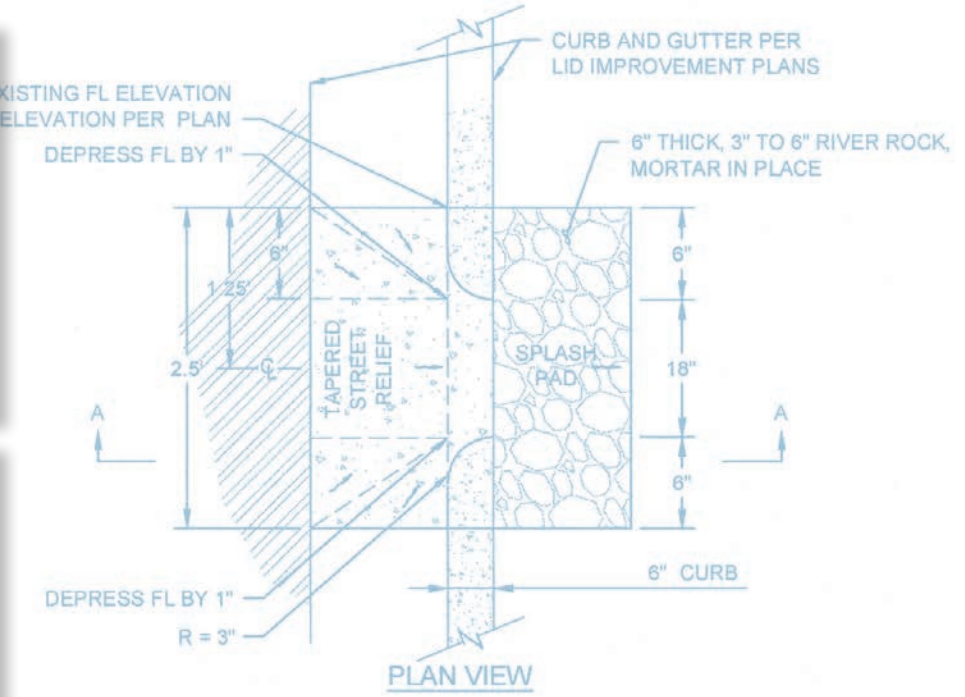
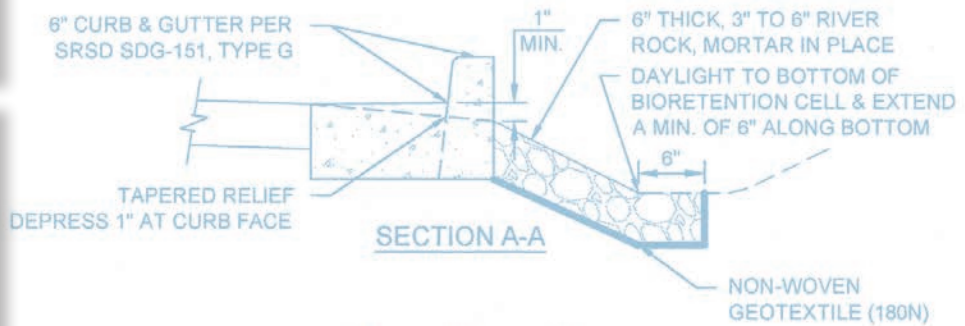
SAN MARCOS STORMWATER TECHNICAL MANUAL

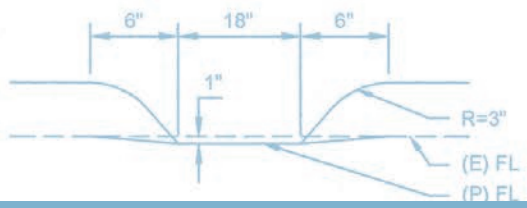












CITY OF SAN MARCOS, TEXAS

REVISED JUNE 1, 2020

CONTENTS

CHAPTER 1. INTRODUCTION

1.1: WHY MANAGE STORMWATER 1:2

1.2: HOW STORMWATER SHOULD BE MANAGED 1:2

CHAPTER 2. WHAT STORMWATER MANAGEMENT IS REQUIRED

2.1: FEDERAL REGULATIONS 2:2

2.2: STATE PERMITS AND RULES 2:2

2.2.1: TEXAS MUNICIPAL SEPARATE STORM SEWER SYSTEM PERMIT 2:2

2.2.2: TEXAS CONSTRUCTION GENERAL PERMIT 2:2

2.2.3: TCEQ EDWARDS AQUIFER RULES 2:3

2.3: SAN MARCOS DEVELOPMENT CODE (SMDC) 2:3

2.3.1: SITE PLANNING 2:3

2.3.1.1: NATURAL DRAINAGE 2:3

2.3.1.2: CUT AND FILL STANDARDS 2:4

2.3.1.3: IMPERVIOUS COVER LIMITATIONS 2:4

2.3.1.4: TREE PRESERVATION 2:6

2.3.1.5: GENERAL DEVELOPMENT LIMITATIONS 2:6

2.3.2: EROSION AND SEDIMENT CONTROL STANDARDS 2:6

2.3.3: POST-CONSTRUCTION STORMWATER PERFORMANCE STANDARDS 2:7

2.3.3.1: STORMWATER QUALITY AND STREAM PROTECTION 2:7

2.3.3.2: FLOOD CONTROL 2:7

2.3.4: PERFORMANCE STANDARD WAIVERS 2:8

CHAPTER 3. HOW TO MANAGE STORMWATER

3.1: SITE PLANNING/SCM SELECTION 3:4

3.1.1: PHASE I – SITE ASSESSMENT 3:4

3.1.1.1: PHASE I-A: IDENTIFY REGULATORY REQUIREMENTS 3:4

3.1.1.2: PHASE 1-B: CONDUCT HYDROLOGIC AND GEOTECHNICAL SURVEY 3:5

3.1.1.3: PHASE 1-C: PROTECT NATURAL FEATURES AND KEY HYDROLOGIC AREAS 3:5

3.1.2: PHASE II – PRELIMINARY DESIGN 3:6

3.1.2.1: PHASE II-A: USE DRAINAGE AND HYDROLOGY AS DESIGN ELEMENTS 3:6

3.1.2.2: PHASE II-B: ESTABLISH CLEARING AND GRADING LIMITS 3:7

3.1.2.3: PHASE II-C: REDUCE/MINIMIZE IMPERVIOUS COVER 3:7

3.1.2.4: PHASE II-D: DETERMINE APPROPRIATE SCMS 3:9

3.1.3: PHASE III – FINAL DESIGN 3:10

3.1.3.1: PHASE III-A SIZE SCMS 3:10

3.1.3.2: PHASE III-B: DEVELOP FINAL DESIGN 3:10

3.2: DETERMINATION OF STORMWATER RUNOFF 3:11

3.2.1: HYDROLOGY 3:11

3.2.1.1: EFFECTS OF URBANIZATION 3:11

3.2.1.2: DESIGN ASSUMPTIONS FOR STORM RUNOFF ANALYSIS 3:11

3.2.1.3: METHOD OF ANALYSIS 3:12

3.2.1.4: RATIONAL METHOD 3:12
3.2.1.5: RUNOFF COEFFICIENT (C) 3:13
3.2.1.6: TIME OF CONCENTRATION 3:14
3.2.1.7: DRAINAGE AREA (A) 3:18
3.2.1.8: THE SOIL CONSERVATION SERVICE METHOD FOR CALCULATION OF PEAK FLOWS 3:19
3.2.1.9: PROBABLE MAXIMUM STORM/FLOOD DEVELOPMENT 3:26
3.2.1.10: IMPERVIOUS COVER DETERMINATION 3:27
3.2.2: CALCULATION OF WATER QUALITY VOLUME 3:28
3.2.2.1: CALCULATING THE TSS LOAD REDUCTION AND WATER QUALITY VOLUME FOR TSS TREATMENT 3:28
3.2.2.2: CALCULATE WATER QUALITY VOLUME BASED ON DESIGN RAINFALL 3:31
3.2.2.3: EXAMPLE DEVELOPMENT SITE CALCULATIONS 3:31
3.2.3: INTEGRATED STORMWATER MANAGEMENT 3:34
3.2.3.1: PERFORMANCE CRITERIA FOR SWM PONDS 3:34
3.2.3.2: DETENTION POND STORAGE DETERMINATION 3:35
3.2.3.3: OUTLET STRUCTURE DESIGN 3:35
3.3: INFRASTRUCTURE AND RIGHT OF WAY 3:36
3.3.1: STREET DRAINAGE 3:36
3.3.2: STREET FLOW 3:36
3.3.2.1: GENERAL 3:36
3.3.2.2: PERMISSIBLE SPREAD OF WATER 3:37
3.3.2.3: DESIGN METHOD 3:37
3.3.3: INLETS 3:40
3.3.3.1: INLET TYPES 3:40
3.3.3.2: DESIGN GUIDELINES 3:40
3.3.3.3: MATERIALS AND SPECIFICATIONS 3:45
3.3.4: STORM DRAIN SYSTEMS 3:45
3.3.4.1: INTRODUCTION 3:45
3.3.4.2: HYDRAULICS OF STORM DRAINAGE SYSTEMS 3:46
3.3.4.3: DESIGN GUIDELINES 3:53
3.3.4.4: MAINTENANCE CONSIDERATIONS 3:55
3.3.5: OPEN CHANNELS 3:55
3.3.5.1: GENERAL 3:55
3.3.5.2: OPEN CHANNEL HYDRAULICS 3:56
3.3.5.3: MANNING'S ROUGHNESS COEFFICIENTS 3:57
3.3.5.4: DESIGN REQUIREMENTS 3:59
3.3.5.5: CHANNEL DROP STRUCTURES 3:60
3.3.5.6: ENERGY DISSIPATERS 3:61
3.3.5.7: STRUCTURE AESTHETICS 3:62
3.3.6: BRIDGES AND CULVERTS 3:63
3.3.6.1: GENERAL 3:63
3.3.6.2: CULVERT HEADWALLS AND ENDWALLS 3:63
3.3.6.3: CULVERT DISCHARGE VELOCITIES 3:65
3.3.6.4: SELECTION OF CULVERT SIZE AND FLOW CLASSIFICATION 3:66
3.3.6.5: HYDRAULIC CONSIDERATIONS IN BRIDGE DESIGN 3:67
CHAPTER 4. RESOURCES

APPENDICES

APPENDIX A. EROSION AND SEDIMENT CONTROL BMPS

1: INTRODUCTION A:2

2: SELECTION AND DESIGN A:3

2.1: TEMPORARY PRACTICES A:3

2.2: PERMANENT PRACTICES A:4

3: MAINTENANCE REQUIREMENTS A:5

4: INSPECTIONS A:5

APPENDIX B. LOW IMPACT DEVELOPMENT PRACTICES

B.1: LOW IMPACT DEVELOPMENT STORMWATER CONTROL MEASURES B:3

B.1.1: INTRODUCTION B:3

B.1.2: INSPECTIONS B:7

B.1.3: REFERENCES B:7

B.2: SITE PLANNING/SCM SELECTION B:8

B.2.1: EXAMPLE LID CONCEPTUAL DESIGN B:10

B.3: RAIN GARDENS AND BIORETENTION FACILITIES B:31

B.3.1: DESCRIPTION B:31

B.3.2: SELECTION CRITERIA B:33

B.3.3: DESIGN SPECIFICATIONS B:33

B.3.4: INSPECTION CRITERIA B:49

B.3.5: RECOMMENDED MAINTENANCE B:50

B.3.6: REFERENCES B:52

B.4: POROUS PAVEMENT B:54

B.4.1: DESCRIPTION B:54

B.4.2: SELECTION CRITERIA B:55

B.4.3: DESIGN B:55

B.4.4: CONSTRUCTION CONSIDERATIONS B:66

B.4.5: INSPECTION CRITERIA B:67

B.4.6: RECOMMENDED MAINTENANCE B:69

B.4.7: REFERENCES B:69

B.5: RAINWATER HARVESTING B:71

B.5.1: DESCRIPTION B:71

B.5.2: SELECTION CRITERIA B:72

B.5.3: DESIGN SPECIFICATIONS B:72

B.5.4: CONSTRUCTION CONSIDERATIONS B:82

B.5.5: INSPECTION CRITERIA B:82

B.5.6: OPERATION AND MAINTENANCE B:83

B.5.7: REFERENCES B:84

B.6: ADDITIONAL LID SCMS B:85
B.6.1: VEGETATED FILTER STRIPS B:85
B.6.2: VEGETATED SWALES B:86
B.6.3: GREEN ROOFS B:88
B.6.4: CONSTRUCTED WETLANDS AND WET PONDS B:90
B.6.5: PROPRIETARY SYSTEMS B:91
B.7: TREATMENT TRAINS B:92
B.8: COMMON DESIGN ELEMENTS B:93
B.8.1: GEOTECHNICAL INVESTIGATION B:93
B.8.2: CURB CUTS B:93
B.8.3: STABILIZATION AND ENERGY DISSIPATION B:97
B.8.4: UNDERDRAIN DESIGN B:98
B.8.5: TRASH RACKS B:100
B.8.6: DIVERSION STRUCTURES B:101
B.8.7: IMPERMEABLE LINERS AND HYDRAULIC RESTRICTION BARRIERS B:102
B.8.8: OUTLET STRUCTURES B:104
B.8.9: UTILITIES B:105
B.8.10: CONNECTIVITY B:105
B.8.11: ADA REQUIREMENTS B:107
B.8.12: REFERENCES B:107

APPENDIX C. STORMWATER DETENTION GUIDELINES

1: INTRODUCTION C:2
1.1: GENERAL C:2
1.2: DETENTION SCMS C:2
1.2.1: TYPES C:2
1.2.2: FUNCTIONALITY C:3
1.2.3: LIMITATIONS C:3
1.2.4: SAFETY CRITERIA FOR SWM PONDS C:3
1.2.5: OPERATION AND MAINTENANCE C:5
1.2.6: MAINTENANCE ACCESS C:5
2: AESTHETICALLY ENHANCED DETENTION BASINS C:6
2.1: BACKGROUND C:6
2.1.1: WHY ARE AESTHETIC STORMWATER DETENTION AND WATER QUALITY FACILITIES IMPORTANT TO DEVELOPERS? C:6
2.1.2: STORMWATER BASIN ZONES C:7
2.2: TOOLS FOR SUCCESS C:8
2.2.1: SITING AND FUNCTIONALITY C:8
2.3: PHYSICAL CHARACTER AND ARCHITECTURAL ELEMENTS C:9
2.4: LANDSCAPE DESIGN C:10
2.5: GUIDELINES FOR SITE ENHANCEMENT C:11

APPENDIX D. DESIGN FIGURES

FIGURES

FIGURE 2.1 SAN MARCOS URBAN DISTRICT MAP 2:8

FIGURE 3.1 IDENTIFY APPLICABLE ZONING REQUIREMENTS, UTILITY EASEMENTS, AND SITE SETBACKS. 3:5

FIGURE 3.2 IDENTIFY IDEAL LOCATIONS FOR LID IMPLEMENTATION ACCORDING TO SITE CONDITIONS. 3:7

FIGURE 3.3 ESTABLISH GRADING ENVELOPE TO PROTECT NATURAL AREAS AND INFILTRATING SOILS. 3:8

FIGURE 3.4 RUNOFF RATE VERSUS TIME HYDROGRAPH. 3:11

FIGURE 3.5 RELATIONSHIP BETWEEN RUNOFF COEFFICIENT AND IMPERVIOUS COVER 3:30

FIGURE 3.6 CONCEPTUAL DEVELOPMENT SITE 3:31

FIGURE 3.7 HYDRAULIC AND ENERGY GRADE LINE IN A CONDUIT (SOURCE FHWA HEC NO. 22) 3:46

FIGURE 3.8 ANGLE OF CONE FOR CHANGE IN PIPE DIAMETER (SOURCE FHWA HEC 22) 3:48

FIGURE 3.9 INTERIOR ANGLE FOR PIPE JUNCTION WITHOUT MANHOLE (SOURCE FHWA HEC 22) 3:50

FIGURE 3.10 SKETCH OF FHWA ACCESS HOLE METHOD (SOURCE FHWA HEC-22, 3RD EDITION) 3:51

FIGURE 3.11 DEFLECTION ANGLE 3:52

FIGURE 3.12 RELATIVE FLOW EFFECT 3:52

FIGURE 3.13 MANHOLE BENCHING METHODS(SOURCE FHWA HEC-22, 3RD EDITION) 3:53

FIGURE B.2.1 STEPS TO DEVELOP A SITE PLAN WITH INTEGRATED STORMWATER MANAGEMENT. B:9

FIGURE B.2.2 EXAMPLE RESIDENTIAL DEVELOPMENT CONCEPTUAL SITE FOR LID AND DETENTION. B:10

FIGURE B.2.3 IDENTIFY APPLICABLE ZONING REQUIREMENTS, UTILITY EASEMENTS, AND SITE SETBACKS. B:11

FIGURE B.2.4 SAN MARCOS ECO-REGIONS B:12

FIGURE B.2.5 TYPOLOGY OF EDWARDS PLATEAU AND BLACKLAND PRAIRIE REGIONS (ADAPTED FROM AND COURTESY OF CALPOLY STUDIO 606) B:13

FIGURE B.2.6 PROTECT NATURAL AND SENSITIVE AREAS (WETLANDS, NATIVE TREE GROVES, STEEP HILLSIDE) AND CONDUCT GEOTECHNICAL SURVEY TO CHARACTERIZE INFILTRATION CAPACITY OF SOILS. B:15

FIGURE B.2.7 EXAMPLE CROSS-SECTION OF A CREEK/RIVER AND THE ASSOCIATED BUFFERS. B:17

FIGURE B.2.8 IDENTIFY AND PROTECT KEY HYDROLOGIC AREAS, SUCH AS INFILTRATING SOILS (BLUE AREA) AND RIPARIAN BUFFER (PURPLE AREA) B:18

FIGURE B.2.9 IDENTIFY IDEAL LOCATIONS FOR LID IMPLEMENTATION ACCORDING TO SITE CONDITIONS. B:20

FIGURE B.2.10 ESTABLISH GRADING ENVELOPE TO PROTECT NATURAL AREAS AND INFILTRATING SOILS. B:21

FIGURE B.2.11 DISCONNECTION AT PORTLAND STATE UNIVERSITY, OREGON, WHERE VEGETATED AREAS ARE DISTRIBUTED THROUGHOUT THE IMPERVIOUS COURTYARD AREA. B:22

FIGURE B.2.12 DOWNSPOUT DIRECTED TO A RAIN GARDEN AFTER PASSING OVER A STONE SPLASH PAD. B:22

FIGURE B.2.13 DOWNSPOUTS DIRECTED TO PERMEABLE PAVEMENT ON DRIVEWAY (COURTESY OF MONTGOMERY COUNTY, MD). B:23

FIGURE B.2.14 TREATMENT TRAIN APPROACH: DISCONNECTED DOWNSPOUT RUNOFF ENTERS A SERIES OF VEGETATED CONTROLS. B:23

FIGURE B.2.15 SITE EXAMPLE DEMONSTRATING PLACEMENT OF PERVIOUS MATERIAL (RED) A ND OPPORTUNITIES TO MINIMIZE CONNECTED IMPERVIOUS AREA. B:24

FIGURE B.2.16 CONVENTIONAL DESIGN (LEFT) AND CONSERVATION DESIGN (RIGHT) (COURTESY TOWN OF PINE PLAINS, NY). B:24

FIGURE B.2.17 SITE PLAN INDICATING ALL POSSIBLE SCM LOCATIONS (BLUE AREAS) AND TYPES (ANNOTATED). B:29
FIGURE B.3.1 A DIAGRAM OF THE BASIC RAIN GARDEN/BIORETENTION SYSTEM COMPONENTS, INCLUDING OPTIONAL COMPONENTS. B:32
FIGURE B.3.2 BASIC BIORETENTION COMPONENTS. B:33
FIGURE B.3.3 CONVENTIONALLY DRAINED BIORETENTION CROSS SECTION SHOWING UNDERDRAIN. B:36
FIGURE B.3.5 UPTURNED UNDERDRAINS INSIDE BIORETENTION OUTLET STRUCTURES CREATE IWS IN SOIL MEDIA TO IMPROVE INFILTRATION, WATER QUALITY, AND PLANT HEALTH. B:36
FIGURE B.3.4 BIORETENTION CELL PROFILE WITH IWS DRAINAGE CONFIGURATION. B:36
FIGURE B.3.6 SCHEMATIC OF A BIORETENTION SYSTEM. B:37
FIGURE B.3.7 CURB CUT OPENINGS WITH FLOW TO VEGETATED AREA B:40
FIGURE B.3.8 INLET AND PRETREATMENT PROVIDED BY MORTARED COBBLE FOREBAY AND ENERGY DISSIPATER. B:41
FIGURE B.3.9 INLETS STABILIZED WITH MORTARED COBBLE. B:41
FIGURE B.3.10 UPTURNED INLET FROM ROOFTOP BUBBLES UP DIFFUSELY ONTO GRAVEL PAD. B:41
FIGURE B.3.11 OFFLINE BIORETENTION AREA WHERE SYSTEM FILLS TO CAPACITY AND EXCESS FLOW BYPASSES ALONG CURBLINE AT INLET. B:42
FIGURE B.3.12 ONLINE BIORETENTION AREA WITH A VERTICAL RISER OVERFLOW WITH A VARIABLE FLOW OUTLET STRUCTURE. B:42
FIGURE B.3.13 BIORETENTION OUTLET STRUCTURES DESIGNED FOR PEAK FLOW MITIGATION. B:43
FIGURE B.3.14 TRIPLE-SHREDDED HARDWOOD MULCH. B:44
FIGURE B.3.15 EXAMPLE BIORETENTION PLANTING PLAN. B:46
FIGURE B.3.16 SIGNAGE POSTED IN FRONT OF ULTRA-URBAN BIORETENTION AREA RAISES PUBLIC AWARENESS. B:47
FIGURE B.3.17 COBBLED BIORETENTION AREA THAT INVITES PEDESTRIAN INTERACTION. B:47
FIGURE B.3.18 OVERFLOW FROM A RAINWATER CISTERN IS DISCHARGED TO A COBBLED BIORETENTION AREA. B:48
FIGURE B.4.1 PERMEABLE PAVEMENT AND BIORETENTION TREATMENT TRAIN. B:57
FIGURE B.4.2 TYPICAL POROUS ASPHALT CROSS SECTION. B:58
FIGURE B.4.3 EXAMPLE OF POROUS ASPHALT. B:59
FIGURE B.4.4 TYPICAL PERVIOUS CONCRETE CROSS SECTION. B:59
FIGURE B.4.6 TYPICAL PICP CROSS SECTION. B:60
FIGURE B.4.7 EXAMPLE OF PICPS IN A HERRINGBONE PATTERN. B:60
FIGURE B.4.5 EXAMPLE OF PERVIOUS CONCRETE. B:60
FIGURE B.4.8 EXAMPLE OF CGPS WITH VOLCANIC ROCK BEDDING COURSE. B:61
FIGURE B.4.9 TYPICAL PLASTIC GRID SYSTEM CROSS SECTION. B:61
FIGURE B.4.10 EXAMPLE OF PLASTIC GRID SYSTEMS. B:62
FIGURE B.4.12 EXAMPLE PERMEABLE PAVEMENT PROFILE FEATURING IWS. B:63
FIGURE B.4.11 EXAMPLE OF PERMEABLE PAVEMENT WITH TRENCHED UNDERDRAIN. B:63
FIGURE B.4.13 OBSERVATION WELL INSTALLED IN PERMEABLE PAVEMENT. B:64
FIGURE B.4.14 AN 8-INCH CONCRETE TRANSITION STRIP IS USED AS AN EDGE RESTRAINT BETWEEN PICP AND IMPERMEABLE ASPHALT. B:66
FIGURE B.5.1 MINIMUM DESIGN COMPONENTS OF RAINWATER HARVESTING SYSTEM. B:74
FIGURE B.5.2 CISTERN LESS THAN 2,000 PSI ON A GRAVEL FOUNDATION. B:74
FIGURE B.5.3 CISTERN GREATER THAN 2,000 PSI ON A CONCRETE FOUNDATION. B:74
FIGURE B.5.4 CISTERN ON A CONCRETE FOUNDATION. B:75
FIGURE B.5.5 DRY CONVEYANCE INLET CONFIGURATION. B:75
FIGURE B.5.6 INLET IN THE TOP OF THE CISTERN. B:75
FIGURE B.5.7 INLET IN THE SIDES OF THE MAN WAY. B:75

FIGURE B.5.9 CISTERN WITH A WET CONVEYANCE INLET CONFIGURATION. B:76
FIGURE B.5.8 CISTERN WITH WET CONVEYANCE FEATURING A DRAWDOWN VALVE FOR MAINTENANCE. B:76
FIGURE B.5.10 INLET FILTERS AT THE GUTTER. B:77
FIGURE B.5.11 INLET FILTERS AT THE DOWNSPOUT. B:77
FIGURE B.5.12 FLOW-THROUGH INLET FILTER. B:77
FIGURE B.5.13 SELF-FLUSHING FILTER WITH A BYPASS. B:78
FIGURE B.5.14 VALVE FOR A FIRST-FLUSH DIVERTER. B:78
FIGURE B.5.15 FIRST FLUSH DIVERTER CONFIGURATION. B:78
FIGURE B.5.16 CISTERN OUTLET. B:79
FIGURE B.5.18 TOP: A CHANNEL DIRECTS OVERFLOW AWAY FROM BUILDING. BOTTOM: A CISTERN OVERFLOWS TO AN ADJACENT BIORETENTION AREA. B:80
FIGURE B.5.19 TWO CISTERNS WITH PIPE CONNECTIONS. B:81
FIGURE B.5.20 TOP: CONCEPTUAL SCHEMATIC OF CISTERN WITH SUBMERSIBLE PUMP. BOTTOM: CONCEPTUAL SCHEMATIC OF CISTERN WITH EXTERNAL PUMP. B:82
FIGURE B.6.1 VEGETATED FILTER STRIP THAT PRETREATS ROADWAY RUNOFF. B:86
FIGURE B.6.2 VEGETATED FILTER STRIP SURROUNDING A BIORETENTION AREA IN A PARKING LOT. B:86
FIGURE B.6.3 VEGETATED FILTER STRIP NEXT TO A PARKING LOT. B:86
FIGURE B.6.4 VEGETATED SWALE IN AN INSTITUTIONAL SETTING. B:87
FIGURE B.6.5 ROADSIDE VEGETATED SWALE. B:87
FIGURE B.6.6 EXTENSIVE GREEN ROOFS REDUCE STORMWATER RUNOFF WHILE PROVIDING COOLING EFFECTS, HABITAT FOR POLLINATORS, AND AESTHETIC VALUE. B:89
FIGURE B.6.7 INTENSIVE GREEN ROOFS PROVIDE RECREATIONAL, AESTHETIC, AND EDUCATIONAL OPPORTUNITIES IN ADDITION TO STORMWATER BENEFITS. B:89
FIGURE B.6.8 A LARGE LINEAR STORMWATER WETLAND. B:90
FIGURE B.6.9 SMALL WETLANDS ALONG THE PERIMETER OF A NEIGHBORHOOD. B:91
FIGURE B.7.1 TREATMENT TRAIN FEATURING A PERMEABLE PAVEMENT CAPTURING RUNOFF BEFORE ENTERING A GRASSED BIORETENTION AREA. B:92
FIGURE B.8.1 TYPICAL CURB CUT DIAGRAM. B:94
FIGURE B.8.2 A TYPICAL CURB CUT. B:95
FIGURE B.8.3 MULTIPLE, SMALL CURB CUTS DISTRIBUTE STREET RUNOFF TO BIORETENTION AREAS WITHOUT EROSIVE FORCE. B:95
FIGURE B.8.5 ARMORED CURB CUTS. B:96
FIGURE B.8.6 COVERED CURB CUT WITH SUMP. B:96
FIGURE B.8.4 CURB CUTS AND VEGETATED FILTER STRIP FOR ROADSIDE BIORETENTION AREA. B:96
FIGURE B.8.7 STONE FLOW DISSIPATER/FOREBAY. B:97
FIGURE B.8.8 CONCRETE ENERGY DISSIPATER. B:97
FIGURE B.8.9 UPTURNED ROOF DOWNSPOUT ENERGY DISSIPATER. B:97
FIGURE B.8.11 TYPICAL LEVEL SPREADER PLAN VIEW. B:98
FIGURE B.8.10 TYPICAL LEVEL SPREADER PROFILE VIEW. B:98
FIGURE B.8.12 FIGURE ILLUSTRATION PROPER INSTALLATION OF ARCED LEVEL SPREADER (LEFT) AND IMPROPER LEVEL SPREADER ARC (RIGHT) B:98
FIGURE B.8.13 UNDERDRAIN BARRIER OPTION 1: SOIL MEDIA BARRIER. B:99
FIGURE B.8.14 UNDERDRAIN BARRIER OPTION 2: GEOTEXTILE LINER. B:99
FIGURE B.8.15 DRAWDOWN PIPE WITH A DOWNTURNED ELBOW. B:100
FIGURE B.8.16 OUTLET STRUCTURE WITH A TRASH RACK AND PROTECTED DRAWDOWN ORIFICE. B:100
FIGURE B.8.17 SCHEMATIC OF A TRASH RACK FOR DEWATERING SEDIMENTATION CHAMBERS AND STORMWATER WETLANDS (ADAPTED FROM BARRETT 2005). B:100
FIGURE B.8.18 TYPICAL DIVERSION STRUCTURE. B:101

