zone 32. In addition to these facilities, the City proposes to use portions of Lateral R and its various branches Rg, Rga, Rfa and Rd, as shown in Figure 3-1.

(3) Drain 4 – Drainage Zone 34

Drain 4 is the existing dual-use facility in Zone 34, which mostly covers the existing downtown storm drainage shed. There are no additional dual-use facilities proposed for Drain 4 in Zone 34.

(4) Drain 5 – Drainage Zone 34

Drain 5 is the main existing dual-use facility in Zone 34, which covers portions of the existing downtown storm drainage shed. There are no additional dual-use facilities proposed for Drain 5 in Zone 34.

(5) Drain 7 – Drainage Zone 34

Drain 7 is the main existing dual-use facility in Zone 34. There are no additional dual-use facilities proposed for Drain 7 in Zone 34.

(6) South Drain – Drainage Zone 36

Drain 8 will continue serving the existing developed areas in Zone 36, and will continue to flow by gravity directly to the FCOC. For the remaining, undeveloped areas in Zone 36, the City proposes to use several existing Laterals as collector facilities for the South Drain Pipeline. Laterals Dd and Tbb are proposed as dual-use collector facilities for the areas east of Highway 99, and Laterals W and X (and their sub-branches) are proposed as the dual-use collector facilities for the areas south of Highway 120 and west of Highway 99. See Figure 3-1 for more detail.

5.2 Buildout Model Results

The 2013 Existing Conditions Model was updated with the assumptions and criteria noted above to create the Buildout Conditions Model. Various simulations were then run with the Buildout Conditions Model to determine what additional infrastructure was needed to properly convey stormwater through the City to the San Joaquin River without causing flooding. The following is a summary of the modeling results and a discussion about the various improvements needed to handle future runoff flows.

(1) Dual-Use Laterals

The modeling results confirmed that the expanded use of SSJID's Laterals to convey City stormwater to SSJID's Drains is a viable alternative. The Laterals function properly when the following conditions are met: A) they are re-constructed as minimum 42-inch diameter pipes; B) all future runoff flows are attenuated and released at the 96-hour pump rate, and C) irrigation water is not present when stormwater is present. Figures 3-1 and 5-1 show the locations of the Laterals scheduled for dual-use.

(2) FCOC Culvert Crossings

When buildout flows are conveyed to the FCOC, the model showed that the FCOC culvert crossings need improvement to prevent flooding, but a complete widening of the FCOC channel is not needed, as was recommended in the 2006 SDMP. The required culvert improvements are summarized in Table 5-1, and their locations are shown on Figure 5-1. Also, to prevent overtopping of the FCOC during operation of the storm drainage system, two additional water level monitoring stations are needed in the FCOC.

(3) South Drain Pipeline and Pump Station

Several model scenarios were run to identify what combination of pumping capacity and pipe diameter were optimal for the South Drain Pipeline and Pump Station. Ultimately, the model results showed that a 140 cfs pump station and 54-inch diameter pipe were the optimal combination. Also, two 36-inch diameter force mains are needed to convey flows from the pump station under State Route 120 to the FCOC. Figure 5-1 shows the alignment of the South Drain Pipeline and the location of the South Drain Pump Station and Force Main. The City plans to phase construction of the South Drain Pump Station to provide pumping capacity on an as-needed basis.

In addition, for Laterals X and Y to function as collector pipes, two new junction structures are needed to divert storm water from these Laterals to the South Drain Pipeline, as shown on Figure 5-1. Moreover, four water level monitoring stations are needed throughout Zone 36 to prevent flooding during operation of the South Drain system. The location in most need of a monitoring station is on the South Drain Pipeline in the vicinity of the Antiqua Subdivision.

(4) Austin Road Pipeline and Pump Station – Zone 36 East

As shown on Figure 3-1, Lateral T is a major conveyance facility that runs the full width of the City before connecting directly to the FCOC. Lateral T currently receives uncontrolled flows from the Direct Discharge Area (DDA), as well as controlled flows from newer existing developments.

To accommodate future runoff from areas east of Highway 99 and south of State Route 120 (Zone 36 East), the City evaluated routing some of this flow to Lateral T for conveyance to the FCOC. However, since Lateral T is already near capacity and since it receives uncontrolled flows from the DDA, the City believes a more prudent approach to serve Zone 36 East is with new facilities that convey the runoff to the South Drain Pipeline.