FIGURE B.8.19 LATERAL HYDRAULIC RESTRICTION LAYERS IN A ROADSIDE BIORETENTION AREA PREVENT HORIZONTAL SEEPAGE WHILE ALLOWING INFILTRATION AT A SAFE DEPTH. B:102
FIGURE B.8.20 BIORETENTION AREA WITH GEOMEMBRANE LINER. B:103
FIGURE B.8.21 SCHEMATIC OF DETENTION BASIN OUTLET STRUCTURE. B:104
FIGURE B.8.22 ACCESS OVER LINEAR SCMS. B:106
FIGURE B.8.23 LOW-LEVEL FENCING. B:107
FIGURE B.8.24 LOW-PROFILE CURBING. B:107
FIGURE C.1 DETENTION BASIN WITH AESTHETIC ENHANCEMENTS, SAN ANTONIO, TX – SOURCE: TETRA TECH C:6
FIGURE C.2 C:7
FIGURE C.3 DETENTION POND AND POCKET WETLAND INTEGRATED INTO SITE FEATURES, NORTH CAROLINA, SOURCE: TETRA TECH. C:8
FIGURE C.4 DETENTION BASIN DESIGNED FOR PLAY ACCESS,SAN ANTONIO, TX, SOURCE: TETRA TECH. C:11
FIGURE D.1 NOMOGRAPH FOR FLOW IN GUTTERS D:2
FIGURE D.2 CURB OPENING INLET IN A SUMP (TYPE S-1) D:3
FIGURE D.3 GRATE INLET IN A SUMP (TYPE S-2) D:4
FIGURE D.4 COMBINATION INLET IN A SUMP (TYPE S-3) D:5
FIGURE D.5 AREA INLET WITHOUT GRATE (TYPE S-4) D:6
FIGURE D.6 CURB OPENING, INLET ON GRADE (TYPE G-1) D:7
FIGURE D.7 GRATE, INLET ON GRADE (TYPE G-2) D:8
FIGURE D.8 COMBINATION INLET ON GRADE (TYPE G-3) D:9
FIGURE D.9 CURB OPENING INLETS D:10
FIGURE D.10 DEPRESSED CURB OPENING INLET CAPACITY IN SUM P LOCATIONS – ENGLISH UNITS D:11
FIGURE D.11 UNDEPRESSED CURB OPENING INLET CAPACITY IN SUMP LOCATIONS – ENGLISH UNITS D:12
FIGURE D.12 CURB OPENING INLET ORIFICE CAPACITY FOR INCLINED AND VERTICAL ORIFICE THROATS – ENGLISH UNITS D:13
FIGURE D.13 DEFINITION OF DEPTH D:14
FIGURE D.14 GRATE INLET CAPACITY IN SUMP CONDITIONS – ENGLISH UNITS D:15
FIGURE D.15 CURB OPENING AND SLOTTED DRAIN INLET LENGTH FOR TOTAL INTERCEPTION – ENGLISH UNITS D:16
FIGURE D.16 CURB OPENING AND SLOTTED DRAIN INLET INTERCEPTION EFFICIENCY D:17
FIGURE D.17 DEPRESSED CURB OPENING INLET D:18
FIGURE D.18 HEADLOSS COEFFICIENTS FOR CASES 2 AND 7 D:19
FIGURE D.19 HEADLOSS COEFFICIENT FOR CASE 5 D:20
FIGURE D.20 MAXIMUM DISTANCES FOR MANHOLE LOCATIONS D:21
FIGURE D.21 UNIFORM FLOW FOR TRAPEZOIDAL CHANNELS D:22
FIGURE D.22 SLOPING AND VERTICAL CHANNEL DROPS D:23
FIGURE D.23 BAFFLED APRON AND ITS DESIGN CURVE D:24
FIGURE D.24 CONCEPTUAL DESIGN OF ALTERNATIVE CHANNEL D:25
FIGURE D.25 HEADWALL ENTRANCE TYPES D:26
FIGURE D.26 CONCEPTUAL DESIGN OF DEBRIS FINS D:27
FIGURE D.27 INLET AND OUTLET CONDITIONS FOR CULVERTS D:28
FIGURE D.28 HYDRAULICS OF A CULVERT UNDER OUTLET CONTROL CONDITION D:29
FIGURE D.29 TYPES OF FLOW FOR BRIDGE DESIGN D:30

TABLES

TABLE 2.1 WATER QUALITY VOLUME AND TREATMENT LEVEL TABLE 2:7
TABLE 3.1 RATIONAL METHOD RUNOFF COEFFICIENTS FOR COMPOSITE ANALYSIS 3:13
TABLE 3.2 MANNING'S "N" FOR OVERLAND FLOW 3:15
TABLE 3.3 DEPTH-DURATION-FREQUENCY TABLE FOR THE SAN MARCOS REGION 3:16
TABLE 3.4 INTENSITY-DURATION-FREQUENCY TABLE FOR THE SAN MARCOS REGION 3:17
TABLE 3.5 SAN MARCOS REGION INTENSITY-DURATION-FREQUENCY CURVE COEFFICIENTS 3:17
TABLE 3.6 SAN MARCOS 100-YEAR STORM FREQUENCY DISTRIBUTION ORDINATES IN 5-MINUTE INCREMENTS 3:20
TABLE 3.7 NRCS RUNOFF CURVE NUMBERS FOR URBAN AREAS AND AGRICULTURAL LANDS (ASSUMING ARC-II CONDITION) 3:24
TABLE 3.8 PROBABLE MAXIMUM PRECIPITATION DEPTHS FOR THE CITY OF SAN MARCOS 3:26
TABLE 3.9 SCM AND TSS REMOVAL EFFICIENCY 3:28
TABLE 3.10 RELATIONSHIP BETWEEN FRACTION OF ANNUAL RAINFALL AND RAINFALL DEPTH (IN) 3:30
TABLE 3.11 MINIMUM CLEAR WIDTHS FOR ROADWAY DESIGN WHEN GUTTER IS FLOWING FULL 3:37
TABLE 3.12 COEFFICIENTS FOR EQUATION 3.15, STREETS WITHOUT CURB SPLIT 3:38
TABLE 3.13 COEFFICIENTS FOR EQUATION 3.16, STREETS WITH CURB SPLIT – HIGHER GUTTER 3:39
TABLE 3.14 COEFFICIENTS FOR EQUATION 3.17, STREETS WITH CURB SPLIT – LOWER GUTTER 3:39
TABLE 3.15 SPLASH-OVER VELOCITY CALCULATION EQUATIONS (ENGLISH) 3:43
TABLE 3.16 CLOGGING COEFFICIENTS FOR MULTIPLE UNITS 3:44
TABLE 3.17 MANNING'S ROUGHNESS COEFFICIENT 3:47
TABLE 3.18 (SOURCE FHWA HEC 22) 3:48
TABLE 3.19 (SOURCE ASCE MANUALS AND REPORTS OF ENGINEERING PRACTICE NO. 77) 3:48
TABLE 3.20 (SOURCE ASCE MANUALS AND REPORTS OF ENGINEERING PRACTICE NO. 77) 3:49
TABLE 3.21 (SOURCE ASCE MANUALS AND REPORTS OF ENGINEERING PRACTICE NO. 77) 3:49
TABLE 3.22 (SOURCE ASCE MANUALS AND REPORTS OF ENGINEERING PRACTICE NO. 77) 3:50
TABLE 3.23 CORRECTION FOR BENCHING 3:53
TABLE 3.24 MAXIMUM VELOCITY 3:53
TABLE 3.25 COMPUTATION OF COMPOSITE ROUGHNESS COEFFICIENT FOR EXCAVATED AND NATURAL CHANNELS 3:57
TABLE 3.26 MINIMUM ROUGHNESS COEFFICIENTS OF NEW OR ALTERED CHANNELS 3:58
TABLE 3.27 LENGTH OF DOWNSTREAM APRON 3:61
TABLE 3.28 VALUES OF CULVERT ENTRANCE LOSS COEFFICIENTS 3:64
TABLE B.3.1 ADVANTAGES AND LIMITATIONS OF BIORETENTION AREAS B:32
TABLE B.3.2 ITERATIVE DESIGN STEP PROCESS B:34
TABLE B.3.3 DECISION TABLE FOR DETERMINING UNDERDRAIN AND IMPERMEABLE LINER REQUIREMENTS B:35
TABLE B.3.4 MINIMUM BIORETENTION MEDIA DEPTH TO TREAT POLLUTANTS OF CONCERN (HUNT ET AL. 2012) B:38
TABLE B.3.5 BIORETENTION SOIL MEDIA SPECIFICATIONS (HUNT ET AL. 2012) B:39
TABLE B.3.6 LIST OF REGIONALLY APPROPRIATE SPECIES B:46
TABLE B.3.7 INSPECTION AND MAINTENANCE TASKS B:52
TABLE B.4.1 ADVANTAGES AND LIMITATIONS OF PERMEABLE PAVEMENT B:55
TABLE B.4.2 ITERATIVE DESIGN STEP PROCESS B:56
TABLE B.4.3 DECISION TABLE FOR DETERMINING UNDERDRAIN AND IMPERMEABLE LINER REQUIREMENTS B:62

TABLE B.4.4 GEOTEXTILE LAYER SPECIFICATIONS B:64
TABLE B.4.5 SUBGRADE TREATMENTS FOR INFILTRATION ENHANCEMENT B:67
TABLE B.4.6 OPERATION AND MAINTENANCE TASKS FOR PERMEABLE PAVEMENT B:68
TABLE B.5.1 ADVANTAGES AND LIMITATIONS OF RAINWATER HARVESTING B:72
TABLE B.5.2 ITERATIVE DESIGN STEP PROCESS B:73
TABLE B.5.3 IRRIGATION AREA REQUIREMENTS FOR CISTERNS IN THE EDWARDS AQUIFER RECHARGE, CONTRIBUTING, AND TRANSITION ZONES (APPLICABLE TO ALL AREAS) B:79
TABLE B.5.4 INSPECTION AND MAINTENANCE TASKS FOR CISTERNS B:84
TABLE B.6.1 ADVANTAGES AND LIMITATIONS OF FILTER STRIPS B:85
TABLE B.6.2 ADVANTAGES AND LIMITATIONS OF VEGETATED SWALES B:87
TABLE B.6.3 ADVANTAGES AND LIMITATIONS OF GREEN ROOFS B:88
TABLE B.6.4 ADVANTAGES AND LIMITATIONS OF STORMWATER WETLANDS B:90
TABLE B.8.1 UNDERDRAIN SPECIFICATIONS (BARRETT 2005) B:99
TABLE B.8.2 GEOTEXTILE LAYER SPECIFICATIONS (BARRETT 2005) B:100
TABLE B.8.3 CLAY LINER SPECIFICATIONS (BARRETT 2005) B:103
TABLE B.8.4 PROTECTIVE GEOTEXTILE FABRIC SPECIFICATIONS (BARRETT 2005) B:103

CHAPTER 1. INTRODUCTION

1.1: WHY MANAGE STORMWATER 1:2

1.2: HOW STORMWATER SHOULD BE MANAGED 1:2

This manual is written to provide designers technical criteria and guidance necessary to meet stormwater requirements in the San Marcos Development Code and to provide information on the benefits of integrated stormwater management which combines water quality and flood control benefits.

This manual is organized as a main volume and three appendices. The intent is for users to be able to quickly navigate to the information they need using the following links:

1.1: WHY MANAGE STORMWATER

Urban land development increases the intensity of storm runoff flows and the amount of stormwater pollution reaching local water resources. Buildings, roads, and other impervious surfaces shed rain more rapidly than areas covered by vegetation, and most typical urban land uses require rapid drainage of stormwater. The very rapid, direct connection of developed land across paved surfaces and through drainage conveyances to waterways tends to carry more pollutants more quickly from the land surface to water resources. A number of water quality problems and impairments in Texas are attributed in full or in part to such urban stormwater runoff carried through storm sewers and channelized streams.

The management of stormwater during and after construction of development projects helps to minimize the impacts on receiving waters and can actually provide many additional benefits to communities

1.2: HOW STORMWATER SHOULD BE MANAGED

Erosion and sediment control best management practices (BMPs) and post-construction stormwater control measures (SCMs) should be integrated into every development project.

Erosion and Sediment Control BMPs. Erosion control practices are those which aim to limit grading or soil exposure. Sediment control practices are utilized to capture sediment that is allowed to move across a construction site in stormwater runoff. Often it is easier and cheaper to maintain vegetative or other soil cover than to design and install sediment control devices. Temporary BMPs are used during active construction however, sometimes permanent BMPs are necessary to prevent erosion on-site after a project is completed. Appendix A details the various types of erosion and sediment control BMPs.

Post Construction SCMs. Post-construction SCMs (called water quality control, stormwater detention facilities, drainage facilities or water pollution abatement control facilities in the San Marcos Development Code) are designed and installed primarily to improve water quality and decrease stormwater runoff volume and peak flows after construction is completed. There are two categories of post-construction SCMs: traditional and low impact development (LID) measures. Traditional SCMs are typically structural facilities, while LID SCMs can be either structural (e.g. bioretention facility) or non-structural (e.g. disconnected impervious area). LID practices employ principles such as preserving and recreating natural landscape features, minimizing effective imperviousness, and employing processes of infiltration, filtration, storage, evaporation and detention of stormwater runoff. By implementing LID principles and practices, water can be managed in a way that reduces the impact of built areas and promotes the natural movement of water within an ecosystem or watershed. Applied on a broad scale, LID can maintain or restore a watershed's hydrologic and ecological functions. However, the most benefit can be achieved when traditional stormwater control measures and LID measures are both integrated into a development project through sustainable site design. Sustainable site design incorporates approaches to new and redevelopment projects which reduce impacts on watersheds by conserving natural areas, and better integrating stormwater management. This type

of integrated stormwater management employs principles such as preserving and recreating natural landscape features, minimizing effective imperviousness, and employing processes of infiltration, filtration, storage, evaporation and detention of stormwater runoff. Appendices B and C provide additional details and design guidance for both types of post-construction SCMs.

More information on utilizing sustainable site design practices can be found in Chapter 4 of this manual.

Integrating both traditional and LID stormwater control measures into new and redevelopment projects in San Marcos via sustainable site design can:

- Maintain and enhance the pre-development hydrologic regime of urban and developing watersheds;
- Work with nature to manage stormwater as close to its source as possible, treating stormwater as a resource rather than a waste product;
- Conserve and use valuable on-site features to protect water quality;
- Create functional and appealing site drainage that can increase property values;
- Improve air quality;
- Decrease impacts on stormwater infrastructure;
- Increase open space;
- Provide shading;
- Reduce flooding;
- Help recharge groundwater and the aquifer (where appropriate); and
- Reduce construction, maintenance and inspection costs.

The integration of LID SCMs allows for landscapes to serve multiple benefits. For example, a single parking lot island can break up pavement and provide safety zones for pedestrians but also manage stormwater, provide shade and reduce flooding during rain events. A regional stormwater management facility in a park can also serve as a playing field or habitat interpretation resources. Riparian buffers can be used as greenways for walking and biking. LID measures in the right-of-way can serve as shade for pedestrians and be used to calm traffic.

CHAPTER 2. WHAT STORMWATER MANAGEMENT IS REQUIRED

2.1: FEDERAL REGULATIONS 2:2

2.2: STATE PERMITS AND RULES 2:2
2.2.1: TEXAS MUNICIPAL SEPARATE STORM SEWER SYSTEM PERMIT 2:2
2.2.2: TEXAS CONSTRUCTION GENERAL PERMIT 2:2
2.2.3: TCEQ EDWARDS AQUIFER RULES 2:3

2.3: SAN MARCOS DEVELOPMENT CODE (SMDC) 2:3
2.3.1: SITE PLANNING 2:3
2.3.1.1: NATURAL DRAINAGE 2:3
2.3.1.2: CUT AND FILL STANDARDS 2:4
2.3.1.3: IMPERVIOUS COVER LIMITATIONS 2:4
2.3.1.4: TREE PRESERVATION 2:6
2.3.1.5: GENERAL DEVELOPMENT LIMITATIONS 2:6
2.3.2: EROSION AND SEDIMENT CONTROL STANDARDS 2:6
2.3.3: POST-CONSTRUCTION STORMWATER PERFORMANCE STANDARDS 2:7
2.3.3.1: STORMWATER QUALITY AND STREAM PROTECTION 2:7
2.3.3.2: FLOOD CONTROL 2:7
2.3.4: PERFORMANCE STANDARD WAIVERS 2:8

In addition to providing many water quality, flooding and quality of life benefits, stormwater management is required by several federal, state, and local regulations. Site designers are required to ensure that their designs meet each of the requirements for the region where construction is to occur.

2.1: FEDERAL REGULATIONS

Development projects may be required to comply with the United States Army Corp of Engineers Section 404 permit. If jurisdictional waters of the United States exist on the property to be reclaimed, the applicant shall provide the City with a copy of all reports, studies, plans and other data that are submitted to the U.S. Army corps of Engineers in conjunction with an application for approval of a Federal Section 404 permit.

No wetland or other significant natural features shall be reclaimed within a water quality zone or buffer zone established under Article 2 of Chapter 6 of the City's Development Code authorized under a mitigation plan, which demonstrates that reclamation or alteration of the floodplain will improve the water quality of the runoff and/or stabilize an existing area of erosion and will continue the maintenance of flood and flow characteristics of the waterway, and which provides for protection of altered areas during and after alteration and development of adjacent land.

The mitigation plan for reclamation or alteration of a buffer zone or other floodplain area may provide for restoration, creation, enhancement, or preservation of aquatic habitats to ensure that activities result in minimal adverse effects to the aquatic environment. The mitigation plan may, but need not be, the same as a mitigation plan required for obtaining a Section 404 permit.

2.2: STATE PERMITS AND RULES

The Texas Commission on Environmental Quality (TCEQ) administers several state-wide permits and regional rules which could apply to development projects in San Marcos.

2.2.1: TEXAS MUNICIPAL SEPARATE STORM SEWER SYSTEM PERMIT

The Clean Water Act amendments of 1987 established a framework for regulating stormwater discharges from municipal, industrial, and construction activities under the National Pollutant Discharge Elimination System program. Under the Clean Water Act, municipalities of sufficient size throughout the nation are issued a municipal separate storm sewer system (MS4) permit. In the state of Texas, the TCEQ issues these permits. The primary goal of the permit is to stop polluted discharges from entering the municipally-owned stormwater conveyance system and then discharging to local receiving waters. San Marcos is subject to the conditions of the Small MS4 General Permit, TPDES Permit No. TXR040000. The permit requires the City of San Marcos to meet multiple requirements, including:

- Create ordinances or other regulatory mechanisms to require erosion and sediment controls, as well as sanctions to ensure compliance, to the extent allowable under state and local law.
- Address stormwater runoff from new development and redeveloped sites that discharge into the small MS4 that disturb one acre or more, including projects that disturb less than one acre that are part of a larger common plan of development.

2.2.2: TEXAS CONSTRUCTION GENERAL PERMIT

Stormwater discharges associated with construction activities that disturb one acre or more are also regulated by the Texas Construction General Permit (CGP) Permit Number TXR150000. The CGP describes the requirements for stormwater pollution prevention plans as well as permit applicability and coverage, limitations on permit coverage, and authorizations to discharge stormwater. Find the link to the CGP in the Resources section.

2.2.3: TCEQ EDWARDS AQUIFER RULES

The TCEQ regulates stormwater discharges within the Edwards Aquifer Recharge and Contributing Zones (https://www.tceq.texas.gov/gis/edwards-viewer.html.)The regulations are focused on protection of groundwater quality and generally prohibit the infiltration of stormwater runoff except when the amounts are small (e.g., associated with swales) or when it is applied in a very distributed and redundant manner such as in retention/irrigation systems. Appendix B of this manual discusses the geology of these zones and how stormwater management in these areas should be designed in these areas. For example, LID measures must be lined over the Edwards Aquifer Recharge zone. Even though runoff volume is not reduced in lined systems, there is still value associated with filtering the stormwater and slowing its discharge. For stormwater management TCEQ requirements in the Edwards Aquifer Recharge and Contributing Zones, please see the Edwards Aquifer Technical Guidance manual (see Resources for link).

2.3: SAN MARCOS DEVELOPMENT CODE (SMDC)

Local requirements and standards for stormwater management are described in the San Marcos Development Code, Chapters 3 and 6. These standards apply to new development or redevelopment within the City limits and within the City's Extraterritorial Jurisdiction (ETJ). An existing building or site may be repaired, maintained or modernized without providing additional environmental protections, provided there is no increase in gross floor area or improved site area. When there are additions or increases in floor area or improved site area up to 25 percent cumulatively, these environmental standards are applicable to the additional floor or site area only. When the additions or increases are above 25 percent the standards apply to the entire building or site (see Chapter 6, Article 1, Division 1, Section 6.1.1.1, A.4)

The only exceptions to this are: the clearing of underbrush and the maintenance or removal of individual trees on a parcel of land where development has already occurred; provided, however, that the clearing or removal is not for the purpose of construction; the hand clearing of underbrush and the trimming of trees necessary to allow sufficient access to the property for planning and engineering purposes; and agricultural activities or related maintenance (see Chapter 6, Article 1, Division 1, Section 6.1.1.1, B).

2.3.1: SITE PLANNING

The Development Code requires that applicants utilize specific sustainable site design techniques for new and redevelopments.

2.3.1.1: Natural Drainage

Chapter 6 of the Development Code requires that natural drainage patterns shall be preserved whenever possible and soil compaction avoided. Open surface drainage through grass-lined swales and leaving portions of a subdivision in an underdeveloped and natural state is preferred as well. Green streets utilizing drainage BMPs such as bioretention, pervious pavers, and bioswales shall be utilized whenever possible.

In addition, SMDC Chapter 3 prohibits diversion of stormwater away from natural channels except when the stormwater is returned to the natural channel prior to leaving the developer's property and (for watersheds larger than 20 acres) an analysis is conducted to confirm that the peak flow is not increased at the point that the water reenters the watercourse.

Natural channel design and other channel protection strategies are described further in Chapter 4 of this manual.

The Development Code discourages construction of enclosed storm sewers and impervious channel linings. If stormwater drainage systems and/or culverts are used, these systems must be designed to control sediment, neutralize contaminants and dissipate energy by the use of multiple smaller outlets, whenever practical, by locating discharges to maximize overland flow and by any other strategies that will accomplish the City's objectives.

> To encourage ecological preservation along the San Marcos River, stabilization of eroding creek banks is permitted in order to protect threatened property, but only as approved by appropriate Federal and State agencies and the Responsible Official. All these projects must be designed to stabilize existing conditions only. Excavation or filling along the San Marcos River is only allowed in accordance with Chapter 39 of the San Marcos City Code and with Section 6.1.2.4 of the Development Code.

2.3.1.2: Cut and Fill Standards

Cut and fill on slopes must be minimized and comply with the standards found in the Development Code Article 1, Division 2, Section 6.1.2.2. (An administrative adjustment may be allowed for a stormwater control measures or for a cut or fill of not more than eight feet. See Article 1, Division 2, Section 6.1.2.3.) Cut area surfaces and fill areas must be restored and stabilized in accordance with Appendix A of this manual.

2.3.1.3: Impervious Cover Limitations

Reducing the creation of impervious cover is one of the primary site design techniques for reducing stormwater impacts. The less impervious cover that is created, the stormwater runoff will be generated from the project post-construction. This also means that there will be less stormwater to be managed by the developer therefore lower capital costs associated with designing and building SCMs.

The San Marcos Municipal Code limits the amount of impervious cover (IC) on new and redevelopment projects based on three criteria—slope, zoning district and geographic location.

Details regarding how to determine impervious cover on a project site can be found in Section 3.2. of this manual.

2.3.1.3.1: Steep Slope IC Limitations

The maximum impervious cover allowed anywhere in the City on slopes of 15–25 percent gradient is 35 percent impervious cover and 20 percent impervious cover maximum on slopes of greater than 25 percent gradient.

2.3.1.3.2: Zoning Districts IC Limitations

The San Marcos Zoning Regulations (Chapter 4 of the Development Code) applies impervious cover restrictions based on zoning district.

2.3.1.3.3: Enhanced Protection IC Zone Limitations

The City has developed specific impervious cover standards for enhanced protection zones to protect water quality in these more sensitive areas throughout the City and its ETJ. There are three types of enhanced protection zones: water quality zones, buffer zones, and sensitive feature protection zones. These standards apply to development affecting any waterway including the Blanco and San Marcos Rivers located within the City or its Extraterritorial Jurisdiction unless otherwise stated in the SMDC and except as follows:

- Development effecting any waterway having a drainage basin of less than 50 acres outside the EARZ, Transition Zone, and Contributing Zone within the Transition Zone and 5 acres within the EARZ, Transition Zone, and Contributing Zone within the Transition Zone measured upstream from the proposed development; and
- The construction of barns or other accessory structures related to agricultural uses.

Water quality zones and buffer zones must be designated when a plat is required for a development and must be shown on all associated watershed protection plans, plats, site plan permits, and building plans. Unless required by the Responsible Official to be dedicated as a flowage easement and dedicated for public maintenance, water quality zones and buffer zones must be privately held and maintained.

A water quality zone needs to be established for each waterway per the requirements found at Section 6.2.2.1 of the SMDC and buffer zones also need to be established per Section 6.2.2.2.

A sensitive feature protection zone must be established around each sensitive feature in the Edwards Aquifer Recharge Zone, Edwards Aquifer Transition Zone, and Contributing Zone within the Edwards Aquifer Transition Zone unless an applicant submits an enhanced geologic assessment or an enhanced topographic information in accordance with Sections 6.3.2.1(D) or 6.2.2.3(C) of the SMDC. The required area of the zones around sensitive features is specified Section 6.2.2.3 of the SMDC.

2.3.1.3.3.1: Water Quality and Buffer Zone Impervious Cover Limitations

The impervious cover limitations vary based on whether the zones are inside or **outside** of the EARZ.

Section 6.2.3.2 describes requirements for water quality and buffer zones outside the Edwards Aquifer Recharge Zone as follows:

- No impervious cover is allowed in a water quality zone except for the following exceptions (see SMDC Section 6.2.3.5(B) for specific details): replacement of impervious cover subject to flood protection standards in Chapter 39 of the San Marcos City Code, arterial, residential and collector street crossings in accordance with specific requirements, utility line crossings that are in compliance with all City and TCEQ requirements, fences that do not obstruct or dam surface water flows, trails and related facilities, other than buildings, for walking, running, and non-motorized biking.
- For buffers with a gradient of less than 15 percent the maximum impervious cover is 30 percent. However, impervious cover limitations may exceed 30 percent in these buffers—except for the San Marcos River Corridor—based upon approval of a mitigation plan which demonstrates that the water quality impacts of the impervious cover within the buffer zone can be mitigated through utilization of water pollution abatement control facilities that incorporate best management practices for the entire development site. No impervious cover may be transferred to a buffer zone that exceeds the requirements of Article 2, Division 3 of the Development Code.
- The maximum impervious cover in buffer zones is reduced when steep slopes are present in accordance with the table below.

IMPERVIOUS COVER (MAX)			
SLOPES	SMRC	BUFFER ZONE NO MITIGATION	BUFFER ZONE WITH MITIGATION
< 15%	30%	30%	50%
15% - 25%	20%	20%	—
> 25%	10%	10%	—

Section 6.2.3.3 describes requirements for water quality and buffer zones **inside** the Edwards Aquifer Recharge Zone as follows:

- The only development or impervious cover allowed in a water quality zone is as follows (see SMDC Section 6.2.3.5(B) for specific details): replacement of impervious cover subject to flood protection standards in Chapter 39 of the San Marcos City Code, arterial, residential and collector street crossings in accordance with specific requirements; utility line crossings that are in compliance with all City and TCEQ requirements; and fences that do not obstruct or dam surface water flows, trails and related facilities, other than buildings, for walking, running, and non-motorized biking.
- The maximum impervious cover for buffers with a gradient of less than 20 percent is 10 percent. No impervious cover is allowed on steeps slopes over 20 percent. However, impervious cover may exceed these limitations, to the level shown in the below table, based upon approval of a mitigation plan which demonstrates that the water quality impacts of the impervious cover within the buffer zone can be mitigated through utilization of water pollution abatement control facilities that incorporate best management practices for the entire development site.

IMPERVIOUS COVER (MAX)		
SLOPES	BUFFER ZONE NO MITIGATION	BUFFER ZONE WITH MITIGATION
< 20%	10%	20%
≥ 20%	0%	10%

2.3.1.3.3.2: Sensitive Feature Protection Zone Impervious Cover Limitation

No development or impervious cover is allowed within a sensitive feature protection zone except for fences that do not obstruct surface water flows and pervious trails and other facilities, other than buildings, for walking, running, or non-motorized biking. (Decomposed granite is not considered a pervious trail surface.)

2.3.1.3.4: EARZ IC Limitations

The total of all impervious cover that may be developed on a site in the Recharge Zone must not exceed the following percentages of the gross area of the site (based on the size of the site on October 8, 2001).

SIZE OF SITE	IMPERVIOUS COVER LIMIT
Up to and including three acres	40%
More than three acres and less than five acres	30%
Five acres or more	20%

2.3.1.4: Tree Preservation

Trees are an integral part of healthy aquifers and river corridors, managing stormwater runoff, controlling erosion and dust, abating noise, reducing building energy costs, enhancing property values, and providing wildlife habitat. The preservation of existing trees, replacement of lost trees and the planting of new trees should be part of any sustainable site design.

The City's general tree preservation requirements are found at SMDC Section 6.4.2.1 and they apply to all new development within the City and not within the ETJ, including development projects undertaken by the City of San Marcos.

2.3.1.5: General Development Limitations

The Development Code also imposes the following pollution prevention or source control limitations on development within water quality and buffer zones.

- New point discharges of runoff into water quality or buffer zones may be required to be dissipated to sheet flow conditions throughout the zone.
- The use of fertilizers and pesticides is prohibited within water quality or buffer zones.
- For development within the Edwards Aquifer Recharge Zone, the use of septic tanks, holding tanks, evapotranspiration units, cesspools or other private or individual sewage disposal systems will not be allowed in water quality or buffer zones.

2.3.2: EROSION AND SEDIMENT CONTROL STANDARDS

The erosion and sedimentation control techniques for construction activities detailed in the San Marcos MS4 Ordinance in Chapter 86, Article 8, Division 2 of the San Marcos City Code and Appendix A of this manual must be utilized to reduce environmental impacts from development and must be installed prior to commencing construction; be maintained during construction; and not be removed until vegetation is established and the construction area is stabilized. In addition, Section 3.9.1.5 of the Development Code requires that, during construction, all on-site drainage must be maintained so that discharges are not increased upstream or downstream of the site as compared to pre-project conditions unless full contained with a drainage easement or designated right-of-way.

In addition, temporary erosion and sedimentation controls are required to be installed and maintained for the following activities that are or may not be covered by any type of watershed protection plan in the Edwards Aquifer Recharge Zone, Transition Zone, Contributing Zone within the Transition Zone or San Marcos River Protection Zone:

- The construction or expansion of one single family home or accessory structure on a legally platted lot, or on an unsubdivided tract of land at least two acres in size, for which a legal description was contained in a deed recorded before March 1, 2000.
- The installation or maintenance of utility lines by a governmental entity.
- Landscaping activities involving more than 5,000 square feet of area of landscape installation.
- The resurfacing of existing paved roads, parking lots, sidewalks, or other development-related impervious surfaces.

All temporary erosion and sedimentation controls for projects within the Edwards Aquifer Recharge Zone also must meet the applicable standards and requirements of the TCEQ Edwards Aquifer Protection Program, Complying with the Edwards Aquifer Rules, Technical Guidance on Best Management Practices, RG – 348, Chapter 2 (link can be found in Resources).

> Appendix A includes a list of highly erodible soils in the San Marcos Area.

2.3.3: POST-CONSTRUCTION STORMWATER PERFORMANCE STANDARDS

The San Marcos Municipal Code requires that post-construction SCMs be designed to meet two types of performance standards: stormwater quality/stream protection and flood control. Stormwater quality and stream protection standards can be found in Chapter 6, Article 1 Division 4 of the Municipal Code and in flood control performance standards are located in Chapter 3, Article 9.

While both traditional and LID SCMs are allowed to meet the following standards, the use of sustainable site design and LID SCMs can be used to reduce peak flow rates and reduce or eliminate the need for detention onsite when design in accordance with this manual (SMDC, Section 3.9.1.1.F.3.)

2.3.3.1: Stormwater Quality and Stream Protection

The Development Code requires projects retain or detain the calculated Water Quality Volume (WQV) on-site and design SCMs to have a drawdown time of 48 hours. In addition, the WQV must be treated onsite to meet total suspended solids (TSS) loading reductions. The design storm and treatments levels for the WQV are specific to location (Table 2.1).

TABLE 2.1 WATER QUALITY VOLUME AND TREATMENT LEVEL TABLE

LOCATION	WQV AMOUNT DESIGN RAINFALL	WQV TREATMENT LEVEL
Edwards Aquifer Recharge Zone	1.60"	89%
Edwards Aquifer Transition Zone and Contributing Zone within the Transition Zone	1.25"	85%
San Marcos River Protection Zone	1.25"	80%
San Marcos River Corridor	1.60"	80%

Single Family Home Exception:
Exceptions to stormwater quality and stream protection requirements include development applications proposing solely the construction or expansion of a single-family home as long as the development includes disconnected impervious cover and provides treatment through vegetative filter strips or similar means.

2.3.3.2: Flood Control

Chapter 3, Article 9 of the Development Code describes the flood control requirements for new and redevelopment projects. Post-construction SCMs must be designed and constructed so that the rate of runoff from a site after construction shall be equal to or less than the runoff prior to construction for the two, ten, twenty-five, and one-hundred-year storm frequencies.

Additional details about required sizing and other SCM design standards can be found in Section 3 of this manual.

2.3.4: PERFORMANCE STANDARD WAIVERS

The City allows for the waiver of these standards in certain areas of the City as long as certain criteria are met.

The stormwater quality/stream protection standards may be waived for some properties in the City's Urban Stormwater Management District as shown in Figure 2.1 San Marcos Urban District Map

This waiver is only possible if it is demonstrated through drainage analysis that the properties cause no adverse impacts and a payment is made into the Stormwater Management Fund in accordance with Section 6.1.1.3 of the Development Code.

Flood control requirements found in Chapter 3, Article 9 may be waived under two circumstances: for non-residential small site permits that demonstrate no adverse impacts and for other projects in the City's High Intensity Zones (established in the Preferred Scenario Map of the Comprehensive Plan—see link in Resources) if no adverse impacts are demonstrated and a payment is made into the Stormwater Management Fund.



FIGURE 2.1 SAN MARCOS URBAN DISTRICT MAP

CHAPTER 3. HOW TO MANAGE STORMWATER

3.1: SITE PLANNING/SCM SELECTION 3:4
3.1.1: PHASE I – SITE ASSESSMENT 3:4
3.1.1.1: PHASE I-A: IDENTIFY REGULATORY REQUIREMENTS 3:4
3.1.1.2: PHASE 1-B: CONDUCT HYDROLOGIC AND GEOTECHNICAL SURVEY 3:5
3.1.1.3: PHASE 1-C: PROTECT NATURAL FEATURES AND KEY HYDROLOGIC AREAS 3:5
3.1.2: PHASE II – PRELIMINARY DESIGN 3:6
3.1.2.1: PHASE II-A: USE DRAINAGE AND HYDROLOGY AS DESIGN ELEMENTS 3:6
3.1.2.2: PHASE II-B: ESTABLISH CLEARING AND GRADING LIMITS 3:7
3.1.2.3: PHASE II-C: REDUCE/MINIMIZE IMPERVIOUS COVER 3:7
3.1.2.4: PHASE II-D: DETERMINE APPROPRIATE SCMS 3:9
3.1.3: PHASE III – FINAL DESIGN 3:10
3.1.3.1: PHASE III-A SIZE SCMS 3:10
3.1.3.2: PHASE III-B: DEVELOP FINAL DESIGN 3:10

3.2: DETERMINATION OF STORMWATER RUNOFF 3:11
3.2.1: HYDROLOGY 3:11
3.2.1.1: EFFECTS OF URBANIZATION 3:11
3.2.1.2: DESIGN ASSUMPTIONS FOR STORM RUNOFF ANALYSIS 3:11
3.2.1.3: METHOD OF ANALYSIS 3:12
3.2.1.4: RATIONAL METHOD 3:12
3.2.1.5: RUNOFF COEFFICIENT (C) 3:13
3.2.1.6: TIME OF CONCENTRATION 3:14
3.2.1.7: DRAINAGE AREA (A) 3:18
3.2.1.8: THE SOIL CONSERVATION SERVICE METHOD FOR CALCULATION OF PEAK FLOWS 3:19
3.2.1.9: PROBABLE MAXIMUM STORM/FLOOD DEVELOPMENT 3:26
3.2.1.10: IMPERVIOUS COVER DETERMINATION 3:27
3.2.2: CALCULATION OF WATER QUALITY VOLUME 3:28
3.2.2.1: CALCULATING THE TSS LOAD REDUCTION AND WATER QUALITY VOLUME FOR TSS TREATMENT 3:28
3.2.2.2: CALCULATE WATER QUALITY VOLUME BASED ON DESIGN RAINFALL 3:31
3.2.2.3: EXAMPLE DEVELOPMENT SITE CALCULATIONS 3:31
3.2.3: INTEGRATED STORMWATER MANAGEMENT 3:34
3.2.3.1: PERFORMANCE CRITERIA FOR SWM PONDS 3:34
3.2.3.2: DETENTION POND STORAGE DETERMINATION 3:35
3.2.3.3: OUTLET STRUCTURE DESIGN 3:35

3.3: INFRASTRUCTURE AND RIGHT OF WAY 3:36
3.3.1: STREET DRAINAGE 3:36
3.3.2: STREET FLOW 3:36
3.3.2.1: GENERAL 3:36

3.3.2.2: PERMISSIBLE SPREAD OF WATER 3:37
3.3.2.3: DESIGN METHOD 3:37
3.3.3: INLETS 3:40
3.3.3.1: INLET TYPES 3:40
3.3.3.2: DESIGN GUIDELINES 3:40
3.3.3.3: MATERIALS AND SPECIFICATIONS 3:45
3.3.4: STORM DRAIN SYSTEMS 3:45
3.3.4.1: INTRODUCTION 3:45
3.3.4.2: HYDRAULICS OF STORM DRAINAGE SYSTEMS 3:46
3.3.4.3: DESIGN GUIDELINES 3:53
3.3.4.4: MAINTENANCE CONSIDERATIONS 3:55
3.3.5: OPEN CHANNELS 3:55
3.3.5.1: GENERAL 3:55
3.3.5.2: OPEN CHANNEL HYDRAULICS 3:56
3.3.5.3: MANNING'S ROUGHNESS COEFFICIENTS 3:57
3.3.5.4: DESIGN REQUIREMENTS 3:59
3.3.5.5: CHANNEL DROP STRUCTURES 3:60
3.3.5.6: ENERGY DISSIPATERS 3:61
3.3.5.7: STRUCTURE AESTHETICS 3:62
3.3.6: BRIDGES AND CULVERTS 3:63
3.3.6.1: GENERAL 3:63
3.3.6.2: CULVERT HEADWALLS AND ENDWALLS 3:63
3.3.6.3: CULVERT DISCHARGE VELOCITIES 3:65
3.3.6.4: SELECTION OF CULVERT SIZE AND FLOW CLASSIFICATION 3:66
3.3.6.5: HYDRAULIC CONSIDERATIONS IN BRIDGE DESIGN 3:67



CONSTRUCTED WETLAND IN SAN ANTONIO, TEXAS.

Integrated Stormwater Management (ISM) relies upon evaluating the needs for preventing damage both onsite and throughout the watershed from rainfall events ranging between a small shower that barely produces runoff to the 100-year event. The frequent events which create the majority of runoff, carry the majority of pollution, and cause the majority of erosion issues at outfalls and in natural channels are managed by Stormwater Control Measures (SCMs). The large infrequent events (5-yr, 25-yr, up to 100-yr) are managed by detention and conveyance practices that are sized using Hydrologic and Hydraulic Modeling programs such as HEC-HMS and HEC-RAS. This chapter describes the fundamentals of how SCMs and detention/retention are sized concurrently to achieve ISM goals and reduce project costs. SMDC Article 3.9.1.1.F.3 states

Low impact development practices can be used to reduce peak flow rates to reduce or eliminate detention requirements when designed in accordance with the Stormwater Technical Manual criteria.

This chapter provides the technical criteria necessary to prepare a compliant design.

3.1: SITE PLANNING/SCM SELECTION

Sustainable site design works best when incorporated early in the site planning process. When initially laying out the site, the designer should evaluate where traditional and LID SCMs would be best utlized. Performing this evaluation concurrent with the design will reduce the environmental footprint of a site, while retaining and enhancing the owner or developer's purpose and vision. Several strategies that can be incorporated with sustainable site design include:

- Prevent stormwater impacts rather than having to mitigate for them;
- Manage stormwater (quantity and quality) as close to the source as possible and minimize the use of large or regional collection and conveyance;
- Preserve natural areas, healthy soils, native vegetation and reduce the impact on watershed hydrology;
- Use natural drainage pathways as a framework for site design to reduce the cost of construction and infrastructure.
- Provide stream protection through managing runoff volumes from frequent rainfall events that occur every year.
- Reduce soil compaction during construction to maintain infiltration capacities of the soil;
- On disturbed sites, to increase the organic content of the soil to maximize its water retention capacity and improve success of re-vegetation;
- Minimize the amount of disturbance to existing, mature stands of vegetation;
- Utilize simple, non-structural methods for stormwater management that are lower cost and lower maintenance than structural controls;
- Reduce capacity requirements for detention SCMs by incorporating LID SCMs early in the planning process;
- Create a multifunctional landscape which considers construction and maintenance implications; and
- Use appropriate plant species and communities for the eco-region and the designed media.

The site planning process can be broken down into three phases; site assessment, preliminary site design, and final design.

3.1.1: PHASE I – SITE ASSESSMENT

The first phase of site planning is composed of the site assessment. Steps 1 through 3 below define the site assessment process.

3.1.1.1: Phase I-A: Identify Regulatory Requirements

Stormwater management SCMs must be consistent with the applicable federal, state, and local regulations. A submittal requirement checklist for drainage has been prepared by the City of San Marcos and is available online. Please contact COSM Engineering and Capitol Improvement Department for the latest version of the checklist. Each of the items below should be evaluated by the site planning and design team.

Identify applicable zoning land use, subdivision, and other local regulations. San Marcos' zoning requirements can be found online through the Planning and Development Services website.

Identify setbacks, easements and utilities. Setbacks are defined by the San Marcos Municipal Code. Easements that could be present are a road or sidewalk (right-of-way) easement; a public utility easement that allows a utility to run gas, water, sewer or power lines through a private property; or a railway easement; drainage and flood maintenance easements or SCM maintenance areas. Local utilities departments (e.g., electric, wastewater) should be consulted to determine whether utilities are above or below ground and the required distance that site disturbance should be maintained from any utilities present.

Identify applicable local environmental regulations. Chapter 6 of the San Marcos Development Code (SMDC) specifies the types of environmental regulations which are applicable to various parts of the City.

Identify areas of contaminated soils at brownfield sites. A brownfield is a property, the expansion, redevelopment, or reuse of which may be complicated by the presence or potential presence of a hazardous substance, pollutant, or contaminant. Contaminated areas may need to be capped with an impervious surface and/or avoided for the purposes of SCM infiltration.

After all of the site assessment steps have been completed, all of the physical requirements (setbacks, easements, etc.) should be displayed on the site plan (Figure 3.1). By doing so, designers can visualize which parts of the site should not be used, and which are available for construction and for stormwater management facilities.

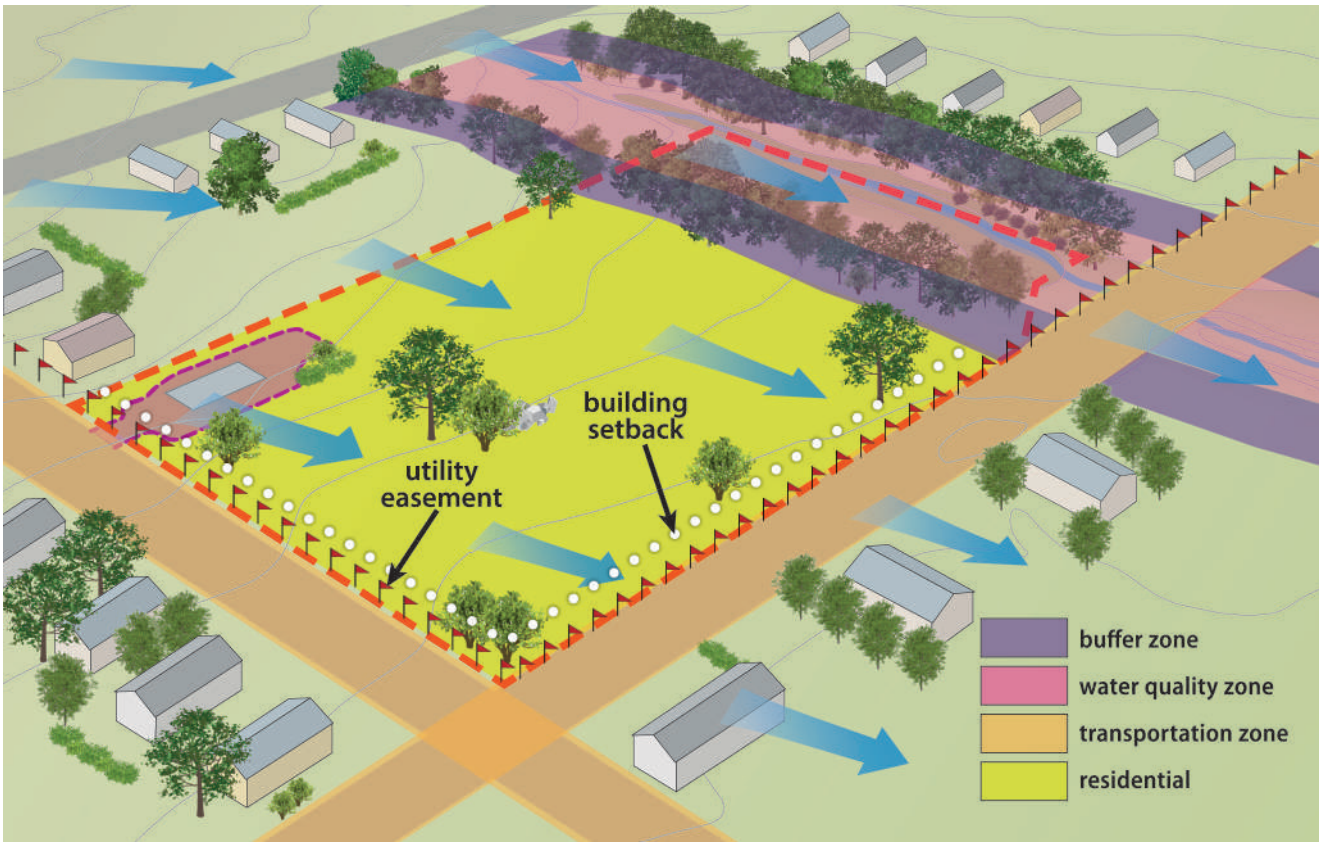


FIGURE 3.1 IDENTIFY APPLICABLE ZONING REQUIREMENTS, UTILITY EASEMENTS, AND SITE SETBACKS.

3.1.1.2: Phase 1-B: Conduct Hydrologic and Geotechnical Survey

Identifying hydrologic and geotechnical features is critical to determining which SCMs can be used at a site and the best locations. However, to fully examine a site and its ecological conditions which will influence SCM design, more in-depth site analysis should be done by geotechnical professionals, hydrologists, ecologists, biologists and/or other professionals with site assessment experience in order to test infiltration rates, asses soil type and quality, and be able to properly identify existing vegetation. There are several components to completing the hydrologic and geotechnical investigations.

Conduct geotechnical survey, including drainage characteristics, hydrologic flow paths, and soil infiltration rates. A soil evaluation including soil infiltration testing is intended to identify and protect soils that provide greater infiltration as potential locations for LID SCMs. In addition, natural drainage characteristics and hydrologic flow paths should be identified.

Determine the eco-region the project is in and what specific considerations for LID practice which may be necessary. The eco-regions of the San Marcos area can help inform LID design decisions by placing in context, hydrology, geology, soil types, rainfall patterns, and plant communities. Consideration of the different landscape characteristics assists in items as preliminary as SCM placement or choice, to farther reaching items such as long-term maintenance. Eco-regions and their impact on SCM selection and design are discussed in detail in Appendix B.

3.1.1.3: Phase 1-C: Protect Natural Features and Key Hydrologic Areas

Preserving the hydrology of a site includes reviewing existing hydrologic flow paths and protecting infiltrating soils. Natural hydrologic functions of the site should be preserved to the maximum extent practicable. Incorporating these techniques into a site design results in a hydrologically functional landscape that can slow runoff rates, protect receiving waters, and reduce the total volume of runoff.

Identify natural areas to be conserved or restored. Site planners and designers should consider how to use existing natural features of the site in an effort to retain natural hydrologic functions and potentially reduce the cost of drainage infrastructure.

Protect areas of natural hydrologic function. Hydrologic site features should be preserved to the maximum extent practicable. Hydrologic site features that should be protected include riparian areas, floodplains, stream buffers, and wetlands. Preservation of natural features includes techniques to foster the identification and preservation of natural areas that can be used in the protection of water resources.

Establish Buffers. A riparian buffer is a special type of natural conservation area along a stream, wetland or shoreline where development is restricted or prohibited. Further information on the buffer zone boundaries can be found in Section §6.2.2.2 of the SMDC.

Protect possible areas for infiltration. Existing healthy soils or media typically serve as essential elements for achieving LID functions and providing source control for stormwater treatment. LID measures in areas that naturally infiltrate will best preserve the natural balance between runoff and infiltration.

Identify areas with steep slopes. Development on slopes with a grade of 15% (7:1) or greater should be avoided if possible to limit soil loss, erosion, excessive stormwater runoff, and the degradation of surface water. When developing a new site, SMDC Section §6.1.2.1.A, mandates that natural drainage patterns be preserved whenever possible.

With the conclusion of Phase I, the initial site assessment has been completed. The decisions made regarding LID and SCM practices during the site assessment process should be documented to ensure that if changes are required in future Phases II and III, the original design ideas are available for reference.

3.1.2: PHASE II – PRELIMINARY DESIGN

At the conclusion of Phase II of the site design, a preliminary site design will be completed. Following the steps outlined below will yield a site plan that incorporates many features of low impact development and includes stormwater control measures. The steps are often iterative and may need to be repeated several times for an optimal result.

3.1.2.1: Phase II-A: Use Drainage and Hydrology as Design Elements

Delineate drainage areas throughout the site. Drainage areas are defined by geographical terrain and man-made drainage systems. Drainage area delineations are critical to determining how much stormwater reaches a point and therefore how to size a stormwater control measure. Data utilized to delineate drainage areas can come from multiple sources, including:

- High accuracy LiDAR based contours, as generated by public or private agencies;
- On-the-ground topographic survey data;
- Historical topography maps, including USGS Quad. Maps for pre-development conditions;
- Roadway construction plans;
- Aerial photos;
- Underground infrastructure plans;
- As-built plans for a site.

Identify the spatial layout of the site using hydrologic flow paths as a feature. Natural hydrologic functions (e.g., flow paths) should be included as a fundamental component of the preliminary design. Naturally present functions should be retained, or if that is not an option, replicate natural functions with appropriate SCM placement.