To convey future Zone 36 East runoff to the South Drain Pipeline, a pipeline will be constructed in Austin Road that routes flows from Lateral T to Lateral Tbb. Lateral Tbb will convey stormwater under Highway 99 through an existing highway crossing, then to an existing connection with the South Drain Pipeline. The Lateral Tbb / Hwy 99 crossing is very shallow and cannot serve Zone 36 East by gravity; therefore, a pump station is needed as shown on Figure 5-1.

(5) Zone 39 Regional Pump Station and Pipeline

Stormwater runoff from the developable area within Zone 39 will be conveyed through the regional pipeline to one regional pump station into Walthall Slough via an outfall and swale. Existing stormwater runoff from the Dutra Estate subdivision in Storm Zone 39 will be routed to the proposed separate storm drain basin for Storm Zone 39.

Table 5-1 Summary of Future Storm Drainage Improvements

Location	Recommended Improvement
Drain 5	Replace two 48” pipes with an open corridor
South Drain	Install 54” pipe in Woodward Ave from Main St to Galleria Dr.
South Drain	Install 54” pipe from McKinley Rd to South Drain Pump Station
South Drain	Construct South Drain Pump Station (140 cfs) with two 36” Force Mains under SR 120 and connect to FCOC
South Drain	Install two junction structures to connect Laterals W and X to the South Drain Pipeline
South Drain	Construct Austin Rd Pump Station and Pipeline
FCOC – Farm Rd	Replace two 36-inch culverts with 10-foot by 8-foot box culvert
FCOC – UPRR	Replace 42-inch and 36-inch culverts with two 60-inch culverts
FCOC – Roth Rd	Replace 42-inch and 48-inch culverts with two 12-foot and 8-foot box culverts
FCOC – French Camp Rd	Replace 66-inch culvert with two 12-foot and 8-foot box culverts
Zone 39	Construct Regional Pump Station and Pipeline for Developable Area
Zone 30, 32, 36	Increase 36” to 42” pipes for Dual-Use Laterals

CHAPTER 6 CAPITAL IMPROVEMENT PROGRAM

6.1 Drainage Improvement Prioritization

This chapter prioritizes the drainage improvements identified in Chapters 4 and 5 in order of greatest need. Group 1 improvements are needed immediately to either solve serious existing deficiencies or to support pending development projects. Groups 2 and 3 improvements follow in order of importance to remedy any remaining existing deficiencies or to serve demands that are 5 years or more in the future. This chapter also provides construction cost estimates for the various improvements, which have been escalated to May 2011 dollars based on the current ENR Cost Index for San Francisco of 9133.56. The improvements and their estimated construction costs are presented in Table 6-1.

The storm drainage improvements serving future growth are consistent with the City's Public Facilities Implementation Plan (PFIP). The PFIP Program includes all water, wastewater, storm drainage, and transportation facilities required to meet the City's targets for Level of Service. The PFIP Program ensures that infrastructure required for growth is constructed in a timely manner and financed in a way that equitably divides financial responsibility in proportion to the demands placed on new facilities. The full PFIP program resides in a document separate from the 2013 SDMP, and persons interested in learning more about the PFIP program should contact the City.

Table 6-1 Capital Improvement Program and Estimated Total Costs

Priority	Project Description	Cost Estimates
Group 1	Drain 3 – Monterey Place Improvements	$400,000
	FCOC – Union Pacific Railroad Crossing	$425,000
	South Drain – Main St to Galleria Dr	$3,700,000
	South Drain – McKinley Ave to South Drain Pump Station	$750,000
	South Drain – Two Junction Structures Laterals W and X	$150,000
	South Drain – South Drain Pump Station & Force Main (FM)	$2,500,000
	Trails (Zone 39) – Regional Pump Station, FM & Pipeline	$3,150,000
	Water Level Monitoring System (2)	$100,000
	Zone 36 – Increase 36” to 42” Pipes for Dual-Use Laterals	$60,000
Group 2	FCOC – Roth Road Crossings	$500,000
	FCOC – French Camp Road Crossing	$750,000
	FCOC – Farm Road Crossing	$250,000
	Drain 5 – Replace two 48” pipes with an open corridor	$300,000
	Water Level Monitoring System (2)	$100,000
Group 3	South Drain – Austin Rd Pump Station and Pipeline	$1,700,000
	Water Level Monitoring System (2)	$100,000
	Zone 30 & 32 – Increase 36” to 42” Pipe for Dual-Use Laterals	$120,000
Grand Total		$15,055,000

APPENDIX A1: CITY/SSJID DRAINAGE AGREEMENT

APPENDIX A2: LIST OF PUMP AND MONITORING STATIONS

APPENDIX A3: CITY STANDARD PLANS FOR STORM