Determine approximate conveyance and SCM locations. Natural hydrologic functions, including interception, depression storage, and infiltration, should be distributed throughout the site to the extent possible. In conserving predevelopment and retrofit hydrology, runoff volume, peak runoff rate, flow frequency and duration, and water quality control must be considered.

Spatial layout should use the natural landforms and hydrologic flow paths identified in Phase I-B as a major design element of the site. Common LID elements using that premise include designing open drainage systems to function as both treatment and conveyance devices. Impervious elements such as parking lots, roadways, and sidewalks can be designed on the existing contours to minimize effects on the natural hydrologic flow path (Figure 3.2).

3.1.2.2: Phase II-B: Establish Clearing and Grading Limits

Clearing and grading should be kept to the minimum amount required for construction in order to minimize total impacts to the site.

Define the limits of clearing and grading on the site plan. Limits of clearing and grading refer to the total site area that is to be developed, including all impervious and pervious areas.

Minimize disturbance to areas outside the limits of clearing and grading. Disturbance to areas that are designated as those outside clearing and grading limits should be minimized. Figure 3.3 illustrates the use of orange construction fencing to preserve the natural features, drainage pathways, and maintain infiltration on suitable soils at the example site as identified in previous steps.

3.1.2.3: Phase II-C: Reduce/Minimize Impervious Cover

One of the most effective LID measures is the reduction of overall impervious surfaces. Therefore, reducing the overall surface area for impervious areas is one of the most cost-effective methods of reducing surface runoff and required treatment.

Evaluate conceptual design to reduce impervious surfaces. Reducing the construction of impervious surfaces reduces the volume of stormwater runoff created. Minimizing runoff at a site also reduces the size of SCMs required. Contact the City of San

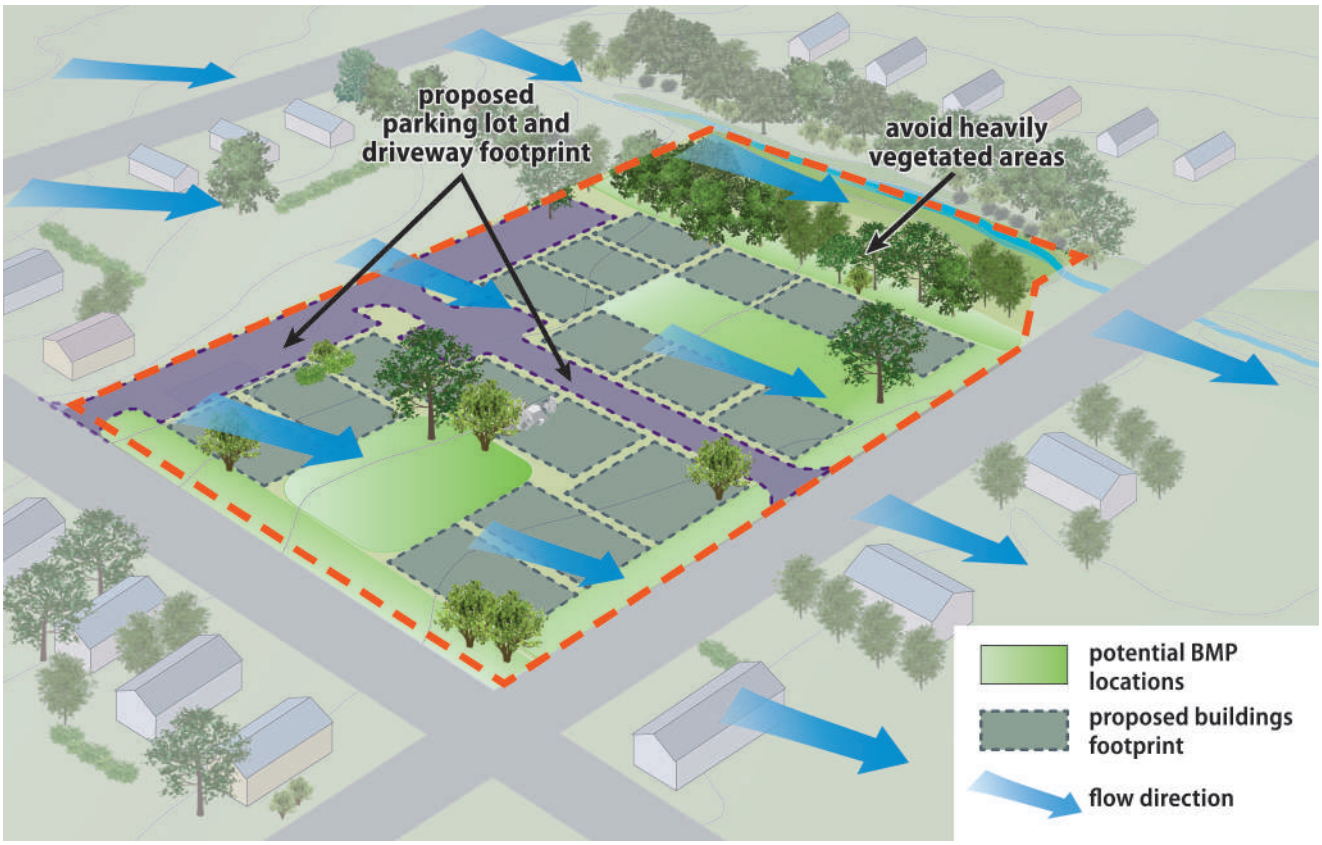


FIGURE 3.2 IDENTIFY IDEAL LOCATIONS FOR LID IMPLEMENTATION ACCORDING TO SITE CONDITIONS.

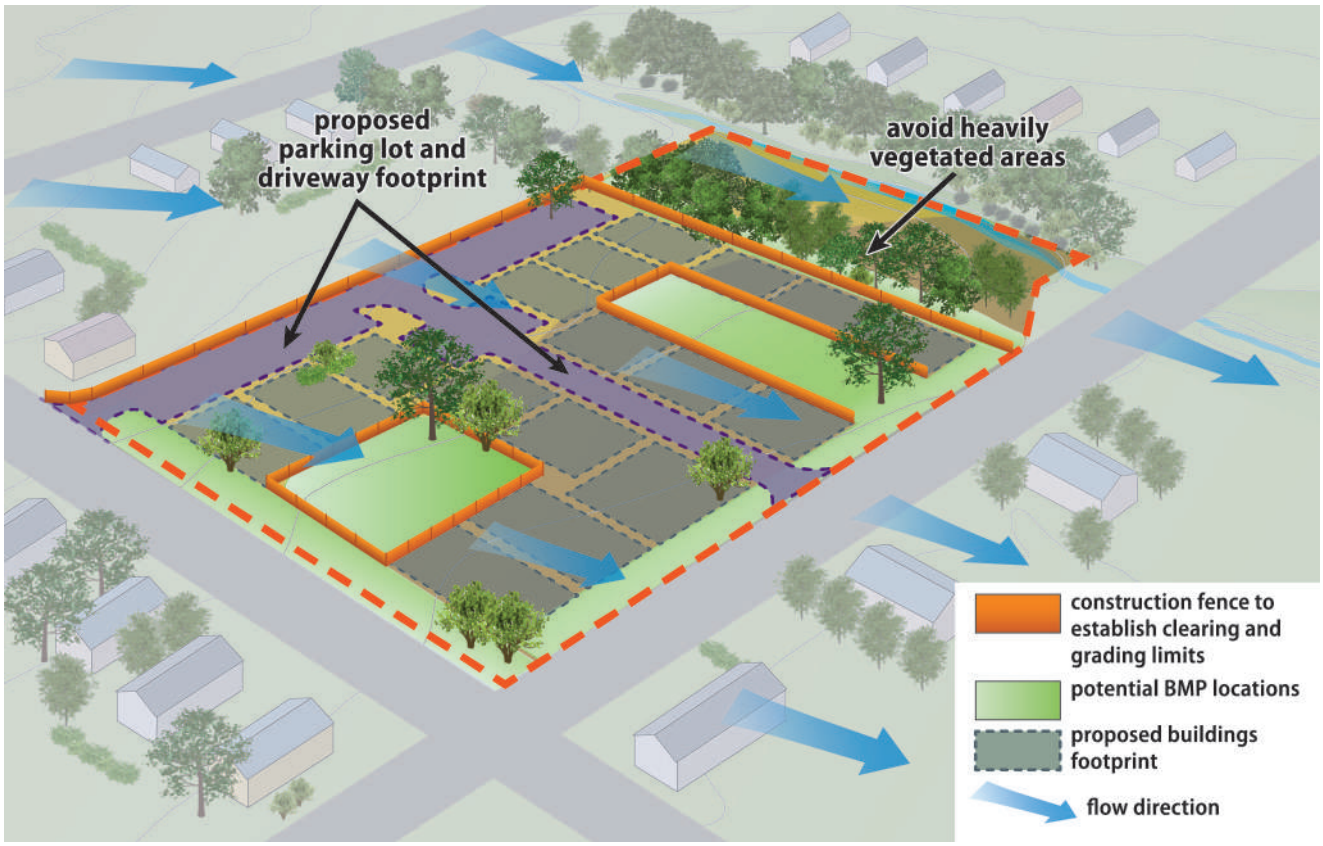


FIGURE 3.3 ESTABLISH GRADING ENVELOPE TO PROTECT NATURAL AREAS AND INFILTRATING SOILS.

Marcos regarding code allowances for the reduction of impervious cover such as reducing parking or increasing building height.

Investigate potential for impervious area disconnection. Diverting stormwater runoff from impervious areas such as rooftops and pavement to adjacent pervious areas is an effective way to infiltrate stormwater runoff and to reduce flow rates. This technique is referred to as impervious area disconnection.

The first step in disconnection is to identify the source of runoff and understand how it will be managed once disconnection occurs. By disconnecting impervious areas and directing the flow to infiltration basins or designated buffer areas, up to 70 percent of runoff that would contribute to stormwater runoff is infiltrated close to the source instead. Proper design will align pervious surfaces with building drainage to maximize opportunities for disconnection.

In addition to directing downspouts to vegetated areas, roof runoff may also be directed to cisterns and other rain barrels for later consumption, or even to depressed storage or other underground storage areas. Some design considerations include:

- Slowing down the water after it leaves the downspout if the volume and velocity is high;
- Keeping the disconnected runoff away (10' minimum) from other impervious surfaces to reduce the chance for re-connection;
- Not placing the disconnected runoff into a steep slope area which could cause erosion and concentrate flows; and
- Directing the runoff into features specifically designed to receive (and either store, soak, treat, or convey) this runoff.

Determine if conservation subdivision design is a viable option. Conservation design, also known as open space design or cluster development, includes laying out the elements of a development project in such a way that the site design takes advantage of a site's natural features, preserves the more sensitive areas, and identifies any site constraints and opportunities to prevent or reduce impacts.

Conservation subdivisions typically incorporate smaller lot sizes to reduce overall impervious cover while providing more undisturbed open space and protection of water resources. This approach concentrates structures and impervious surfaces in a compact area in one portion of the development site in exchange for providing open space and natural areas elsewhere on the site. Typically, smaller lots and/or nontraditional lot designs are used to cluster development and create more conservation areas on the site. Conservation developments have many benefits compared with conventional commercial developments or residential subdivisions. They can reduce:

- Impervious cover;
- Stormwater pollution:
- Construction costs; and
- The need for grading and landscaping, while providing for the conservation of natural areas.

Preservation of natural areas and conservation designs can help to preserve predevelopment hydrology of the site and aid in reducing stormwater runoff and pollutant load. Undisturbed vegetated areas also promote soil stabilization and provide for filtering and infiltration of runoff. Maintaining existing vegetation can be particularly beneficial to sites with floodplains, wetlands, stream banks, steep slopes, critical environmental features, or where erosion controls are difficult to establish, install, or maintain.

3.1.2.4: Phase II-D: Determine Appropriate SCMs

Stormwater control measures employ a number of processes: settling/sedimentation, filtration, sorption, photolysis, biological processes, and chemical processes for pollutant removal. In addition to pollutant removal, LID SCMs provide hydrologic controls by reducing peak flows and volume through processes of infiltration, evaporation, and storage and reproducing predevelopment hydrologic functions. Traditional SCMs primarily provide hydrologic controls via detention and retention of water.

Evaluating which LID SCM (or SCMs) are required on a site involves determining what the end goals and requirements are. There are several SCMs (such as bioretention facilities) that qualify as volume type SCMs and can be utilized to meet water quality volume and treatment standards. Other SCMs (such as vegetated filter strips and vegetated swales) help to improve water quality, but do not meet the City of San Marcos' volume requirements. These SCMs are typically used in tandem with volume-based SCMs to meet the total treatment requirements.

Determine which SCMs should be used to ensure multiple benefits are achieved. During SCM selection, it is important to consider the unit processes of a SCM to ensure that the management practice will provide the necessary benefits and avoid potential complications or failures.

Using multiple treatment processes either in individual or multiple SCMs is called a treatment train. Meeting targeted treatment objectives can usually be achieved using a series of LID SCMs in a treatment train. Treatment trains can often be designed along rights-of-way, in parking lots, underground, or incorporated into landscaped areas. LID site planning should result in a treatment train of LID strategies and SCMs to meet treatment and water quality goals. For further details on treatment train SCM implementation, see Appendix B: LID Practices.

A number of factors should be considered for choosing appropriate SCMs for a site. For example, the presence of group C or D soils on a site might preclude the use of an infiltration SCM or require the use of an underdrain into the design of infiltration SCMs (see Appendix B, Section 3.3). Additionally, the low level of precipitation and high evapotranspiration rates usually present in San Marcos would likely exclude the use of a SCM requiring a permanent pool, such as a stormwater wetland, because precipitation is not great enough to maintain a continual or permanent pool of water. Native vegetation, which is adapted to the local climate and soils, should be used for vegetated SCMs, as much as possible, when soils allow. If native soils are replaced with imported soils to improve infiltration, non-native noninvasive but drought-tolerant plants might be a desired choice. For information regarding appropriate vegetation, see the SARB Low Impact Development Manual (link provided in Appendix A.). Other geotechnical, site-specific considerations include the level of the underlying water table and bedrock, any existing infrastructure in retrofit designs, and the presence of areas of concern that exhibit soil and ground water contamination.

Account for contaminated soils on brownfields sites. Preparing brownfields for redevelopment often requires capping of contaminated soils, creating even larger impervious surfaces. The challenge for managing stormwater on brownfield sites is allowing this capping while mitigating the impervious surface conditions that can negatively impact local waterways. However, LID measures exist that can retain, treat and then release stormwater without it ever coming in contact with contaminated soils.

Ensure infiltration is managed properly within Edwards Aquifer Zones. In the recharge zone, direct infiltration of stormwater is

prohibited per the Texas Commission on Environmental Quality's (TCEQ) Edwards Rules. Thus, bioretention and porous pavement facilities must be designed with underdrains. In the contributing, transition, and artesian zones, infiltration is allowed and there are currently no regulations prohibiting infiltration. Ultimately, SCM designs in these areas will depend on the performance goals for the site.

Integrate SCMs into downtown in a way allowed by SmartCode. Incorporating LID into San Marcos' SmartCode transects can help improve water quality, offset the negative impacts of development, and protect the existing natural features such as rivers and creeks present in all transect zones. Most SCMs such as bioretention systems, are appropriate across all transects as the design and implementation of this practice can be modified to work across all land uses and types.

At the completion of Phase II, the site planning for the project is complete. At that point in the site planning process, the development area should be delineated and the approximate type and potential locations for appropriate SCMs should be identified. Appendix B provides substantial detail about SCM applicability and design requirements and can be referenced during the process. The preliminary plan should be documented in addition to the decisions that were made in developing the preliminary plan for future reference and to ensure that the LID planning concepts are carried through to project construction. After the preliminary design is completed, the final design is achieved through identifying the appropriate LID facility type and size for meeting stormwater management needs and requirements.

3.1.3: PHASE III – FINAL DESIGN

3.1.3.1: Phase III-A Size SCMs

Determine the appropriate SCM size using the SCM sizing tools. The level of control that is required for a site to achieve stormwater management goals can be determined through a site-specific hydrologic evaluation. The hydrologic evaluation is performed using hydrologic modeling and analysis techniques. A stepwise process is followed to conduct a hydrologic evaluation:

1. Delineate the watershed and subwatershed areas.
2. Define the WQ design storm (for determination of stormwater runoff and SCM sizing guidance, refer to Section 3.2).
3. Select the hydrologic and hydraulic methods of analysis to be used. Freely available and commercial packages can be used to model SCMs but require approval from the Director of Engineering.
4. Collect hydrologic data for predevelopment conditions.
5. Using hydrologic models, evaluate predevelopment (baseline) conditions.
6. Using hydrologic models, evaluate the post development conditions and the hydrologic benefits from decreasing and disconnecting impervious areas, and compare the benefits to baseline conditions.
7. Using hydrologic model, evaluate the hydrologic control from implementation of one or more LID SCMs.

For detailed design steps, refer to Chapter 3, Appendix B: LID Measures and Appendix C: Stormwater Detention Guidance.

3.1.3.2: Phase III-B: Develop Final Design

Following iterations of **Phases II A-D and SCM sizing in Phase III-A**, additional conventional stormwater control measures (e.g., detention, retention) can be added to the site design as necessary to meet site drainage and other requirements. Combining stormwater quality and conventional SCMs (detention, retention, etc.), is called Integrated Stormwater Management which is discussed in Section 3.2.3. Review of the earlier documentation of decisions made during planning phases should also be conducted to ensure that the intent of the LID planning principles were carried through to the final design. The iterative review process can result in more or less area required for stormwater management.

The following items should be completed during Phase III-B:

- Integrate traditional and LID stormwater management controls.
- Verify geotechnical and drainage requirements have been met.
- Complete SCM design.
- Complete site plans.

The key to finalizing the SCM design process is to consult the design instructions for the selected SCM types in Appendix B and C of this manual. By following those instructions, the designer can develop final details, plan views, cross sections, profiles, and notes. The example in III-B-2 illustrates the final sizing and site layout for the properly sited and sized water quality SCMs. For expanded explanation of Site Planning and SCM selection, refer to Appendix B.

3.2: DETERMINATION OF STORMWATER RUNOFF

3.2.1: HYDROLOGY

If continuous records of the amounts of runoff from urban areas were as readily available as records of precipitation, they would provide the best source of data on which to base the design of storm drainage and flood protection systems. Unfortunately, such records are available in very few areas in sufficient quantity to permit an accurate prediction of stormwater runoff. The accepted practice, therefore, is to relate runoff to rainfall, thereby providing a means for predicting the amount of runoff to be expected from urban watersheds at given recurrence intervals.

Numerous methods of rainfall runoff computations are available on which the design of storm drainage systems may be based. The method chosen is dependent upon the engineer's technical familiarity and the size of the area to be analyzed. For the method chosen the engineer will be responsible for making reasonable assumptions as to the development characteristics of the study area.

3.2.1.1: Effects of Urbanization

It has long been recognized that urban development has a pronounced effect on the rate of runoff from a given rainfall event. The hydraulic efficiency of a drainage area is generally increased as a byproduct of urbanization which in effect reduces the storage capacity of a watershed. This reduction of a watershed's storage capacity is a direct result of the elimination of pervious surfaces, small ponds, and holding areas. This comes about by the grading and paving of building sites, streets, drives, parking lots, and sidewalks and by construction of buildings and other facilities characteristic of urban development. The result of the improved hydraulic efficiency is illustrated graphically in Figure 3.4, which is a plot of the runoff rate versus time for the same storm with two different stages of watershed development.

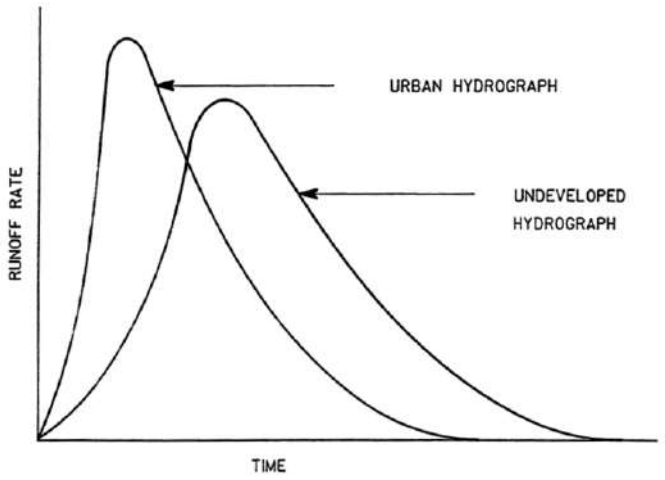


FIGURE 3.4 RUNOFF RATE VERSUS TIME HYDROGRAPH.
Source: City of Austin, Watershed Management Division

3.2.1.2: Design Assumptions for Storm Runoff Analysis

a. When analyzing an area for channel or storm drain design purposes, urbanization of the full watershed without stormwater detention facilities shall be assumed (except as noted in (D) below). Zoning maps, future land use maps, and master plans should be used as aids in establishing the anticipated surface character of the ultimate development. The selection of design runoff coefficients and/or percent impervious cover factors are explained in the following discussions of runoff calculation.

b. An exception to (A) above may be granted per the Director of Engineering, as long as the design is in conformance with the purpose of the Land Development Code §3.9.1.2.

c. In designing a storm drain system, full development of adjoining and interior tracts without detention shall be assumed.

d. In the event the engineer desires to incorporate the flow reduction benefits of existing upstream detention facilities, the following field investigations and hydrologic analysis will be required: (Please note that under no circumstances will the previously approved construction plans of the upstream detention facilities suffice as an adequate analysis. While the responsibility of the individual site or

subdivision plans rests with the engineer of record, any subsequent engineering analysis must ensure that all the incorporated detention facilities work collectively.)

a. A field survey of the existing physical characteristics of both the outlet structure and ponding volume. Any departure from the original engineer's design must be accounted for. If a dual use for the detention facility exists, (e.g., storage of equipment) then this too should be accounted for.

b. A comprehensive hydrologic analysis which simulates the flow attenuation produced by the existing detention facility in the upstream contributing area. This should not be limited to a linear additive analysis but rather a network of hydrographs which considers incremental timing of discharge and potential coincidence of outlet peaks.

3.2.1.3: Method of Analysis

Numerous methods of rainfall-runoff computation are available on which the design of storm drainage and flood control systems may be based. The Rational Method is accepted as adequate for drainage areas totaling 100 acres or less. The National Resources Conservation Service (formerly the Soil Conservation Service) hydrologic methods (available in the NRCS TR-20, and the US Army Corps of Engineers' Hydrologic Engineering Center's HEC-HMS program) should be used for drainage areas larger than 100 acres but may also be used for drainage areas of any size. The method of analysis must remain consistent when drainage areas are combined and the method which applies to the largest combined drainage area should be used unless the situation requires the use of NRCS hydrologic methods (i.e., a detention facility connected to a downstream storm drainage system). The engineer can use other methods but must have their acceptability approved by the Director of Engineering.

3.2.1.4: Rational Method

The Rational Method is based on the direct relationship between rainfall and runoff, and is expressed by the following equation:

$$Q_p = CiA \qquad \text{[Equation 3.1]}$$

Where:

Q_p is defined as the peak runoff in cubic feet per second. Actually, Q_p is in units of acre-inches per hour. Since this rate of acre-in/hr differs from cubic feet per second by less than one (1) percent (1 acre-in/hr = 1.008 cfs), the more common units of cfs are used.

C is the composite coefficient of runoff representing the ratio of peak runoff rate "Q_p" to average rainfall intensity rate "i" for the soil types and land uses characteristic of the contributing drainage area. See Section 3.2.1.5 for determining the proper coefficient.

"i" is the average intensity of rainfall in inches per hour for a period of time equal to the time of concentration (T_c) for the drainage area to the design point under consideration.

"A" is the area in acres contributing runoff to the point of design.

The following basic assumptions are associated with the Rational Method:

a. The storm duration is equal to the time of concentration.

b. The computed peak rate of runoff at the design point is a function of the average rainfall rate over a duration equal to the time of concentration at that point.

c. The return period or frequency of the computed peak flow is the same as that for the design storm.

d. The necessary basin characteristics can be identified and the runoff coefficient does not vary during a storm.

e. Rainfall intensity is constant during the storm duration and spatially uniform for the area under analysis.

f. The maximum rate of discharge at the point of design will occur when the entire area above the point of design is contributing runoff.

3.2.1.5: Runoff Coefficient (C)

The proportion of the total rainfall that will reach the drainage system depends on the surface vegetation condition, soil type, imperviousness of the surface, land slope and ponding characteristics of the area. Impervious surfaces, such as asphalt pavements and roofs of buildings, will be subject to approximately 100 percent runoff (regardless of the slope). On-site inspections and aerial photographs may prove valuable in estimating the nature of the surfaces within the drainage area.

It should be noted that the runoff coefficient "C" is the Rational Method variable which is least amenable to precise determination. A reasonable coefficient must be chosen to represent the integrated effects of infiltration, surface ponding, evaporation, flow routing and interception, all of which affect the time distribution and peak rate of runoff.

It is often desirable to develop a composite runoff coefficient based upon the percentages of different types of surfaces in the drainage area. This procedure is often applied to typical "sample blocks" as a guide to selection of reasonable values of the coefficient for an entire area. Suggested coefficients with respect to specific surface types are given in Table 3.1. "C" values for developed conditions should be based on maximum allowable impervious cover as listed in the City's zoning and land development codes.

Table 3.1 Rational Method Runoff Coefficients for Composite Analysis

Runoff Coefficient (C)							
	Return Period						
Character of Surface	2 Years	5 Years	10 Years	25 Years	50 Years	100 Years	500 Years
Developed							
Asphaltic	0.73	0.77	0.81	0.86	0.90	0.95	1.00
Concrete	0.75	0.80	0.83	0.88	0.92	0.97	1.00
Grass Areas (Lawns, Parks, etc.)							
Poor Condition*							
Flat, 0-2%	0.32	0.34	0.37	0.40	0.44	0.47	0.58
Average, 2-7%	0.37	0.40	0.43	0.46	0.49	0.53	0.61
Steep, over 7%	0.40	0.43	0.45	0.49	0.52	0.55	0.62
Fair Condition**							
Flat, 0-2%	0.25	0.28	0.30	0.34	0.37	0.41	0.53
Average, 2-7%	0.33	0.36	0.38	0.42	0.45	0.49	0.58
Steep, over 7%	0.37	0.40	0.42	0.46	0.49	0.53	0.60
Good Condition***							
Flat, 0-2%	0.21	0.23	0.25	0.29	0.32	0.36	0.49
Average, 2-7%	0.29	0.32	0.35	0.39	0.42	0.46	0.56
Steep, over 7%	0.34	0.37	0.40	0.44	0.47	0.51	0.58
Undeveloped							
Cultivated							
Flat, 0-2%	0.31	0.34	0.36	0.40	0.43	0.47	0.57
Average, 2-7%	0.35	0.38	0.41	0.44	0.48	0.51	0.60
Steep, over 7%	0.39	0.42	0.44	0.48	0.51	0.54	0.61

TABLE 3.1 RATIONAL METHOD RUNOFF COEFFICIENTS FOR COMPOSITE ANALYSIS

RUNOFF COEFFICIENT (C)							
	RETURN PERIOD						
CHARACTER OF SURFACE	2 YEARS	5 YEARS	10 YEARS	25 YEARS	50 YEARS	100 YEARS	500 YEARS
Pasture/Range							
Flat, 0-2%	0.25	0.28	0.30	0.34	0.37	0.41	0.53
Average, 2-7%	0.33	0.36	0.38	0.42	0.45	0.49	0.58
Steep, over 7%	0.37	0.40	0.42	0.46	0.49	0.53	0.60
Forest/Woodlands							
Flat, 0-7%	0.22	0.25	0.28	0.31	0.35	0.39	0.48
Average, 2-7%	0.31	0.34	0.36	0.40	0.43	0.47	0.56
Steep, over 7%	0.35	0.39	0.41	0.45	0.48	0.52	0.58

Assumptions

1. Composite "C" value for developed conditions (CDEV) is: CDEV = IC1 + (1-I)C2

   Where:

   I = Impervious cover, percent

   C1 = "C" value for impervious cover

   C2 = "C" value for pervious area (grass, lawns, parks, etc.)

2. For maximum allowable impervious coverage values for various land use types, refer to the City of San Marcos Land Development Code.

*Grass cover less than 50 percent of the area.

**Grass cover on 50 to 75 percent of the area.

***Grass cover larger than 75 percent of the area.

Source: 1. Rossmiller, R.L. "The Rational Formula Revisited."
2. City of Austin, Watershed Engineering Division

3.2.1.6: Time of Concentration

The time of concentration is the time for surface runoff to flow from the most remote point in the watershed to the point of interest. This applies to the most remote point in time, not necessarily the most remote point in distance. Runoff from a drainage area usually reaches a peak at the time when the entire area is contributing. However, runoff may reach a peak prior to the time the entire drainage area is contributing if the area is irregularly shaped or if land use characteristics differ significantly within the area. Sound engineering judgment should be used to determine a flow path representative of the drainage area and in the subsequent calculation of the time of concentration. The time of concentration to any point in a storm drainage system is a combination of the sheet flow (overland), the shallow concentrated flow and the channel flow, which may include storm drains. The minimum time of concentration for any drainage area shall be 5 minutes. Additionally, the minimum slope used for calculation of sheet and shallow flow travel time components should be 0.005 feet per foot (0.5%). The preferred procedure for estimating time of concentration is the NRCS method as described in NRCS's Technical Release 55 (TR-55). This method is outlined below. The overall time of concentration is calculated as the sum of the sheet, shallow concentrated and channel flow travel times. Note that there may be multiple shallow concentrated and channel segments depending on the nature of the flow path.

$$T_C = T_{t(sheet)} + T_{t(shallow\ concentrated)} + T_{t(channel)} \qquad \text{[Equation 3.2]}$$

3.2.1.6.1: Sheet Flow

Sheet flow is shallow flow over land surfaces, which usually occurs in the headwaters of streams. The engineer should realize that sheet flow occurs for only very short distances, especially in urbanized conditions. Sheet flow for both natural (undeveloped) and developed conditions should be limited to a maximum of 200 feet. Sheet flow for developed conditions should be based on the actual pavement or grass conditions for areas that are already developed and should be representative of the anticipated land use within the headwater area in the case of currently undeveloped areas. In a typical residential subdivision, sheet flow may be the distance from one end of the lot to the other or from the house to the edge of the lot. In some heavily urbanized drainage areas, sheet flow may not exist in the headwater area. The NRCS method employs Equation 3.3, which is a modified form kinematic wave equation, for the calculation of the sheet flow travel time. Maximum allowed travel time for Sheet Flow is 20 minutes.

$$T_t = \frac{0.42(nL)^{0.8}}{P_2^{0.5}(S)^{0.4}} \qquad \text{[Equation 3.3]}$$

Where,

Tt = Sheet flow travel time in minutes

L = Length of the reach in feet

n = Manning's n (see Table 3.2)

P2 = 2-year, 24-hour rainfall in inches (see Table 3.3)

S = Slope of the ground in ft/ft

Table 3.2 Manning's "n" for Overland Flow

Surface Description	Manning's "n"[1]
Concrete (rough or smoothed finish)	0.015
Asphalt	0.016
Fallow (no residue)	0.05
Cultivated Soils:	
Residue cover ≤ 20%	0.06
Residue cover > 20%	0.17
Grass:	
Short-grass prairie	0.15
Dense grasses[2]	0.24
Range (natural)	0.13
Woods:[3]	
Light underbrush	0.40
Dense underbrush	0.80

[1] The Manning's n values are a composite of information compiled by Engman (1986).
[2] Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass, and native grass mixtures.
[3] When selecting n, consider cover to a height of about 0.1 feet. This is the only part of the plant cover that will obstruct sheet flow.

3.2.1.6.2: Shallow Concentrated Flow

Sheet flow usually becomes shallow concentrated flow collecting in swales, small rills, and gullies. Shallow concentrated flow is assumed not to have a well-defined channel and has flow depths of 0.1 to 0.5 feet. The travel time for shallow concentrated flows can be computed by Equations 3.4 and 3.5. These two equations are based on the solution of Manning's equation with different assumptions for n (Manning's roughness coefficient) and r (hydraulic radius, feet). For unpaved areas, n is 0.05 and r is 0.4; for paved areas, n is 0.025 and r is 0.2.

$$\text{Unpaved} \quad T_t = \frac{L}{60*16.1345*S^{0.5}} \qquad \text{[Equation 3.4]}$$

$$\text{Paved} \quad T_t = \frac{L}{60*20.3282*S^{0.5}} \qquad \text{[Equation 3.5]}$$

Where,

T_t = Travel time for shallow concentrated flows in minutes

L = Length of the reach in feet

S = Slope of the ground in feet/feet

3.2.1.6.3: Channel or Storm Drain Flow

The velocity in an open channel or a storm drain not flowing full can be determined by using Manning's Equation. Channel velocities can also be determined by using backwater profiles. For open channel flow, average flow velocity is usually determined by assuming a bank-full condition. Note that the channel flow component of the time of concentration may need to be divided into multiple segments in order to represent significant changes in channel characteristics.

For storm drain flow under pressure conditions (where the hydraulic grade line is higher than the lowest crown of a storm drain) the following equation should be applied:

$$V = Q/A \qquad \text{[Equation 3.6]}$$

Where:

V = Average velocity, feet/s

Q = Design discharge, cfs

A = Cross-sectional area, ft^2

Flow travel time through a channel can be calculated by Equation 3.7:

$$T_t = \sum \frac{L_i}{60V_i} \qquad \text{[Equation 3.7]}$$

Where:

L_i = The i-th channel segment length, feet

V_i = The average flow velocity within the channel segment, feet/s

T_t = Total flow travel time through the channel, minutes

Rainfall Intensity (i)

Rainfall intensity (i) is the average rainfall rate in inches per hour and is selected on the basis of design rainfall duration and design frequency of occurrence. The design duration is equal to the time of concentration for the drainage area under consideration. The design frequency of occurrence is a statistical variable which is established by design standards or chosen by the engineer as a design parameter. The selection of the frequency criteria is necessary before applying any hydrologic method. Storm drainage improvements in San Marcos must be designed to intercept and carry the runoff from a 25-year frequency storm (4% annual chance event), with an auxiliary or overflow system capable of carrying a 100-year frequency storm (1% annual chance event).

The rainfall intensity used in the rational method can be read from the intensity-duration-frequency (IDF) curves based on the selected design frequency and design duration. The design engineer can also calculate the value of rainfall intensity from the best-fit IDF Equation 3.8 to be discussed later in this sub-section with known Tc value for the entire drainage area of interest.

The depth-duration frequency (DDF) and IDF values are shown in Table 3.3 and Table 3.4. These tables have been updated to reflect precipitation-frequency values for San Marcos obtained from NOAA's Atlas 14 Precipitation-Frequency Atlas of the United States (NOAA Atlas 14, Volume 11, version 2.0). Rainfall point locations were taken at the San Marcos City Hall located at 630 E. Hopkins Street.

TABLE 3.3 DEPTH-DURATION FREQUENCY TABLE FOR THE SAN MARCOS REGION

Annual Chance Exceedance	Recurrence Interval (Year)	Depth of Precipitation (Inches)							
		5-min	15-min	60-min	2-hr	3-hr	6-hr	12-hr	24-hr
50%	2	0.52	1.06	1.95	2.43	2.71	3.19	3.65	4.15
20%	5	0.66	1.33	2.47	3.11	3.50	4.18	4.84	5.54
10%	10	0.78	1.56	2.92	3.75	4.28	5.18	6.04	6.92
4%	25	0.95	1.89	3.56	4.70	5.46	6.74	7.90	9.07
2%	50	1.09	2.16	4.07	5.52	6.51	8.15	9.58	11.00
1%	100	1.23	2.45	4.62	6.43	7.71	9.80	11.60	13.20
0.5%	200	1.38	2.74	5.23	7.45	9.06	11.70	13.90	15.80
0.2%	500	1.59	3.14	6.12	8.95	11.10	14.50	17.30	19.90

*Atlas 14 Rainfall Point Location: San Marcos City Hall

Table 3.4 Intensity-Duration-Frequency Table for the San Marcos Region

Annual Chance Exceedance	Recurrence Interval (year)	Precipitation Intensity (inches per hour)							
		5-min	15-min	60-min	2-hr	3-hr	6-hr	12-hr	24-hr
50%	2	6.25	4.24	1.95	1.21	0.90	0.532	0.303	0.173
20%	5	7.90	5.31	2.47	1.56	1.17	0.698	0.402	0.231
10%	10	9.34	6.24	2.92	1.87	1.42	0.864	0.501	0.288
4%	25	11.40	7.57	3.56	2.35	1.82	1.130	0.656	0.378
2%	50	13.00	8.65	4.07	2.76	2.17	1.360	0.795	0.457
1%	100	14.80	9.78	4.62	3.21	2.57	1.640	0.959	0.550
0.5%	200	16.60	11.00	5.23	3.73	3.02	1.950	1.150	0.660
0.2%	500	19.00	12.50	6.12	4.48	3.68	2.420	1.440	0.829

*Atlas 14 Point Location: San Marcos City Hall

The following equation mathematically represents the San Marcos area intensity-duration-frequency curves:

$$i = \frac{a}{(t+b)^c}$$ [Equation 3.8]

Where,

i = Average rainfall intensity, inches per hour

t = Storm duration in minutes, which is equal to the time of concentration for the entire drainage area of interest

a, b, and c = Coefficients for different storm frequencies (see Table 3.5)

Table 3.5 San Marcos Region Intensity-Duration Frequency Curve Coefficients

Annual Chance Exceedance	Recurrence Interval (year)	Fitting Parameters for IDF Curves (Equation 3.8)		
		A	B	C
50%	2	46.949	9.522	0.750
20%	5	54.734	8.937	0.732
10%	10	58.315	8.132	0.709
4%	25	62.103	7.109	0.680
2%	50	64.030	6.398	0.657
1%	100	64.735	5.477	0.631
0.5%	200	66.309	4.889	0.609
0.2%	500	68.848	4.352	0.581

*Altas 14 Point Location: San Marcos City Hall

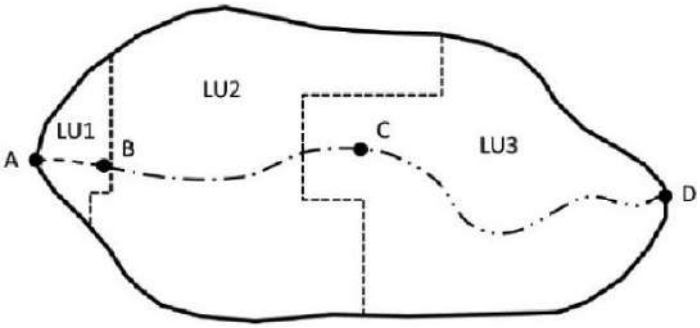
The a, b and c parameters listed in Table 3.5 were derived using nonlinear regression methods and the data included in Table 3.4. The IDF curves and the IDF equations are applicable for all design frequencies shown. They are required for use in determining peak flows by the Rational Method.

3.2.1.7: Drainage Area (A)

The size (acres) of the watershed needs to be determined for application of the Rational Method. The area may be determined through the use of topographic maps, supplemented by field surveys where topographic data has changed or where the contour interval is too great to distinguish the direction of flow. The drainage divide lines are determined based on topography, street layout, lot grading, building structure configuration and orientation, drainage system layout and other features that are created by the urbanization process.

Example 3.1

An urbanized watershed is shown on the following figure. Three types of flow conditions exist between the most distant point in the watershed and the outlet. The calculation of time of concentration and travel time in each reach is as follows:



Reach	Description of Flow	Slope (%)	Length (Ft.)	"n" Value/ Surface Type
A to B	Sheet flow (grass lawn)	1.8	50	0.24
B to C	Shallow concentrated flow (gutter)	2.0	840	Paved
C to D	Storm drain with inlets (D=3 feet)	1.5	1,200	0.015

For reaches A-B and B-C, the travel time can be calculated from Equations 3.3 and 3.4.

$T_{t(A-B)} = 0.42(nL)^{0.8}/(P_2)^{0.5}s^{0.4}$

$T_{t(A-B)} = 0.42(0.24 \times 50)^{0.8}/(3.44)^{0.5}(0.018)^{0.4}$

$= 8.25$ minutes

$T_{t(B-C)} = L/(60(20.3282)(s^{0.5}))$

$T_{t(B-C)} = 840/(60(20.3282)(0.020)^{0.5})$

$= 4.87$ minutes

The flow velocity in reach C-D needs to be calculated from Manning's Equation, using the assumption of full pipe flow, as follows:

$V(C-D) = (1.49/n)\ R^{0.67}s^{0.5}$

$= (1.49/n)\ (D/4)^{0.67}s^{0.5}$

$= (1.49/0.015)\ (3/4)^{0.67}\ (0.015)^{0.5}$

$= 10.0$ ft/s

The channel flow travel time is calculated by dividing the length by the velocity.

$T_{t(C-D)} = 1200/(60 \times 10.0) = 2.0$ min

The total time of concentration is calculated by adding the component sheet, shallow concentrated and channel flow segments.

$T_c = 8.25 + 4.87 + 2.0 = 15.12$ min

The runoff coefficients (C) for the three (3) areas are given as follows for the 100-year storm (1% annual chance event).

Area	Land Use	C	Area
LU_1	Grass Area (fair condition, flat)	0.41	3
LU_2	Commercial (composite of paved and grassed areas)	0.85	20
LU_3	Industrial (composite of paved and grassed areas)	0.81	30
		Total	53

The rainfall intensity (i) of the 100-year storm can be calculated using Equation 3.8 together with the coefficients in Table 3.5 for a time of concentration of 15.12 minutes as 9.60 inches per hour.

The composite runoff coefficient (C) = (0.41 X 3 + 0.85 X 20 + 0.81 X 30)/53= 0.80

Thus, the peak flow Q_p = CiA = 0.80 X 9.60 in/hr X 53 acre = 407 cfs

3.2.1.8: The Soil Conservation Service Method for Calculation of Peak Flows

The Soil Conservation Service hydrologic method is widely used by engineers and hydrologists for analyses of small urban watersheds. This method is based on extensive analytical work using a wide range of statistical data concerning storm patterns, rainfall-runoff characteristics and many hydrologic observations in the United States.

The SCS method can be applied to urban drainage areas of any size. The major parameters required to calculate a runoff hydrograph with the method include the rainfall distribution, runoff curve numbers, time of concentration and drainage area. For detailed information regarding the SCS method and the TR-20 program, the user is referred to the following NRCS publications. These can be obtained from the Natural Resources Conservation Service at http://www.wcc.nrcs.usda.gov/. They are:

NEH-4: "Hydrology," Section 4, National Engineering Handbook

TR-20: Computer Program for Project Formulation, Hydrology

TR-55: Urban Hydrology for Small Watersheds

TP-149: A Method for Estimating Volume and Rate of Runoff in Small Watersheds

The HEC-HMS programs can be downloaded from the US Army Corps of Engineers website at http://www.hec.usace.army.mil/.

3.2.1.8.1: Hydrologic Modeling

The City of San Marcos has adopted the following methods for hydrologic modeling:

- Infiltration Loss: SCS Curve Number
- Hydrograph Transform: SCS Unit Hydrograph
- Precipitation Distribution: Frequency
- Precipitation Duration: 24 hours

The DDF and IDF values to be used for the San Marcos region are shown in Table 3.3 and Table 3.4. When using the HEC-HMS model, the computational time interval should be selected based on criteria for the minimum lag time. The maximum computational time interval used in a HEC-HMS model should be 6 minutes.

Table 3.6 San Marcos 100-year Storm Frequency Distribution Ordinates in 5-Minute Time Increments

Time	Incremental Rainfall (Inches)	Cumulative Rainfall (Inches)	Incremental Rainfall (Percentage)	Cumulative Rainfall (Percentage)	Time	Incremental Rainfall (Inches)	Cumulative Rainfall (Inches)	Incremental Rainfall (Percentage)	Cumulative Rainfall (Percentage)
0:00	0.0000	0.0000	0.0000	0.0000	12:30	0.1651	8.8637	0.0125	0.6721
0:05	0.0086	0.0086	0.0006	0.0006	12:35	0.1796	9.0433	0.0136	0.6857
0:10	0.0086	0.0172	0.0007	0.0013	12:40	0.1662	9.2096	0.0126	0.6984
0:15	0.0087	0.0259	0.0007	0.0020	12:45	0.1554	9.3650	0.0118	0.7101
0:20	0.0087	0.0346	0.0007	0.0026	12:50	0.1464	9.5114	0.0111	0.7212
0:25	0.0088	0.0434	0.0007	0.0033	12:55	0.1388	9.6501	0.0105	0.7318
0:30	0.0088	0.0522	0.0007	0.0040	13:00	0.1322	9.7823	0.0100	0.7418
0:35	0.0089	0.0610	0.0007	0.0046	13:05	0.1186	9.9009	0.0090	0.7508
0:40	0.0089	0.0700	0.0007	0.0053	13:10	0.1136	10.0144	0.0086	0.7594
0:45	0.0090	0.0790	0.0007	0.0060	13:15	0.1091	10.1235	0.0083	0.7677
0:50	0.0090	0.0880	0.0007	0.0067	13:20	0.1051	10.2286	0.0080	0.7756
0:55	0.0091	0.0971	0.0007	0.0074	13:25	0.1015	10.3300	0.0077	0.7833
1:00	0.0091	0.1062	0.0007	0.0081	13:30	0.0982	10.4282	0.0074	0.7908
1:05	0.0092	0.1154	0.0007	0.0088	13:35	0.0734	10.5016	0.0056	0.7963
1:10	0.0093	0.1247	0.0007	0.0095	13:40	0.0709	10.5726	0.0054	0.8017
1:15	0.0093	0.1340	0.0007	0.0102	13:45	0.0686	10.6412	0.0052	0.8069
1:20	0.0094	0.1434	0.0007	0.0109	13:50	0.0665	10.7077	0.0050	0.8120
1:25	0.0094	0.1528	0.0007	0.0116	13:55	0.0645	10.7722	0.0049	0.8168
1:30	0.0095	0.1623	0.0007	0.0123	14:00	0.0627	10.8349	0.0048	0.8216
1:35	0.0096	0.1718	0.0007	0.0130	14:05	0.0610	10.8959	0.0046	0.8262
1:40	0.0096	0.1815	0.0007	0.0138	14:10	0.0594	10.9553	0.0045	0.8307
1:45	0.0097	0.1911	0.0007	0.0145	14:15	0.0579	11.0132	0.0044	0.8351
1:50	0.0098	0.2009	0.0007	0.0152	14:20	0.0565	11.0698	0.0043	0.8394
1:55	0.0098	0.2107	0.0007	0.0160	14:25	0.0552	11.1250	0.0042	0.8436
2:00	0.0099	0.2206	0.0007	0.0167	14:30	0.0540	11.1789	0.0041	0.8477
2:05	0.0099	0.2305	0.0008	0.0175	14:35	0.0528	11.2317	0.0040	0.8517
2:10	0.0100	0.2405	0.0008	0.0182	14:40	0.0517	11.2834	0.0039	0.8556
2:15	0.0101	0.2506	0.0008	0.0190	14:45	0.0506	11.3340	0.0038	0.8594
2:20	0.0102	0.2608	0.0008	0.0198	14:50	0.0496	11.3837	0.0038	0.8632
2:25	0.0102	0.2710	0.0008	0.0205	14:55	0.0487	11.4323	0.0037	0.8669
2:30	0.0103	0.2813	0.0008	0.0213	15:00	0.0478	11.4801	0.0036	0.8705
2:35	0.0104	0.2916	0.0008	0.0221	15:05	0.0330	11.5131	0.0025	0.8730
2:40	0.0104	0.3021	0.0008	0.0229	15:10	0.0323	11.5454	0.0024	0.8755
2:45	0.0105	0.3126	0.0008	0.0237	15:15	0.0317	11.5771	0.0024	0.8779
2:50	0.0106	0.3232	0.0008	0.0245	15:20	0.0311	11.6081	0.0024	0.8802
2:55	0.0107	0.3339	0.0008	0.0253	15:25	0.0305	11.6386	0.0023	0.8825
3:00	0.0108	0.3446	0.0008	0.0261	15:30	0.0299	11.6685	0.0023	0.8848
3:05	0.0108	0.3555	0.0008	0.0270	15:35	0.0294	11.6979	0.0022	0.8870

Table 3.6 San Marcos 100-year Storm Frequency Distribution Ordinates in 5-Minute Time Increments

Time	Incremental Rainfall (Inches)	Cumulative Rainfall (Inches)	Incremental Rainfall (Percentage)	Cumulative Rainfall (Percentage)	Time	Incremental Rainfall (Inches)	Cumulative Rainfall (Inches)	Incremental Rainfall (Percentage)	Cumulative Rainfall (Percentage)
3:10	0.0109	0.3664	0.0008	0.0278	15:40	0.0289	11.7267	0.0022	0.8892
3:15	0.0110	0.3774	0.0008	0.0286	15:45	0.0284	11.7551	0.0022	0.8914
3:20	0.0111	0.3885	0.0008	0.0295	15:50	0.0279	11.7830	0.0021	0.8935
3:25	0.0112	0.3996	0.0008	0.0303	15:55	0.0274	11.8104	0.0021	0.8956
3:30	0.0113	0.4109	0.0009	0.0312	16:00	0.0270	11.8374	0.0020	0.8976
3:35	0.0114	0.4223	0.0009	0.0320	16:05	0.0266	11.8639	0.0020	0.8996
3:40	0.0114	0.4337	0.0009	0.0329	16:10	0.0262	11.8901	0.0020	0.9016
3:45	0.0115	0.4452	0.0009	0.0338	16:15	0.0258	11.9159	0.0020	0.9036
3:50	0.0116	0.4569	0.0009	0.0346	16:20	0.0254	11.9412	0.0019	0.9055
3:55	0.0117	0.4686	0.0009	0.0355	16:25	0.0250	11.9662	0.0019	0.9074
4:00	0.0118	0.4804	0.0009	0.0364	16:30	0.0247	11.9909	0.0019	0.9093
4:05	0.0119	0.4924	0.0009	0.0373	16:35	0.0243	12.0152	0.0018	0.9111
4:10	0.0120	0.5044	0.0009	0.0382	16:40	0.0240	12.0392	0.0018	0.9129
4:15	0.0121	0.5165	0.0009	0.0392	16:45	0.0237	12.0629	0.0018	0.9147
4:20	0.0122	0.5288	0.0009	0.0401	16:50	0.0233	12.0862	0.0018	0.9165
4:25	0.0124	0.5411	0.0009	0.0410	16:55	0.0230	12.1092	0.0017	0.9182
4:30	0.0125	0.5536	0.0009	0.0420	17:00	0.0227	12.1320	0.0017	0.9200
4:35	0.0126	0.5661	0.0010	0.0429	17:05	0.0225	12.1544	0.0017	0.9217
4:40	0.0127	0.5788	0.0010	0.0439	17:10	0.0222	12.1766	0.0017	0.9233
4:45	0.0128	0.5916	0.0010	0.0449	17:15	0.0219	12.1985	0.0017	0.9250
4:50	0.0129	0.6045	0.0010	0.0458	17:20	0.0217	12.2202	0.0016	0.9266
4:55	0.0131	0.6176	0.0010	0.0468	17:25	0.0214	12.2416	0.0016	0.9283
5:00	0.0132	0.6308	0.0010	0.0478	17:30	0.0211	12.2627	0.0016	0.9299
5:05	0.0133	0.6441	0.0010	0.0488	17:35	0.0209	12.2836	0.0016	0.9315
5:10	0.0134	0.6575	0.0010	0.0499	17:40	0.0207	12.3043	0.0016	0.9330
5:15	0.0136	0.6711	0.0010	0.0509	17:45	0.0204	12.3247	0.0015	0.9346
5:20	0.0137	0.6848	0.0010	0.0519	17:50	0.0202	12.3449	0.0015	0.9361
5:25	0.0138	0.6986	0.0010	0.0530	17:55	0.0200	12.3649	0.0015	0.9376
5:30	0.0140	0.7126	0.0011	0.0540	18:00	0.0198	12.3847	0.0015	0.9391
5:35	0.0141	0.7267	0.0011	0.0551	18:05	0.0150	12.3997	0.0011	0.9403
5:40	0.0143	0.7410	0.0011	0.0562	18:10	0.0148	12.4145	0.0011	0.9414
5:45	0.0144	0.7554	0.0011	0.0573	18:15	0.0147	12.4292	0.0011	0.9425
5:50	0.0146	0.7700	0.0011	0.0584	18:20	0.0145	12.4437	0.0011	0.9436
5:55	0.0148	0.7848	0.0011	0.0595	18:25	0.0144	12.4581	0.0011	0.9447
6:00	0.0149	0.7997	0.0011	0.0606	18:30	0.0142	12.4723	0.0011	0.9458
6:05	0.0197	0.8193	0.0015	0.0621	18:35	0.0141	12.4863	0.0011	0.9468
6:10	0.0199	0.8392	0.0015	0.0636	18:40	0.0139	12.5002	0.0011	0.9479
6:15	0.0201	0.8593	0.0015	0.0652	18:45	0.0138	12.5140	0.0010	0.9489

Table 3.6 San Marcos 100-year Storm Frequency Distribution Ordinates in 5-Minute Time Increments

Time	Incremental Rainfall (Inches)	Cumulative Rainfall (Inches)	Incremental Rainfall (Percentage)	Cumulative Rainfall (Percentage)	Time	Incremental Rainfall (Inches)	Cumulative Rainfall (Inches)	Incremental Rainfall (Percentage)	Cumulative Rainfall (Percentage)
6:20	0.0203	0.8797	0.0015	0.0667	18:50	0.0136	12.5276	0.0010	0.9500
6:25	0.0206	0.9002	0.0016	0.0683	18:55	0.0135	12.5411	0.0010	0.9510
6:30	0.0208	0.9210	0.0016	0.0698	19:00	0.0134	12.5545	0.0010	0.9520
6:35	0.0210	0.9420	0.0016	0.0714	19:05	0.0132	12.5677	0.0010	0.9530
6:40	0.0213	0.9633	0.0016	0.0730	19:10	0.0131	12.5808	0.0010	0.9540
6:45	0.0215	0.9848	0.0016	0.0747	19:15	0.0130	12.5938	0.0010	0.9550
6:50	0.0218	1.0066	0.0017	0.0763	19:20	0.0129	12.6067	0.0010	0.9559
6:55	0.0220	1.0286	0.0017	0.0780	19:25	0.0127	12.6194	0.0010	0.9569
7:00	0.0223	1.0509	0.0017	0.0797	19:30	0.0126	12.6320	0.0010	0.9579
7:05	0.0226	1.0735	0.0017	0.0814	19:35	0.0125	12.6445	0.0009	0.9588
7:10	0.0229	1.0964	0.0017	0.0831	19:40	0.0124	12.6569	0.0009	0.9598
7:15	0.0232	1.1196	0.0018	0.0849	19:45	0.0123	12.6692	0.0009	0.9607
7:20	0.0235	1.1431	0.0018	0.0867	19:50	0.0122	12.6814	0.0009	0.9616
7:25	0.0238	1.1669	0.0018	0.0885	19:55	0.0121	12.6935	0.0009	0.9625
7:30	0.0241	1.1911	0.0018	0.0903	20:00	0.0120	12.7055	0.0009	0.9634
7:35	0.0245	1.2156	0.0019	0.0922	20:05	0.0119	12.7174	0.0009	0.9643
7:40	0.0248	1.2404	0.0019	0.0941	20:10	0.0118	12.7291	0.0009	0.9652
7:45	0.0252	1.2656	0.0019	0.0960	20:15	0.0117	12.7408	0.0009	0.9661
7:50	0.0256	1.2912	0.0019	0.0979	20:20	0.0116	12.7524	0.0009	0.9670
7:55	0.0260	1.3171	0.0020	0.0999	20:25	0.0115	12.7639	0.0009	0.9679
8:00	0.0264	1.3435	0.0020	0.1019	20:30	0.0114	12.7753	0.0009	0.9687
8:05	0.0268	1.3702	0.0020	0.1039	20:35	0.0113	12.7866	0.0009	0.9696
8:10	0.0272	1.3975	0.0021	0.1060	20:40	0.0112	12.7978	0.0009	0.9704
8:15	0.0277	1.4251	0.0021	0.1081	20:45	0.0111	12.8090	0.0008	0.9713
8:20	0.0281	1.4532	0.0021	0.1102	20:50	0.0110	12.8200	0.0008	0.9721
8:25	0.0286	1.4818	0.0022	0.1124	20:55	0.0110	12.8310	0.0008	0.9730
8:30	0.0291	1.5109	0.0022	0.1146	21:00	0.0109	12.8418	0.0008	0.9738
8:35	0.0296	1.5406	0.0022	0.1168	21:05	0.0108	12.8526	0.0008	0.9746
8:40	0.0302	1.5708	0.0023	0.1191	21:10	0.0107	12.8633	0.0008	0.9754
8:45	0.0308	1.6015	0.0023	0.1214	21:15	0.0106	12.8740	0.0008	0.9762
8:50	0.0314	1.6329	0.0024	0.1238	21:20	0.0106	12.8845	0.0008	0.9770
8:55	0.0320	1.6649	0.0024	0.1262	21:25	0.0105	12.8950	0.0008	0.9778
9:00	0.0326	1.6975	0.0025	0.1287	21:30	0.0104	12.9054	0.0008	0.9786
9:05	0.0473	1.7448	0.0036	0.1323	21:35	0.0103	12.9157	0.0008	0.9794
9:10	0.0482	1.7931	0.0037	0.1360	21:40	0.0103	12.9260	0.0008	0.9802
9:15	0.0492	1.8422	0.0037	0.1397	21:45	0.0102	12.9362	0.0008	0.9809
9:20	0.0501	1.8923	0.0038	0.1435	21:50	0.0101	12.9463	0.0008	0.9817

Table 3.6 San Marcos 100-year Storm Frequency Distribution Ordinates in 5-Minute Time Increments

Time	Incremental Rainfall (Inches)	Cumulative Rainfall (Inches)	Incremental Rainfall (Percentage)	Cumulative Rainfall (Percentage)	Time	Incremental Rainfall (Inches)	Cumulative Rainfall (Inches)	Incremental Rainfall (Percentage)	Cumulative Rainfall (Percentage)
9:25	0.0512	1.9435	0.0039	0.1474	21:55	0.0101	12.9564	0.0008	0.9825
9:30	0.0522	1.9957	0.0040	0.1513	22:00	0.0100	12.9663	0.0008	0.9832
9:35	0.0534	2.0491	0.0040	0.1554	22:05	0.0099	12.9763	0.0008	0.9840
9:40	0.0546	2.1037	0.0041	0.1595	22:10	0.0098	12.9861	0.0007	0.9847
9:45	0.0559	2.1595	0.0042	0.1638	22:15	0.0098	12.9959	0.0007	0.9855
9:50	0.0572	2.2167	0.0043	0.1681	22:20	0.0097	13.0056	0.0007	0.9862
9:55	0.0587	2.2754	0.0044	0.1725	22:25	0.0097	13.0152	0.0007	0.9869
10:00	0.0602	2.3356	0.0046	0.1771	22:30	0.0096	13.0248	0.0007	0.9877
10:05	0.0618	2.3974	0.0047	0.1818	22:35	0.0095	13.0344	0.0007	0.9884
10:10	0.0636	2.4610	0.0048	0.1866	22:40	0.0095	13.0438	0.0007	0.9891
10:15	0.0655	2.5265	0.0050	0.1916	22:45	0.0094	13.0532	0.0007	0.9898
10:20	0.0675	2.5940	0.0051	0.1967	22:50	0.0094	13.0626	0.0007	0.9905
10:25	0.0698	2.6637	0.0053	0.2020	22:55	0.0093	13.0719	0.0007	0.9912
10:30	0.0722	2.7359	0.0055	0.2075	23:00	0.0092	13.0811	0.0007	0.9919
10:35	0.0966	2.8325	0.0073	0.2148	23:05	0.0092	13.0903	0.0007	0.9926
10:40	0.0998	2.9323	0.0076	0.2224	23:10	0.0091	13.0994	0.0007	0.9933
10:45	0.1032	3.0356	0.0078	0.2302	23:15	0.0091	13.1084	0.0007	0.9940
10:50	0.1070	3.1426	0.0081	0.2383	23:20	0.0090	13.1174	0.0007	0.9947
10:55	0.1113	3.2538	0.0084	0.2467	23:25	0.0090	13.1264	0.0007	0.9954
11:00	0.1160	3.3698	0.0088	0.2555	23:30	0.0089	13.1353	0.0007	0.9960
11:05	0.1292	3.4990	0.0098	0.2653	23:35	0.0089	13.1441	0.0007	0.9967
11:10	0.1353	3.6344	0.0103	0.2756	23:40	0.0088	13.1529	0.0007	0.9974
11:15	0.1424	3.7768	0.0108	0.2864	23:45	0.0087	13.1617	0.0007	0.9980
11:20	0.1507	3.9275	0.0114	0.2978	23:50	0.0087	13.1704	0.0007	0.9987
11:25	0.1606	4.0880	0.0122	0.3100	23:55	0.0086	13.1790	0.0007	0.9993
11:30	0.1726	4.2606	0.0131	0.3231	0:00	0.0086	13.1876	0.0007	1.0000
11:35	0.1563	4.4169	0.0119	0.3349					
11:40	0.1754	4.5923	0.0133	0.3482					
11:45	0.2023	4.7947	0.0153	0.3636					
11:50	0.3162	5.1109	0.0240	0.3876					
11:55	0.3934	5.5043	0.0298	0.4174					
12:00	0.7162	6.2205	0.0543	0.4717					
12:05	1.2238	7.4443	0.0928	0.5645					
12:10	0.4977	7.9420	0.0377	0.6022					
12:15	0.3484	8.2903	0.0264	0.6286					
12:20	0.2206	8.5110	0.0167	0.6454					
12:25	0.1876	8.6986	0.0142	0.6596					

*This table is based on the Frequency rainfall distribution as defined in HEC-HMS for the 100-year storm recurrence interval.
Rainfall distribution for other storm recurrence intervals will be different.

Table 3.7 NRCS Runoff Curve Numbers for Urban Areas and Agricultural Lands (assuming ARC-II condition)

Cover Type	Hydrologic Condition	Curve Numbers for Hydrologic Soil Group A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.)					
Poor condition (grass cover 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right of way)		98	98	98	98
Streets and roads:					
Paved; curbs and storms drains (excluding right of way)		98	98	98	98
Paved; open ditches (including right of way)		83	89	92	93
Gravel (including right of way)		76	85	89	91
Dirt (including right of way)		72	82	87	89
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation)		77	86	91	94
Agricultural lands					
Grassland, or range-continuous forage for grazing[1]	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow-continuous grass, protected from grazing and generally mowed for hay		30	58	71	78
Brush—brush-weed-grass mixture with brush the major element[2]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	30[3]	48	65	73
Woods—grass combination (orchard or tree farm).[4]	Poor	57	73	82	86
	Fair	43	65	76	79
	Good	32	58	72	79
Woods[5]	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	30	55	70	77
Farmsteads—buildings, lanes, driveways and surrounding lots		59	74	82	86

[1] Poor: Less than 50% ground cover or heavily grazed with no mulch.
Fair: 50-75% ground cover and not heavily grazed.
Good: Greater than 75% ground cover and lightly or only occasionally grazed.

[2] Poor: Less than 50% ground cover.
Fair: 50 to 75% ground cover.
Good: Greater than 75% ground cover.

[3] Actual curve number is less than 30; use CN = 30 for runoff computations.

[4] Curve numbers shown were computed for areas with 50% woods and 50% grass (pasture) cover. Other combinations of conditions may be computed from the curve numbers for woods and pasture.

[5] Poor: Forest litter, small trees and brush are destroyed by heavy grazing or regular burning.
Fair: Woods are grazed but not burned, and some forest litter covers the soil.
Good: Woods are protected from grazing, and litter and brush adequately cover the soil.
Source: National Resources Conservation Service. TR-55: Urban Hydrology for Small Watersheds

3.2.1.8.2: Conservation Service Runoff Curve Numbers

The National Resources Conservation Service has developed an index, the runoff curve number, to represent the combined hydrologic effect of soil type, land use, agricultural land treatment class, hydrologic condition, and antecedent soil moisture. These watershed factors have the most significant impact in estimating the volume of runoff, and can be assessed from soil surveys, site investigations and land use maps.

The curve number is an indication of the potential runoff for a given antecedent soil moisture condition, and it ranges in value from zero to 100. The National Resources Conservation Service runoff curve numbers are grouped into three (3) antecedent soil moisture conditions—Antecedent Runoff Condition (ARC) I, ARC II and ARC III. Values of runoff curve numbers for all three (3) conditions may be computed following guidelines in Part 630, Chapter 10 of the National Engineering Handbook. ARC I is the dry soil condition and ARC III is the wet soil condition. ARC II is normally considered to be the average condition. The Antecedent Runoff Condition (ARC) was previously referred to as the Antecedent Moisture Condition (AMC) in older NRCS publications.

However, studies of hydrologic data indicate that ARC II is not necessarily representative of the average condition throughout Texas. Instead, investigations have shown that the average condition ranges from ARC I in west Texas to between ARC II and ARC III in east Texas. The NRCS curve number values provided in Table 3.7 are for an ARC II. If it is desired to change to an ARC I or III condition, the adjustments given in Part 630, Chapter 10 of the National Engineering Handbook should be used. Justification must be provided for the selection of an ARC other than condition II.

The National Resources Conservation Service has classified more than 4,000 soils into four (4) hydrologic groups, identified by the letters A, B, C, and D, to represent watershed characteristics.

Group A: (Low runoff potential). Soils having a high infiltration rate even when thoroughly wetted and consisting chiefly of deep, well-drained to excessively drained sands or gravels.

Group B: Soils having a moderate infiltration rate when thoroughly wetted and consisting chiefly of moderately deep to deep, moderately well to well-drained soils with moderately fine to moderately coarse texture.

Group C: Soils having a slow infiltration rate when thoroughly wetted and consisting chiefly of soils with a layer that impedes downward movement of water or soil with moderately fine to fine texture.

Group D: (High runoff potential). Soils having a very slow infiltration rate when thoroughly wetted and consisting chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a claypan or clay layer at or near the surface and shallow soils over nearly impervious material.

Table 3.7 lists the curve numbers for the four (4) soil groups under various land uses, land treatment and hydrologic conditions. Curve numbers for fully developed conditions should be based on maximum allowable impervious cover listed in the City of San Marcos zoning and land development codes. When calculating fully developed peak runoff rates it is recommended that the undeveloped curve number and the maximum allowable impervious cover be used as input parameters. In order to determine the soil classifications in the San Marcos region, the Natural Resource Conservation Service Soil Survey of Hays County, Texas should be used. Digital versions of these soil datasets are available online at the link provided in Appendix A: Resources.

3.2.1.8.3: Time of Concentration

The procedures for estimating time of concentration for the NRCS method are described in the SCS Technical Release 55 (TR-55) and in Section 3.2.1.6 of this manual. Three (3) types of flow (sheet flow, shallow concentrated flow and channel flow) are considered. Note that Table 3.2 shall be used for determination of sheet flow Manning's roughness coefficients rather than the table included in TR-55.

In hydrograph analysis, the time of concentration can be defined as the time from the end of excess rainfall to the point of inflection on the falling limb of the hydrograph. The time of concentration determines the shape of the runoff hydrograph. Times of concentration are required for the existing and developed conditions to adequately model the impact of the development on stormwater runoff. The methodology presented in TR-55 provides a reasonable approach for the estimation of time of concentration. The lag time, defined as the time between the center of mass of excess rainfall to the runoff peak, is typically used in the HEC-HMS implementation of the SCS methodology. The lag time can be estimated with Equation 3.9.

$$T_{lag} = 0.6\ T_c \qquad \text{[Equation 3.9]}$$

In general, times of concentration for the developed condition should be calculated based on conservative assumptions that consider the increased hydraulic efficiency expected with an ultimate developed condition. Times of concentration should be representative of the overall drainage area, not simply based on the longest (in either distance or time) flow path. Sheet flow for both existing and proposed conditions should be limited to 100 feet. This length should be considered a maximum; sheet flow lengths should be measured and justified for all conditions. Additionally, the minimum slope used for calculation of sheet and shall flow travel time components should be 0.005 feet per foot (0.5%).

3.2.1.9: Probable Maximum Storm/Flood Development

The purpose of this section is to describe a method for developing the Probable Maximum Flood (PMF) within the City of San Marcos jurisdiction. The PMF is calculated by obtaining the Probable Maximum Precipitation (PMP) for a specific storm duration and drainage area. The PMP rainfall depths presented in this section were derived for the San Marcos area and are only applicable for designing and managing dams within City of San Marcos' full purpose, limited purpose and extraterritorial jurisdictions. Typically, a PMF runoff model requires both a temporal and spatial distribution of the PMP. However, if the drainage area is less than 10 square miles, the spatial distribution is not required (i.e. the drainage area is considered small enough that the PMP values can reasonably be considered point rainfall values). The PMP values shown in this section are valid only for drainage areas less than 10 square miles.

1.1.1.1.1: Probably Maximum Precipitation (PMP)

The PMP values were derived using Hydrometeorological Report No. 52 (HMR-52) and Hydrometeorological Report No. 51 (HMR-51) per the guidance provided in the Hydrologic and Hydraulic Guidelines for Dams in Texas (January 2007) available from the Dam Safety Program at the Texas Commission on Environmental Quality (TCEQ). Table 3.8 contains a summary of PMP depths and intensities for various storm durations for drainage areas less than 10 square miles.

TABLE 3.8 PROBABLE MAXIMUM PRECIPITATION DEPTHS FOR THE CITY OF SAN MARCOS

STORM DURATION	DEPTH (IN)
1 hr	17.4
2 hr	21.6
3 hr	24.9
6 hr	31.1
12 hr	37.6
24 hr	44.7
48 hr	50.0
72 hr	53.4

1. All values are valid for drainage areas less than 10 square miles.
2. Do not use these depths with the Soil Conservation Service (SCS) Type III distribution. The relevant storm distributions were derived using the Hydrologic and Hydraulic Guidelines for Dams in Texas.

3.2.1.9.1: Probable Maximum Flood (PMF)

To determine the PMF, each of the possible storm durations (1, 2, 3, 6, 12, 24, 48, and 72-hour storms) needs to be analyzed in order to determine the critical duration. The critical duration is the storm duration that produces the highest water surface elevation behind the dam. The PMF for each storm duration is derived using the PMP depths from Table 3.8 and using a rainfall-runoff model (i.e. HEC-HMS, TR-20). The rainfall-runoff model should use the temporal

distribution as provided in the Hydrologic and Hydraulic Guidelines for Dams in Texas. The runoff parameters used in the PMF model are the same as those used for runoff analyses of the more frequent storm events, with the exception of curve numbers and the temporal distribution of rainfall.

Runoff curve numbers for the PMF need to reflect the assumption that the soils will be saturated. Therefore, the runoff curve number should be based on ARC III. The appropriate curve number should be chosen using the tables provided in Section 3.2.1.8.2. These are ARC II values which can be converted to ARC III values using Table 10.1 in Part 630, Chapter 10 of the National Engineering Handbook. Note that the ARC was previously referred to as the Antecedent Moisture Condition (AMC) in older NRCS publications.

3.2.1.10: Impervious Cover Determination

Impervious cover determination is necessary to calculate the benefits of reducing impervious cover during the site planning process and to assess the hydromodification impacts of a proposed project.

In reference to Article 3.9.1.3., roads, sidewalks, parking areas, buildings and other impermeable construction covering the natural land surface shall be considered as impervious cover. To estimate the percent of impervious cover created by the development of a parcel or tract of land, the following methods should be used:

(1) Roadways (except cul-de-sacs):

(Wr + 2Wc) x (lf) = square feet

Where:

Wr = width of roadway pavement

Wc = width of curb

lf = linear feet

(2) cul-de-sacs (use either subsection a or subsection b of this Section, as is applicable):

a. (P/2 + 10) (Wr + 2Wc) = square feet

Where:

P = perimeter of right-of-way

Wr = width of roadway

Wc = width of curb

b. (Wr + 2Wc) x (lf) + (r + Wc)2 = square feet

Where:

Wr = width of roadway

Wc = width of curb

lf = 3.142 (linear feet to point of tangency with cul-de-sac turnaround

R = radius of turnaround pavement

(3) Driveway approaches:

(16) x (9.5) x (N) = square feet

Where:

16 = average driveway width in feet

9.5 = length of driveway approach for standard ten-foot curb base minus curb. (This factor may vary depending on ROW and resulting curb base.)

N = total number of lots

(4) Single-family residential areas:

(3,000) (N) = square feet

Where:

3,000 = average impervious cover for single-family residential lots

N = number of lots

(5) Multifamily, commercial and industrial land uses: The impervious cover for other land uses, including multifamily, commercial and industrial, shall be determined by addressing the following factors:

1. Ground floor building square footage of all structures.
2. Estimated square footage of off-street parking.
3. Driveways and/or roadways on-site.

(b) If alternative pervious surfaces for parking or other related uses are proposed, adjustment factors applied to the impervious cover shall be based on the materials perviousness as listed in the manufacturers testing data. The use of any alternative surfaces must be authorized by the Engineering Department. The adjustment is only for water quantity calculations related to flood control and adjustments are not provided for water quality calculations.

(Ord. No. 2009-73, § 30, 12-1-09)

3.2.2: CALCULATION OF WATER QUALITY VOLUME

The Post-Construction Stormwater Performance Standards for stormwater quality and stream protection are defined in the San Marcos Development Code Section 6.1.4.1, B and C as:

Water Quality Volume and Treatment Level Table

LOCATION	WQV AMOUNT DESIGN RAINFALL	WQV TREATEMENT LEVEL
Edwards Aquifer Recharge Zone	1.60"	89%
Edwards Aquifer Transition Zone and Contributing Zone within the Transition Zone	1.25"	85%
San Marcos River Protection Zone	1.25"	80%
San Marcos River Corridor	1.60"	89%

These standards exceed and replace the TCEQ Edwards Aquifer Criteria within the City of San Marcos' jurisdiction. The TCEQ calculation template for water quality in the Edward Aquifer zones that has been modified to meet the required SMDC TSS treatment level, can be used to calculate the water quality volume and SCM size as it relates to TSS treatment level. Note that the minimum water quality volume required for the site is calculated using the specified design rainfall listed in the above table. SCM's that utilize a water quality volume (bioretention, wet basin, etc.) for TSS treatment, may need to be increased in size to meet the minimum water quality volume required for the site.

3.2.2.1: Calculating the TSS Load Reduction and Water Quality Volume for TSS Treatment

The following section provides a description of the modifications necessary to apply the TCEQ Edwards Aquifer Rules Technical Manual RG-348 (July 2005) to the City of San Marcos enhanced treatment standards.

Step 1- Determine Required TSS Removal

Equation 3.3 in TCEQ's RG-348 is modified to allow for incorporating the different water quality treatment levels required by the City.

$$L_M = T \times 34.0 \times A_T \times P$$

Where:

L_M = Required TSS Removal (Pounds)

T = Water Quality Treatment Level (decimal fraction)

A_T = Impervious Cover to be Treated (acres)

P = Average Annual Precipitation

Step 2- Select an Appropriate SCM

Select a SCM or series of SCMs that will achieve at least the Water Quality Treatment Level required. The higher the efficiency of the SCM, the less runoff that will need to be treated to achieve the required reduction. Table 3.9 includes the TCEQ list of SCMs and their TSS removal efficiencies. Note that the City of San Marcos has added Rain Gardens to the list.

TABLE 3.9 SCM AND TSS REMOVAL EFFICIENCY

SCM	TSS REDUCTION (%)
Retention/Irrigation	100
Extended Detention Basin (aka Sedimentation)	75
Grassy Swale	70
Vegetated Filter Strip	85
Sand Filter	89
AquaLogicTM Cartridge Filter System	95
Wet Basin	93
Constructed Wetland	93
Bioretention (aka Biofiltration)*	89
Rain Garden*	89
Permeable Concrete with under-drain	93
Permeable Concrete without under-drain	100
Wet Vault	Sizing Dependent

*For design guidance and criteria, the San Marcos Stormwater Technical Manual Appendix B should be used instead of the TCEQ RG-348 bioretention criteria, especially for the filtration media specifications.

For selection and sizing of SCMs to meet this standard, refer to Appendix B and the TCEQ Edwards Aquifer Technical Guidance Manual.

Step 3- Calculate TSS Load Removed by SCMs

The following section describes how to determine the load removed by a proposed SCM(s). The load removed depends on the amount of TSS entering the SCM(s) and its effectiveness.

The load entering each SCM is calculated from the sum of the contribution of the impervious and pervious areas with their respective stormwater concentrations for the SCM catchment area. This calculation assumes that no runoff bypasses the treatment facility.

L_R = (SCM efficiency) x 0.226 x P x (A_I x 0.9 x 170 mg/L + A_P x 0.03 x 80 mg/L)

Which simplifies to:

L_R = (SCM efficiency) x P x (A_I x 34.6 + A_P x 0.54)

Where:

L_R = Load removed by SCM

SCM efficiency = TSS removal efficiency (expressed as a decimal fraction from Table 3.9)

A_I = impervious tributary area to the SCM (ac)

A_P = pervious tributary area to the SCM (ac)

P = average annual precipitation (33 in for San Marcos)

Step 4: Calculate Fraction of Annual Runoff to be Treated

Based on the load reduction calculated above for each of the SCMs installed at the site and the required load reduction, calculate the fraction of annual runoff to be treated using the equation below. This calculation assumes a constant concentration of TSS in the runoff.

$$F = \frac{L_M}{\sum L_R}$$

Where:

F = Fraction of the annual rainfall treated by the SCM

L_R = Load removed for each SCM from Step 3 calculation (pounds)

L_M = Required load reduction from Step 1 (pounds)

The value for F must be less than 1.0, since a value greater than that indicates that more runoff than would occur in an average year must be treated and that is infeasible. If a value for F of more than 1.0 is calculated a more efficient SCM must be selected for the site or additional SCMs must be selected to form a treatment train that can achieved the required reduction in TSS.

Step 5: Calculate Water Quality Volume for TSS Treatment

This step relates the statistical properties of storm size and flow rate in the regulated area to the total volume of runoff. Water quality volume for capture and treat devices is developed from Table B.1.2, which relates rainfall depth to the percentage of annual rainfall that occurs in storms less than or equal to this depth (i.e., 100% of the annual rainfall occurs in storms of 4 inches or less on average, while 78% of the annual runoff occurs in storms of an inch or less). For SCMs designed to capture and treat the runoff, the value, F, calculated in Step 4 is used to enter Table 3.10 and find the rainfall depth associated with this fraction. For flow through type devices (grassy swales and wet vaults), refer to guidance in TCEQ's RG-348.

TABLE 3.10 RELATIONSHIP BETWEEN FRACTION OF ANNUAL RAINFALL AND RAINFALL DEPTH (IN)

F	Rainfall Depth	F	Rainfall Depth	F	Rainfall Depth	F	Rainfall Depth
1.00	4.00	0.80	1.08	0.60	0.58	0.40	0.29
0.99	3.66	0.79	1.04	0.59	0.56	0.39	0.28
0.98	3.33	0.78	1.00	0.58	0.54	0.38	0.27
0.97	3.00	0.77	0.97	0.57	0.52	0.37	0.25
0.96	2.80	0.76	0.94	0.56	0.50	0.36	0.24
0.95	2.60	0.75	0.92	0.55	0.49	0.35	0.23
0.94	2.40	0.74	0.89	0.54	0.47	0.34	0.23
0.93	2.20	0.73	0.86	0.53	0.46	0.33	0.22
0.92	2.00	0.72	0.83	0.52	0.45	0.32	0.21
0.91	1.80	0.71	0.80	0.51	0.44	0.31	0.20
0.90	1.70	0.70	0.78	0.50	0.42	0.30	0.19
0.89	1.60	0.69	0.75	0.49	0.41	0.29	0.18
0.88	1.50	0.68	0.73	0.48	0.40	0.28	0.18
0.87	1.44	0.67	0.71	0.47	0.38	0.27	0.17
0.86	1.38	0.66	0.69	0.46	0.37	0.26	0.16
0.85	1.32	0.65	0.67	0.45	0.36	0.25	0.15
0.84	1.26	0.64	0.66	0.44	0.34		
0.83	1.20	0.63	0.64	0.43	0.33		
0.82	1.16	0.62	0.62	0.42	0.32		
0.81	1.12	0.61	0.60	0.41	0.31		
0.80	1.08	0.60	0.58	0.40	0.29		

Once the appropriate rainfall depth has been determined from Table 3.10, the water quality volume for each SCM can be calculated from:

WQV = Rainfall depth x Runoff Coefficient x Area

Where the rainfall depth is determined from Table 3.10, the runoff coefficient for the tributary area from Figure 3.5 or calculated using Runoff Coefficient equations below and the area is the portion of site contributing runoff to the SCM.

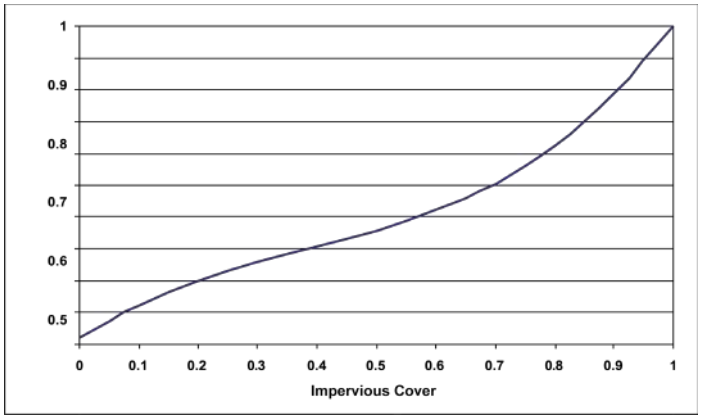


FIGURE 3.5 RELATIONSHIP BETWEEN RUNOFF COEFFICIENT AND IMPERVIOUS COVER

$$\text{Runoff Coefficient} = 1.72(IC)^3 - 1.97(IC)^2 + 1.23(IC) + 0.02$$

Where: IC = fraction of impervious cover

Offsite drainage should be conveyed around or through the site without entering a SCM. Occasionally, it is not feasible to prevent off-site runoff from entering a SCM on the tract. When this occurs the size of the SCM should be increased to account for the additional runoff generated by this area.

To properly size the SCM to account for this volume, all the calculations are performed based on the site characteristics alone until WQV equation is reached. At that point the runoff coefficient is determined based only on the impervious cover of the site, but then it is multiplied times the entire tributary area (including offsite areas) to determine capture volume. In this manner adding offsite drainage always results in a larger pond than if runoff from the site alone were treated.

3.2.2.2: Calculate Water Quality Volume Based on Design Rainfall

For determining the water quality volume based on the design rainfall, first determine the Design Rainfall based on the sites location from Water Quality Volume and Treatment Level Table in Section 6.1.4.1, B and C of the SMDC. Then use this rainfall depth in the equation below.

$$WQV = \text{Design Rainfall depth} \times \text{Runoff Coefficient} \times \text{Area}$$

The runoff coefficient for the tributary area can be determined from Figure B.1.1 or calculated using Runoff Coefficient equations below and the area is the portion of site contributing runoff to the SCM.

$$\text{Runoff Coefficient} = 1.72(IC)^3 - 1.97(IC)^2 + 1.23(IC) + 0.02$$

Where: IC = fraction of impervious cover

If the SCM being used for the treatment of TSS requires a water quality volume, use the larger of the two volumes calculated here or in step 5 of the previous section. For SCMs that do not require a capture volume, the water quality volume determined in this section will require capture on site and released over a period of 48-hours.

3.2.2.3: Example Development Site Calculations

A developer is proposing a redevelopment of a residential site in San Marcos. The site is located within the Edwards Aquifer Transition Zone and in a neighborhood that allows ND-3 zoning per the Comprehensive Plan. The project site is an existing large residential lot with a total area of 2.26 acres (Figure 3.6). All zoning, setback, and local regulations were followed during the site planning and design phase. The site is currently large lot residential with less than five percent impervious (0.11 acres) and all of the soils onsite are hydrologic soil group B.

The first stage in determining the available area for development was to delineate the watershed for the adjacent creek and determine the required water quality and buffer zones. A drainage area delineation for the overall watershed was completed and resulted in a total drainage area of 37.8 acres (Exhibit A). For sites within the Transition Zone where the watershed area is between 5 and 50 acres but the creek is Non FEMA-mapped, a 25 foot water quality zone is required. In addition, a required 25-foot buffer outside of the water quality zone is required per SMDC Section 6.2.2.2. Therefore,

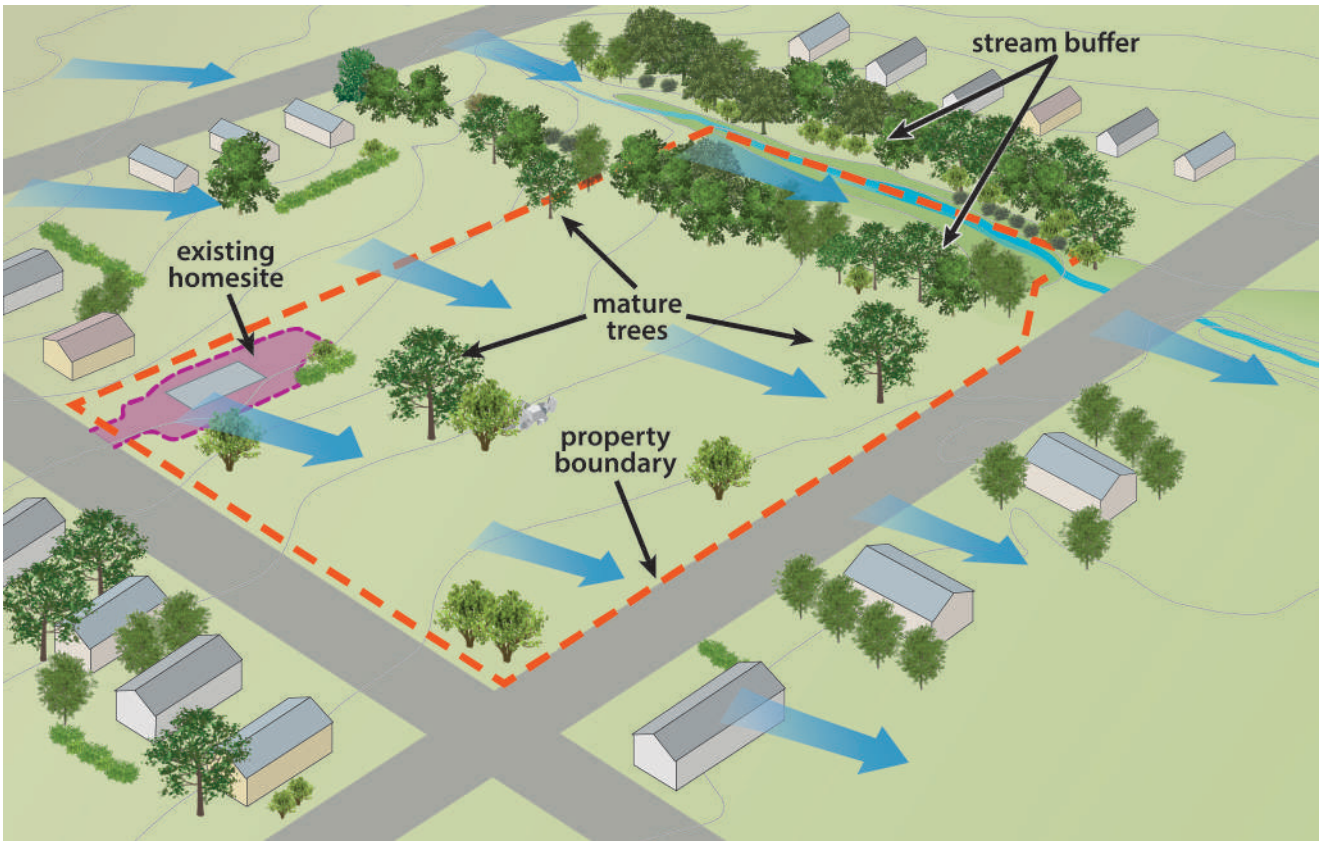


FIGURE 3.6 CONCEPTUAL DEVELOPMENT SITE

by using the waterway centerline offset method and keeping the waterway natural, the site area that can be used for development is 1.67 acres (Exhibit B). For this development, both the buffer and water quality zones are being preserved. See Section 2.3.1.3.3 of this manual for development allowances within the water quality and buffer zones. An alternative procedure for defining the water quality zone starts with the 100-yr floodplain based on the fully developed watershed conditions and then adds the 25-foot buffer zone.

The site planning resulted in a layout such that the proposed impervious areas are generally up-gradient of the pervious areas to enhance implementation of stormwater management and treatment SCMs. The site layout consists of 17 lots, split into two clusters with each cluster surrounding a courtyard. Driveways, parking areas, sidewalks and planting areas are included as part of the site design. The general layout is shown in Exhibit III-B-2 with existing streets on two sides, a creek along another side and existing average residential development on the final side. The water quality and buffer zones along the creek are also included.

3.2.2.3.1: Example Development - Site Impervious Cover Determination

The example development site is considered a non-residential site. Thus, the impervious cover was determined for the site layout for the following items (Exhibit C):

1. Ground floor building square footage of all structures.
2. Estimated square footage of off-street parking.
3. Sidewalks, driveways and/or roadways on-site.

The total amount of impervious area on the site as proposed is 1.22 acres.

3.2.2.3.2: Example Development Site - Calculating the TSS Removal Required and Load Removed By SCM

The proposed site is located in the Edwards Aquifer Transition Zone.

The first step in sizing the facilities is to determine how much treatment is required, based on the total suspended solids (TSS) load removal. The TCEQ equation, modified for the City of San Marcos requirements and the rainfall in the area, becomes:

$L_M = T \times 34.0 \times A_T \times P$

Where:

L_M = Required TSS Removal (Pounds)

T = Water Quality Treatment Level (decimal fraction)

A_T = Impervious Cover to be Treated (acres)

P = Average Annual Precipitation (33-inches for San Marcos)

For this site,

A_{P_EX} = 2.15 acres

A_N = 1.11 acres (1.22 acres proposed – 0.11 acres existing)

A_{P_DEV} = 1.42 acres

Therefore,

$L_M = 0.85 \times 34.0 \times 1.11 \text{ acres} \times 33 \text{ inches}$

L_M = 1,060 pounds of TSS

The next step in performing management is to select a SCM or a series of SCMs that will achieve at least an 85% reduction in TSS. For this site, a bioretention facility (or multiple bioretention facilities) are selected to meet the TSS removal criteria. Bioretention facilities can reduce TSS by up to 89% (see the SCM table above).

Next, the load removed by each bioretention facility must be calculated.

$$L_R = (\text{SCM efficiency}) \times P \times (A_I \times 34.6 + A_P \times 0.54)$$

Where:

L_R = Load removed by SCM (pounds)

SCM efficiency = TSS removal efficiency (decimal fraction)

A_I = Impervious tributary area to the SCM (acres)

A_P = Pervious tributary area to the SCM (acres)

P = Average annual precipitation (33-inches for San Marcos)

For now, let's assume that there will be one bioretention facility for the total site. The facility can capture runoff from the homes with connected downspout or roof drain.

L_R = (0.89) x (33 inches) x [(1.22 acres x 34.6) + (0.31 x 0.54)] Assume all 1.22 acres of the impervious cover and 0.308 Acres of the pervious area can be captured in the SCMs.

L_R = 1,245 lbs

Next, the fraction of annual runoff to be treated must be calculated. The following equation determines the fraction of removal.

$$F = \frac{L_M}{\sum L_R}$$

Where:

F = Fraction of the annual rainfall treated by the SCM

L_R = Load removed for SCM (pounds)

L_M = Required load reduction

For all of the treatment facilities, $F = \frac{1{,}060.0 \text{ lb}}{1{,}245 \text{ lb}} = 0.85$

3.2.2.3.3: Example Development Site – Calculate the Water Quality Volume

A. Determine WQV to Meet the TSS Requirement

The next step is to calculate the water quality volume required to be capture by the SCM(s) in order to meet the TSS reduction requirement. Based on Table B.1.2, a storm event of approximately 1.32 inches is required to be captured.

The water quality volume required to meet the TSS reduction is calculated using the below equation.

WQV = Rainfall Depth x Runoff Coefficient x Area

The runoff coefficient can be calculated as follows.

Runoff Coefficient = $1.72(IC)^3 - 1.97(IC)^2 + 1.23(IC) + 0.02$, where IC is the fraction of impervious cover.

For this example IC (fraction of impervious cover) = 1.22/1.53= 0.797

Since the water quality and buffer zones were left in their natural state, the developed area is 1.67 acres.

The Runoff Coefficient for the site is:

Runoff Coefficient = $1.72(0.797)^3 - 1.97(0.797)^2 + 1.23(0.797) + 0.02 = 0.62$

Therefore, the water quality volume necessary to meet the TSS reductions is:

WQV = 1.32-in/12 in/ft x 0.62 (1.53 Ac) x 43,560 sq. ft./Ac = 4,547 cubic feet.

B. Determine WQV Based on Design Rainfall (Min. WQV Required for Site)

The water quality volume based on the Design Rainfall is determined using the WQV equation listed previously and the appropriate rainfall depth listed in the Water Quality Volume and Treatment Level Table in Section 6.1.4.1, B and C of the SMDC. Since the project is located in the Transition Zone, the WQV Design Rainfall depth is 1.25-inches.

The water quality volume based on the Design Rainfall is:

WQV = 1.25-in/12 in/ft x 0.62 x (1.53 Ac) x 43,560 sq. ft./Ac = 4,306 cubic feet.

Since the WQV calculated to meet the TSS removal requirements is larger than the minimum WQV required for the site, it will be used for the sizing of the bioretention facility.

3.2.2.3.4: Example Development Site- Determine Size of the SCM

Using the larger water quality volume calculated above, a facility can be roughly sized. The bioretention facilities are sized using the required water quality volume increased in size by 20% (to account for sediment accumulation) and the required depth. Each of the bioretention facilities will have the same depth and porosity of materials. That calculation is shown below.

	Depth (ft)	Porosity (%)	Volume/ft²
Ponding	0.75	100	0.75
Media	3.0	30	0.90
Underdrain	1.0	40	0.40
Total	**4.75**		**2.05**

The volume per square foot is determined by multiplying the depth of the treatment layer by the porosity, and then dividing by 100.

The required surface area is determined by dividing the water quality volume for the site by the total volume per square foot. Therefore, the required size of bioretention for the site is:

Bioretention Area: 1.2 x (4,547 ft^3)/ 2.05 = 2,662 ft^2

The next step in the site design process would be to determine potential locations for bioretention and subdivide the site into sub-watersheds for each bioretention area. Then, the volume, capture and removal calculation would be performed for each bioretention area to verify the total capture volume and TSS removal.

In addition to improving the water quality, the City of San Marcos also requires that detention is provided on-site. According to the City code, "only the portion of the volume within a water quality control available after 24 hours of drawdown time may be used or credited towards detention requirements." (See Section 3.2.3: Integrated Stormwater Management). Detention requirements are outlined in Section §3.9.1.1.F of the SMDC.

Bioretention facilities are excellent at treating water quality volume, but they typically cannot detain the volume for the 100-year storm event. However, a hydrograph model will allow for optimizing the site design to maximize the size of the bioretention basins and reducing the size of the detention. Providing appropriate detention at the site could require sacrificing one of the lots in order to build a stormwater pond. Instead, the detention requirements at this site can be met by installing a berm or series of berms within the buffer zone, as shown on Exhibit D. Runoff can reach the detention area via pipes, which can convey the drainage for large storm events.

3.2.3: INTEGRATED STORMWATER MANAGEMENT

Sizing a detention facility to meet the SMDC requires utilizing a hydrograph model. Pre-construction and post-construction conditions should be inputted into the model. The post-construction curve numbers and/or impervious cover percentage should be modified to reflect the post-construction conditions including the benefits of the LID SCMs. A detailed list of appropriate curve numbers can be found in Appendix C: Stormwater Detention Guidance. Once the modified curve numbers are inputted into the hydrograph model, the detention basin can be sized appropriately.

The following sections present the design criteria for stormwater management ponds in the City of San Marcos.

3.2.3.1: Performance Criteria for SWM Ponds

a. Detention ponds shall be designed to reduce post-development peak rates of discharge to existing pre-development peak rates of discharge for the 2-, 10-, 25- and 100-year storm events at each point of discharge from the project or development site. For the post-development hydrologic design of the SWM pond, any off-site areas which drain to the pond shall be assumed to remain in the existing condition. If off-site flows are conveyed through the SWM pond, the SWM pond outlet structure must be designed to safely pass 100-year fully developed off-site flows in accordance with the Safety Criteria set forth in Section 3.2.1.8.

b. Computation of the rate of runoff must be based on an assumption of a fully developed contributing drainage area or watershed. Detention will be waived for single-family residential and small projects with less than 5,000 square feet of impervious cover, including buildings, parking lots and sidewalks.

c. For design purposes, any pond with a drainage area larger than 64 acres shall be classified as a regional pond. Performance criteria for regional ponds shall be reviewed and approved by the City on a project-by-project basis. The determination shall be based on a preliminary engineering study prepared by a licensed engineer in the State of Texas.

d. Maximum retention or "draw-down" time for flood detention ponds shall not exceed 24 hours from the time of peak storage to the time of complete emptying of the pond, as determined by hydrograph routing or other calculations acceptable to the City. This requirement does not apply to facilities in which retention or "draw-down" time is required to be greater than 24 hours. Only the portion of the volume within a water quality control available after 24 hours of drawdown time may be used or credited towards detention requirements.

e. Sediment markers are required for all SWM ponds.

3.2.3.2: Detention Pond Storage Determination

A flow routing analysis using detailed hydrographs must be applied for all detention pond designs. The Soil Conservation Service hydrologic methods (TR-20) and the Hydrologic Engineering Center (HEC) hydrologic methods (HEC-HMS) may be used. The engineer may use other methods but must have their acceptability approved by the Director.

3.2.3.3: Outlet Structure Design

There are two basic types of outlet control structures: those incorporating orifice flow and those incorporating weir flow. Rectangular and V-notch weirs are the most common types.

Generally, if the crest thickness is more than 60% of the nappe thickness, the weir should be considered broad-crested. The coefficients for sharp-crested and broad-crested weirs vary. The respective weir and orifice flow equations are as follows:

A. Rectangular Weir Flow Equation

$$Q = CLH^{(\frac{3}{2})} \quad \text{[Equation 3.10]}$$

Where

Q = Weir discharge, cubic feet per second

C = Weir Coefficient

L = Horizontal length, feet

H = Head on weir, feet

B. V - notch Weir Flow Equation

$$Q = C_v \tan(\Theta/2) H^{2.5} \quad \text{[Equation 3.11]}$$

Where

Q = Weir Flow, cubic feet per second

C_v = Weir Coefficient

Θ = Angle of the weir notch at the apex (degrees)

H = Head on Weir, feet

C. Orifice flow equation

$$Q = C_o A(2gH)^{0.5} \quad \text{[Equation 3.12]}$$

Where

Q = Orifice Flow, cubic feet per second

C_o = Orifice Coefficient (use 0.6)

A = Orifice Area, square feet

g = Gravitation constant, 32.2 feet/sec^2

H = Head on orifice measured from centerline, feet

Analytical methods and equations for other types of structures shall be approved by the Director of Engineering prior to use.

Additional information regarding outlet structure requirements can be found in Section 8.8 of Appendix B: LID Practices.

3.3: INFRASTRUCTURE AND RIGHT OF WAY

The City of San Marcos strives to protect citizens, public and private property through effective, economically viable, and reasonable drainage criteria. Integrated stormwater management discussed in Section 1.2, includes the approaches to comply with water quality and flood control provisions of the San Marcos Development Code (Section 3.9.1.). To protect infrastructure and ensure safe use of the public right of way, stormwater guidance in this manual aims to reach a balance between efficient transportation, public safety access, and cost. These goals require the storm drain system to manage large storm events without sustaining damage while still dealing with frequent small events that can cause nuisance drainage issues and local ponding in streets. In addition, the City of San Marcos desires to preserve natural drainage paths and creekways as an amenity for the community. Section 3.2.3 focuses on design of storm water infrastructure to meet onsite and offsite requirements for stormwater management that meet the larger San Marcos Drainage Master Plan goals.

3.3.1: STREET DRAINAGE

The drainage designs for all new or rehabilitated City streets should comply with the objectives and goals of the City's Drainage Master Plan — one of them being "to limit the impact of storm flows on roadway inundation." This manual provides adequate technical design criteria to limit flooding of roadways. Some of the design criteria includes the following:

- Drainage improvements, being curbs and gutters, inlets and storm sewers, shall be designed to intercept and convey all runoff from the 25-year frequency storm. Additionally, the 100-year frequency storm flows shall be conveyed within specifically defined rights-of-way or drainage easements (Section 3.9.1.1E).
- For bridges and culverts in residential and non-residential streets, the 100-year frequency storm runoff shall not result in a depth at the roadway greater than 12" and 6", respectively, or to the top of the upstream curb elevation, whichever is lower.
- Curb inlets and storm sewers shall be located and designed to minimize interference to traffic by reducing the depth and spread of water in the street section. The street crown will not be allowed to be lowered to obtain additional hydraulic capacity.
- For all new or rehabilitated City streets without curbing, all flows are to be totally contained within a parallel roadside channel or ditch. Additionally, all driveway and street crossing culverts shall have 6:1 end treatments in accordance with TxDOT standard details.
- Stormwater discharges from a public drainage system shall not exceed a velocity of six (6) feet per second without velocity dissipation and erosion protection measures.
- All drainage computations relating to new or rehabilitated construction of City streets shall be prepared by a Licensed Professional Engineer registered in the State of Texas.

3.3.2: STREET FLOW

3.3.2.1: General

The location of inlets and permissible flow of water in streets should be related to the extent and frequency of interference to traffic and the likelihood of flood damage to surrounding property for the 25- and 100-year frequency storms. Interference to traffic is regulated by design limits of the spread of water into traffic lanes, especially regarding arterials. Flooding of surrounding property from streets is controlled by limiting curb buildup to the top of curb for a 25-year storm which is designated as the design storm. Conveyance provisions for the 100-year storm must also be made within defined right of way and easements.

3.3.2.1.1: Interference Due to Flow in Streets

Water which flows in a street, whether from rainfall directly onto the pavement surface or overland flow entering from adjacent land areas, will flow in the gutters of the street until it reaches an overflow point or some outlet, such as a storm sewer inlet. As the flow progresses downhill and additional areas contribute to the runoff, the width of flow will increase and progressively encroach into the traffic lane. On streets where parking is not permitted, as with many arterial streets, flow widths exceeding one traffic lane become a traffic hazard. Field observations show that vehicles will crowd adjacent lanes to avoid curb flow.

As the width of flow increases, it becomes impossible for vehicles to operate without moving through water in an inundated lane. Splash from vehicles traveling in the inundated lane obscures the vision of drivers of vehicles moving at a higher rate of speed in

the open lane. Eventually, if width and depth of flow become great enough, the street loses its effectiveness as a traffic-carrier. During these periods, it is imperative that emergency vehicles such as fire trucks, ambulances and police cars be able to traverse the street by moving along the crown of the roadway.

3.3.2.1.2: Interference Due to Ponding

Storm runoff ponded on the street surface because of grade changes or because of the crown slope of intersecting streets has a substantial effect on the street-carrying capacity. The manner in which ponded water affects traffic is essentially the same as for curb flow; that is, the width of spread into the traffic lane is critical. Ponded water will often completely halt all traffic. Ponding in streets has the added hazard of surprise to drivers of moving vehicles, producing erratic and dangerous responses.

3.3.2.1.3: Street Cross Flow

Whenever storm runoff, other than limited sheet flow, moves across a traffic lane, a serious and dangerous impediment to traffic flow occurs. Cross-flow is allowed only in case of superelevation of a curve or overflow from the higher gutter on a street with cross fall. No more than three (3) cubic feet per second for the 25-year storm shall be allowed to cross flow from the higher elevation to the lower elevation.

3.3.2.1.4: Allowable Flow of Water Through Intersections

As the stormwater flow approaches a street or tee intersection (except alleys), an inlet is required if more than three (3) cubic feet per second for the 25-year storm shall flow across the intersection. No more than three (3) cfs for the 25-year storm shall be allowed to enter the bulb of a cul-de-sac or corner bubble. In both situations, the inlet cannot be placed inside the curb return.

3.3.2.1.5: Valley Gutter

Valley gutters can be useful in diminishing the deterioration of pavements, particularly at intersections where flows tend to concentrate. At the intersection of two (2) arterial streets, a valley gutter cannot be used. At the intersection of two (2) collector streets or local streets, the valley gutter may be used. At an intersection of two (2) different types of streets, the valley gutter may be used across the smaller street only.

3.3.2.2: Permissible Spread of Water

The flow of water in gutters of typical streets during the 25-year storm shall be contained below the top of curb and shall maintain the clear width requirements listed in Table 3.11. The flow of water shall be limited to a maximum of 6" above the top of crown during the 100-year storm event and must be contained within the defined right-of-way and easements. These clear widths at the crown of the roadway or at the high point on a divided roadway are necessary to provide access for vehicles in the event of an emergency.

TABLE 3.11 MINIMUM CLEAR WIDTHS FOR ROADWAY DESIGN WHEN GUTTER IS FLOWING FULL

	Roadway Type	Minimum Clear Width (ft)
	Local	0
Collector	Residential or Neighborhood	12
	All others	12 in each direction
Arterial	4 lanes one way or 8 lanes divided	24 in each direction
	All others	12 in each direction

Equation 3.13 (from equations presented in HEC-22, FHWA-NHI-10-009, modified to solve for depth rather than flow or spread) may be used to determine the depth of gutter flow for a specific street width and flow spread.

$$y = 4H(S/W) - 4H(S^2/W^2) \quad \text{[Equation 3.13]}$$

Where,

y = Water depth in the gutter (feet) (limited to 0.5 feet)

W = Street width (feet)

H = Crown height (feet)

S = Spread on one side of street (feet)

3.3.2.3: Design Method

3.3.2.3.1: Gutter Flow Velocities

To ensure scouring velocities for low flows, the gutter shall have a minimum slope of 0.004 feet per foot.

3.3.2.3.2: Straight Crowns

Flow in gutters on straight crown pavements is normally assumed to be uniform, with Manning's Equation being used to determine the flow. However, because the hydraulic radius assumption in the Manning's Equation is not able to adequately describe the hydraulic characteristics of the gutter cross section, modification of the equation is necessary to accurately compute the flow. The modified Manning's Equation is (HEC-22 Manual, FHWA-NHI-10-009):

$$Q_o = 0.56\ (z/n)\ S_o^{1/2}\ Y_o^{8/3} \qquad \text{[Equation 3.14]}$$

Where,

Q_o = Gutter discharge, cfs

z = Reciprocal of the crown slope, ft/ft

S_o = Street or gutter slope, ft/ft

n = Roughness coefficient

Y_o = Depth of flow in gutter, feet

The nomograph in Figure D.1 in Appendix D of this manual provides a direct solution for flow conditions in triangular channels. For a concrete pavement gutter, an n value equal to 0.016 is recommended. For gutters with small slope (less than one (1) percent) where sediment may accumulate, an n value of 0.02 is recommended.

3.3.2.3.3: Parabolic Crowns

Flows in the gutter of a parabolically crowned pavement are calculated from a variation of Manning's Equation, which assumes steady flow in a prismatic open channel. However, this equation is complicated and difficult to solve for each design case.

To provide a means of determining the flow in the gutter, generalized gutter flow equations for combinations of parabolic crown heights, curb splits and street grades of different street widths have been prepared. All of these equations have a logarithmic form.

Note: The street width used in this section is measured from face-of-curb to face-of-curb.

Streets Without Curb Split. Curb split is the vertical difference in elevation between curbs at a given street cross section. The gutter flow equation for parabolic crown streets without any curb split is:

$$\log Q = K_o + K_1 \log S_o + K_2 \log y_o \qquad \text{[Equation 3.15]}$$

Where,

Q = Gutter flow, cfs

S_o = Street grade, ft/ft

Y_o = Water depth in the gutter, feet

K_o, K_1, K_2 = Constant coefficients shown in Table 3.12 for different street widths:

Table 3.12 Coefficients for Equation 3.15, Streets Without Curb Split

	Coefficients		
Street Width* (ft)	K_0	K_1	K_2
30	2.85	0.50	3.03
36	2.89	0.50	2.99
40	2.85	0.50	2.89
44	2.84	0.50	2.83
48	2.83	0.50	2.78
60	2.85	0.50	2.74

*Note: Based on the Transportation Criteria Manual, the street width is measured from face-of-curb to face-of-curb (FOC-FOC).
Source: City of Austin, Watershed Engineering Division

Streets with Curb Split - Higher Gutter. The gutter flow equation for calculating the higher gutter flows is as follows:

$$\log Q = K_0 + K_1 \log S_0 + K_2 \log y_0 + K\ 3\ (CS) \qquad \text{[Equation 3.16]}$$

Where,

Q = Gutter flow, cfs

S_0 = Street grade, ft/ft

Y_0 = Water depth in the gutter, feet

CS = Curb split, feet

K_o, K_1, K_2, K_3 = Constant coefficients shown in Table 3.13 for different street widths:

TABLE 3.13 COEFFICIENTS FOR EQUATION 3.16, STREETS WITH CURB SPLIT – HIGHER GUTTER

STREET WIDTH* (FT)	COEFFICIENTS				CURB SPLIT RANGE (FT)
	K_0	K_1	K_2	K_3	
30	2.85	0.50	3.03	-0.131	0.0-0.6
36	2.89	0.50	2.99	-0.140	0.0-0.8
40	2.85	0.50	2.89	-0.084	0.0-0.8
44	2.84	0.50	2.83	-0.091	0.0-0.9
48	2.83	0.50	2.78	-0.095	0.0-1.0
60	2.85	0.50	2.74	-0.043	0.0-1.2

Source: City of Austin, Watershed Engineering Division

Streets with Curb Split - Lower Gutter. The gutter flow equation for the lower gutter is:

$$\log Q = K_0 + K_1 \log S_0 + K_2 \log y_0 + K_3 (CS) \quad \text{[Equation 3.17]}$$

Where,

Q = Gutter flow, cfs

S_0 = Street grade in ft/ft

Y_0 = Water depth in the gutter in feet

CS = Curb split in feet

K_0, K_1, K_2, K_3 = Constant coefficients shown in Table 3.14 for different street widths:

TABLE 3.14 COEFFICIENTS FOR EQUATION 3.17, STREETS WITH CURB SPLIT – LOWER GUTTER

STREET WIDTH* (FT)	COEFFICIENTS				CURB SPLIT RANGE (FT)
	K_0	K_1	K_2	K_3	
30	2.70	0.50	2.74	-0.215	0.0-0.6
36	2.74	0.50	2.73	-0.214	0.0-0.8
40	2.75	0.50	2.73	-0.198	0.0-0.8
44	2.76	0.50	2.73	-0.186	0.0-0.9
48	2.77	0.50	2.72	-0.175	0.0-1.0
60	2.80	0.50	2.71	-0.159	0.0-1.2

Source: City of Austin, Watershed Engineering Division

Parabolic Crown Location. The gutter flow equation presented for parabolic crowns with split curb heights is based on a procedure for locating the street crown. The procedure allows the street crown to shift from the street center line toward the high ¼ point of the street in direct proportion to the amount of curb split. The maximum curb split occurs with the crown at the ¼ point of the street. The maximum allowable curb split for a street with parabolic crowns is 0.02 feet per foot of street width.

Example 4-1-1

Determination of Crown Location

Given: 0.4 feet Design split on 30-foot wide street.

Maximum curb split = 0.02 × street width

= 0.02 × 30 feet = 0.6 feet

Maximum Movement = ¼ street width for 30-foot street

= ¼ × 30 feet = 7.5 feet

Split Movement = (Design split × W/Maximum Split × 4)

= (0.4 x 30/.6 × 4) = 5 feet

Curb splits that are determined by field survey, whether built intentionally or not, should be considered when determining the capacity of the curb flow.

Special consideration should be given when working with cross sections which have the pavement crown above the top of curb. When the crown exceeds the height of the curb the maximum depth of water is equal to the height of the curb, not the crown height. It should be noted that a parabolic section where the crown equals the top of curb will carry more water than a section which has the crown one (1) inch above the top of curb.

3.3.3: INLETS

Where inlets are proposed, the City of San Marcos standard inlets with adequate reinforcing steel shall be used. All other types or designs shall be subject to the approval of the Director of Engineering. This Chapter describes the considerations and design criteria for different types of storm water inlets.

3.3.3.1: INLET TYPES

Inlets may be incorporated into LID design per Appendix B.

3.3.3.1.1: Curb Inlet

A Curb Inlet is a vertical opening in the curb covered by a top slab or the upper unit. The City of San Marcos standard details shall be used.

3.3.3.1.2: Grate Inlet

A Grate Inlet is a concrete box with a metal grate on the top that sits on the box. Use of the materials and following the dimensions that are called out in the City of San Marcos Standard Detail is recommended. Any modifications to the standard details will need to be approved by the Director of Engineering. Additionally, where bicycle traffic occurs, grates should be bicycle safe.

3.3.3.1.3: 4-Way Inlet

A 4-Way Inlet has four sides with normal six (6) inch high rectangular openings, with a 6-inch concrete top slab and a minimum 5-foot Concrete Apron around the inlet. Use of the materials and following the dimensions that are called out in the City of San Marcos Standard Detail is recommended. The maximum height of opening for the inlet is nine (9) inches, unless approved by the Director of Engineering.

3.3.3.1.4: Combination Curb Inlet and Grate Inlet

A Combination Curb Inlet and Grate is an Inlet that has a curb opening with a concrete slab and a metal grate in the gutter portion of the section. These inlets are very useful in sag conditions because if clogged by debris the curb openings act as relief valves for the clogged grates.

3.3.3.1.5: Combination Grate and 4-Way Inlet

A Combination Grate and 4-Way inlet is the combination of the inlet described in part 8.2.3 with a grate in place of the top slab. These inlets are very useful in sag conditions where flow approaches a low spot from multiple directions.

3.3.3.1.6: Drop Curb Opening

A Drop Curb Opening is a cut in the curb in order to allow water to drain off the roadway and into a drainage swale. The Curb Openings are located where there is no cover over the opening and the sidewalk does not abut the curb. These inlets are also known as over-side drains or curb slots. The openings come in a variety of forms from metal curb lines to concrete saw tooth openings. In most cases, an opening in the curb connects to a scour-resistant channel or concrete chute to prevent erosion.

3.3.3.1.7: Sidewalk Drains

A Sidewalk Drain is a rectangular opening across a pathway typically made of concrete with a steel cover to convey runoff from one side to the other of the pathway. These types of inlets are typically used across sidewalks in the downtown area to capture the roof runoff from downspouts and discharge them onto the roadways. They can be used in other situations; a typical detail can be found on the City of San Marcos Standard Miscellaneous Details I in the Roadway Standards.

3.3.3.1.8: Slotted Drain

A Slotted Drain Inlet is composed of a length of usually circular pipe with a slot cut in the top on which a metal grate opening is mounted on. The grate is flush with the pavement at the surface and the throat is reinforced for structural integrity. The designer should ensure structural integrity if used along a roadway.

3.3.3.2: DESIGN GUIDELINES

The minimum design frequency for storm drain inlets is based on the maximum design frequency of the infrastructure that is being conveyed to the inlet or opening. Inlets and openings will be located and sized to meet the design criteria of the roadways they service, the ponds they drain to and from, and other drainage system conveyance features that they are a part of. The following formulas for inlet capacities and design guidelines are based on drop inlets on grade and at sag points. Inlet capacities for on grade inlets are less than that of inlets in sump. The capacity of on grade inlets depends on street grades, deflections, cross slopes, depressions, etc. The capacity of inlets in a sag are dependent on the water depth at the curb opening and the height of the curb opening.

3.3.3.2.1: Curb Inlets on Grade

The capacity of inlets on grade will depend on interception capacity and the amount of carry over that is allowed. If curb inlet extensions are used with the curb inlet, they shall be place on the up-gradient end of the curb inlet. If more than one extension is proposed, then verification of the hydraulic capacity of the block out openings will be required to verify that the extensions have sufficient capacity to convey the required design storm to the primary curb inlet. The minimum length of curb inlet opening is 5 feet.

The following procedure is used to design curb inlets on-grade:

Compute depth of flow and ponded width (T) in the gutter section at the inlet.

Determine the ratio of the width of flow in the depressed section (W) to the width of total gutter flow (T) using Equation 3.18. Figure D.6 shows the gutter cross section at an inlet.

$$E_o = K_w / (K_w + K_o) \quad \text{[Equation 3.18]}$$

E_o = ratio of depression flow to total flow

K_w = conveyance of the depressed gutter section (cfs)

K_o = conveyance of the gutter section beyond the depression (cfs).

Use Equation 3.19 to calculate conveyance, K_w and K_o.

$$K = \frac{zA^{5/3}}{nP^{2/3}} \quad \text{[Equation 3.19]}$$

K = conveyance of cross section (cfs)

z = 1.486

A = area of cross section (sq.ft.)

n = Manning's roughness coefficient

P = wetted perimeter (ft.).

Use Equation 3.20 to calculate the area of cross section in the depressed gutter section.

$$A_w = WS_x (T - W/2) + 1/2\, aW \quad \text{[Equation 3.20]}$$

A_w = area of depressed gutter section (sq.ft.)

W = depression width for an on-grade curb inlet (ft.)

S_x = cross slope (ft./ft.)

T = calculated ponded width (ft.)

a = curb opening depression depth (ft.).

Use Equation 3.21 to calculate the wetted perimeter in the depressed gutter section.

$$P_w = ((wS_x + a)^2 + W^2)^{1/2} \quad \text{[Equation 3.21]}$$

P_w = wetted perimeter of depressed gutter section (ft.)

W = depression width for an on-grade curb inlet (ft.) Sx = cross slope (ft./ft.)

a = curb opening depression depth (ft.).

Use Equation 3.22 to calculate the area of cross section of the gutter section beyond the depression.

$$A_o = (\frac{S_x}{2})(T - w)^2 \quad \text{[Equation 3.22]}$$

A_o = area of gutter/road section beyond the depression width (ft2)

S_x = cross slope (ft./ft.)

W = depression width for an on-grade curb inlet (ft.)

T = calculated ponded width (ft.)

Use Equation 3.23 to calculate the wetted perimeter of the gutter section beyond the depression.

$$P_o = T - W \quad \text{[Equation 3.23]}$$

P_o = wetted perimeter of the depressed gutter section (ft.)

T = calculated ponded width (ft.)

W = depression width for an on-grade curb inlet (ft.).

Use Equation 3.24 to determine the equivalent cross slope (Se) for a depressed curb opening inlet.

$$S_e = S_x + (\frac{a}{W})E_o \quad \text{[Equation 3.24]}$$

S_e = equivalent cross slope (ft./ft.)

S_x = cross slope of the road (ft./ft.)

a = gutter depression depth (ft.)

W = gutter depression width (ft.)

E_o = ratio of depression flow to total flow.

Calculate the length of curb inlet required for total interception using Equation 3.25

$$L_r = zQ^{0.42} S^{0.3} (1/nS_e)^{0.6}$$ [Equation 3.25]

L_r = length of curb inlet required (ft.)

z = 0.6

Q = flow rate in gutter (cfs)

S = longitudinal slope (ft./ft.)

n = Manning's roughness coefficient

S_e = equivalent cross slope (ft./ft.).

If no bypass flow is allowed, the inlet length is assigned a nominal dimension of at least Lr, which should be an available (nominal) standard curb opening length. The exact value of Lr should not be used if doing so requires special details, special drawings, structural design, and costly construction.

If bypass flow is allowed, the inlet length is rounded down to the next available standard (nominal) curb opening length.

Determine bypass flow. In bypass flow computations, efficiency of flow interception varies with the ratio of actual length of curb opening inlet supplied (L_a) to required length (L_r) and with the depression to depth of flow ratio. Use Equation 3.26 to calculate bypass flow.

$$Q_{co} = Q(1 - L_a/L_r)^{1.8}$$ [Equation 3.26]

Q_{co} = carryover discharge (cfs)

Q = total discharge (cfs)

L_a = design length of the curb opening inlet (ft.)

L_r = length of curb opening inlet required to intercept the total flow (ft.)

In all cases, the bypass flow must be accommodated at some other specified point in the storm drain system.

Calculate the intercepted flow as the original discharge in the approach curb and gutter minus the amount of bypass flow.

3.3.3.2.2: Curb Inlets in Sump

To calculate the capacity of a Curb Inlet you must first calculate if the inlet is fully submerged or partially full. This will depend on the depth of flow at the curb. If the depth of flow at the curb opening inlet is such as to cause a partially full opening, a weir effect will develop, and the following formula will apply. The minimum length of curb inlet opening is 5 feet.

$$Q = CL(h)^{3/2}$$ [Equation 3.27]

Q = amount of flow in CFS

C = the weir coefficient 3.087

L = the length of drop curb opening required in feet.

h = the head or depth of water at the opening in feet (should include inlet depression 'a').

If the depth of flow at the Curb Inlet is such as to fully submerge the opening, the orifice effect will develop, and the formula used shall be identical to that given under grate inlets with the exception that the head, h, on the curb opening orifice shall be taken as the depth from the top of the water surface to the center of orifice or opening; one hundred (100) percent efficiency will be allowed for curb opening inlets.

$$Q = CA(2gh)^{1/2}$$ [Equation 3.28]

Q = discharge in cubic feet per second.

C = orifice coefficient of discharge (taken as 0.70).

g = acceleration due to gravity (32.2 ft./sec.^{2})

h = head on the opening in feet from top of water surface to the center of opening A = net area of the openings in square feet.

Curb Inlet Extensions are not allowed in sump conditions. The Hydraulic Grade Line shall be designed below the gutter elevation of the drainage structure with the Energy Grade Line below the top of curb.

3.3.3.2.3: Grate Inlets on Grade

The interception capacity of Grate Inlets on grade depends on the cross slope, longitudinal slope, depth of flow, Manning's Roughness coefficient, and the net area of grate opening. The depth of water over the grate shall be calculated based on the cross section. A clogging factor will be applied to Grate Inlets on grade based on Equation 3.2.3.h below.

Use the following procedure for grate inlets on-grade:

1. Compute the ponded width of flow (T).
2. Choose a grate type and size.

3. Find the ratio of frontal flow to total gutter flow (Eo) for a straight cross-slope using Equation 3.18. No depression is applied to a grate on-grade inlet.

4. Find the ratio of frontal flow intercepted to total frontal flow, Rf, using Equations 3.29, 3.30, and 3.31.

If $v > v_o$, use Equation 3.29

$$R_f = 1 - K_u(v - vo) \quad \text{[Equation 3.29]}$$

If v < vo, Equation 3.30

$$R_f = 1.0 \quad \text{[Equation 3.30]}$$

R_f = ratio of frontal flow intercepted to total frontal flow

Ku = 0.09

v = approach velocity of flow in gutter (ft./s.)

v_o = minimum velocity that will cause splash over grate (ft./s.)

For triangular sections, calculate the approach velocity of flow in gutter (v) using Equation 3.31

$$V = 2Q/T_y = 2Q/(T^2 S_x) \quad \text{[Equation 3.31]}$$

v = approach velocity of flow in gutter (ft./s)

Q = flow rate in gutter (cfs)

S_x = cross slope of the road (ft./ft.)

T = calculated ponded width (ft.)

T_y = max ponded depth (ft.)

Otherwise, compute the section flow area of flow and calculate the velocity using Equation 3.32.

V = Q/A [Equation 3.32]

v = approach velocity of flow in gutter (ft./s)

Q = flow rate in gutter (cfs)

A = the section flow area of flow (ft2)

Calculate the minimum velocity (v_o) that will cause splash over the grate using the appropriate equation in Table 3.15.

TABLE 3.15 SPLASH-OVER VELOCITY CALCULATION EQUATIONS (ENGLISH)

GRATE CONFIGURATION	TYPICAL BAR SPACING (IN.)	SPLASH-OVER VELOCITY EQUATION
Parallel Bars	2	v_o = 2.218+4.031L-0.649L2+0.056L3
Parallel Bars	1.2	v_o = 1.762+3.11L-0.451L2+0.033L3
Parallel bars w/ transverse rods	2 parallel/ 4 transverse	v_o = 0.735+2.437L-0.265L2+0.018L3

v_o = splash-over velocity (ft./s or m/s)

L = length of grate (ft.)

5. Find the ratio of side flow intercepted to total side flow, Rs.

$$R_s = [1 + (zv^{1.8})/(S_x L^{2.3})]^{-1} \quad \text{[Eqution 3.33]}$$

R_s = ratio of side flow intercepted to total flow

z = transverse slope

v = approach velocity of flow in gutter (ft./s.)

L = length of grate (ft.).

6. Determine the efficiency of grate, Ef.

$$E_f = [R_f E_o + R_s(1 - E_o)] \quad \text{[Equation 3.34]}$$

R_f = ratio of frontal flow intercepted to total frontal flow

E_o = ratio of depression flow to total flow.

R_s = ratio of side flow intercepted to total flow

7. When calculating the interception capacity of the grate, Q_i a reduction factor (C) due to clogging should be included. If the interception capacity is greater than the design discharge, skip step 9.

$$Q_i = CE_f Q = CQ[R_f E_o + R_s(1 - E_o)] \quad \text{[Equation 3.35]}$$

C= Clogging Factor (see Equation 3.36)

E_f = ratio of grate efficiency

R_f = ratio of frontal flow intercepted to total frontal flow

E_o = ratio of depression flow to total flow.

R_s= ratio of side flow intercepted to total flow

8. Calculate the clogging factor for grate inlets on grade with multiple units using the equation below.

$$C = KC_o/N \quad \text{[Equation 3.36]}$$

C= Multiple Unit Clogging Factor for an inlet with multiple units

C_o = single unit clogging factor (50%)

e = decay ratio (0.5 for grate inlets)

N= number of units

K= clogging coefficient from Table 3.16 below

TABLE 3.16 CLOGGING COEFFICIENTS FOR MULTIPLE UNITS

N =	1	2	3	4	5	6	7	8	>8
GRATE INLET (K)	1.0	1.5	1.8	1.9	1.9	2.0	2.0	2.0	2.0

9. Determine the bypass flow (C_0) using this Equation. Remember to include the varying clogging factor for grate inlets in series.

$$Q = CA(2gh)^{1/2} \quad \text{[Equation 3.37]}$$

10. Depending on the bypass flow, select a larger or smaller inlet as needed. If the bypass flow is excessive, select a larger configuration of inlet and return to step 3. If the interception capacity far exceeds the design discharge, consider using a smaller inlet and return to step 3.

3.3.3.2.4: Grate Inlets In Sump

Grates should be designed assuming a clogging factor of 50%. When calculating the capacity of a grate inlet the net area of opening should be used, minus 50% for the clogging assumed above when calculating its capacity. The flow of water through grate openings may be treated as the flow of water through a rectangular orifice. Use Equation 3.38 to calculate the inlet capacity.

$$Q = CA(2gh)^{1/2} \quad \text{[Equation 3.38]}$$

Q = discharge in cubic feet per second.

C = orifice coefficient of discharge (taken as 0.70).

g = acceleration due to gravity (32.2 ft./sec.^{2})

h = head on the grate in feet.

A = 50% of net area of the openings in the grate in square feet.

3.3.3.2.5: 4-Way Inlet

4-Way Inlets are typically proposed in sump situations. If they are fully submerged, then use Equation 3.38 to calculate the inlet capacity. The head, h, on the inlet opening orifice shall be taken as the depth from the top of the water surface to the center of orifice or opening.

3.3.3.2.6: Combination Curb Inlet and Grate Inlet

The capacity of the grate portion should be calculated based on whether the inlet is fully submerged or it is partially submerged. If the grate portion of the inlet is fully submerged then the capacity will be calculated using the orifice equation (assuming 50% clogging). If the grate portion of the inlet is not partially submerged then it should be calculated as a weir. The capacity of the curb inlet opening portion will also be calculated using the orifice equation if it is fully submerged or by using the weir equation if is not. The depth of flow along the curb needs to be calculated prior to making the determination of partially submerged or fully submerged. The minimum length of curb inlet is 5 feet.

3.3.3.2.7: Combination Grate and 4-Way Inlet

The capacity of the grate portion should be calculated based on whether the inlet is fully submerged or it is partially submerged. If the grate portion of the inlet is fully submerged then the capacity will be calculated using the orifice equation (assuming 50% clogging). If the grate portion of the inlet is not partially submerged then it should be calculated as a weir. The capacity of the curb inlet openings should also be calculated using the orifice equation if they are fully submerged or by using the weir equation if they are not. The depth of flow approaching the combination inlet needs to be calculated prior to making the determination of partially submerged or fully submerged.

3.3.3.2.8: Drop Curb Opening on Grade

A drop curb opening is a curb opening where there is no cover over the opening and the sidewalk does not abut the curb opening. Where drop curb openings are used to take storm water off the streets and into drains, the length of the curb opening can be calculated following the steps in Section 3.3.3.2.1 above. The minimum length of curb opening is 5 feet.

3.3.3.2.9: Drop Curb Opening in Sump

A drop curb opening is a curb opening where there is no cover over the opening and the sidewalk does not abut the curb opening. Where drop curb openings are used to take storm water off the streets and into drains, the length of the curb opening can be calculated from the weir formula using the coefficient of 3.087 using Equation 3.27. The minimum length of curb opening is 5 feet.

Gutter line depressions will be permitted where such depressions will not hamper the flow of traffic. For amount of curb exposure, conform to City of San Marcos inlet standards.

3.3.3.2.10: Sidewalk Drains

Slope of a Sidewalk Drain should match the slope of the sidewalk that it crosses. The capacity of the opening will be determined by using Manning's Equation.

3.3.3.2.11: Slotted Drain

The throat of a slotted drain should be reinforced. The amount of reinforcement will be dependent on the anticipated loads that it will be subject to. Slotted drains should be oriented parallel to the flow so as to maximize the hydraulic efficiency. Slotted drains will only be allowed within public rights-of-way with the approval of the Director of Engineering. The capacity of slotted drains will be calculated using the orifice.

3.3.3.3: MATERIALS AND SPECIFICATIONS

3.3.3.3.1: Cast In Place

Cast in Place Inlets shall meet all the requirements found in latest version of the City of San Marcos Standard Specifications' Item 307 "Concrete Structures".

3.3.3.3.2: Pre-Cast

Pre-Cast Inlets shall meet all the requirements found in latest version of the City of San Marcos Standard Specifications' Item 403 "Storm Drain Junction Boxes and Inlets" or ASTM C478.

3.3.3.3.3: Minimum Structural Loads

The minimum live load should be HS 20 for streets and E 80 for railways. Heaver live load may be needed in special cases, and the designer should determine the required live load.

3.3.3.3.4: Grate

All Grates should meet all the requirements found in the latest version of the City of San Marcos Standard Specifications' Item 407 "Cast Iron Castings". Steel Grates and Frames need to be galvanized with hold down bolts.

3.3.3.3.5: Sidewalk plates

Sidewalk plates can be found on the City of San Marcos Standard Roadway Details "Miscellaneous Construction Standards 1".

3.3.3.3.6: Sidewalk Pipe Railing

All Sidewalk Pipe Railing shall be made of Galvanized Steel Pipe and shall conform to the requirements of the Standard Specifications for Steel for Bridges and Buildings, ASTM A 36, or approved equal. Additional specifications can be found in the latest version of the City of San Marcos Standard Specifications' Item 522 "Sidewalk Pipe Railing".

3.3.3.3.7: Mud Slab

A mud slab is a base slab of low strength concrete used to level up or stabilize the bottom of an excavation for the placement of multiple inlets, multiple boxes or other structures. The mud slab is from two (2) to six (6) inches thick, or thicker if needed. The mud slab shall be wide enough and long enough to encompass all proposed inlet bottoms.

3.3.4: STORM DRAIN SYSTEMS

3.3.4.1: INTRODUCTION

The street system, roadside ditch, swale or channel may direct flow into an inlet, grate, or other collection structure into the storm drain system. This storm drain system will be comprised of inlets, pipes, junction boxes, bends, outlets, and other appurtenances. These systems may include water quality devices to meet state and federal water quality standards. This chapter describes the general guidelines needed to provide an adequate storm drain system and minimize impacts to both upstream and downstream properties.

The following shall be considered during the design of the storm drain systems.

For all ordinary conditions, storm drains shall be designed on the assumption that they will flow full under the design discharge; however, when there are constrictions, turns, submerged, or

inadequate outfall, etc., the hydraulic and energy grade lines shall be computed and plotted in profile. The Energy Grade Line (EGL) shall be below the top of curb and the Hydraulic Grade Line (HGL) shall be below the gutter elevation of the drainage structure. In all cases adequate outfalls shall be provided, including review of point source discharges.

The EGL and HGL will be required on all storm drain systems.

3.3.4.2: Hydraulics of Storm Drainage Systems

3.3.4.2.1: Flow Type Assumptions

The design procedures assume that the flow within each segment of the underground drainage system is steady and uniform. Also, the average velocity within each segment is considered to be constant.

3.3.4.2.2: Partial Flow vs. Pressure Flow

There are two types of considerations for sizing storm drain lines under steady uniform flow assumption. The first is referred to as partial or open channel flow design. The flow depth within the conduit is less than the height of the conduit; so the HGL will be within the conduit. The second is referred to as pressure flow design or full flow. The conduit is fully flowing, and the HGL may be at the soffit or above the soffit of the conduit. See Figure 3.7

For partial flow the design engineer should check for possible hydraulic jumps within the system if the flow is supercritical. If a hydraulic jump occurs within the system, the upper conduits could change to pressure flow or the hydraulic jump could move downstream within the conduit to another design point.

The design may have both partial and pressure flow segments within the same drainage system.

The drainage system should be designed for full flow as this will increase the efficiency of the storm drain system.

3.3.4.2.3: Hydraulic Capacity

The hydraulic capacity is controlled by the conduits size, shape, and frictional resistance. Use Manning's Formula for the design of all conduits.

$Q = AV$ [Equation 3.39]

Q = Flow (cfs)

A = cross section area (sq. ft.9

V = velocity of flow (ft./sec.)

$Q = (1.49)/n\ AR^{0.67}\ S_f^{0.5}$ [Equation 3.40]

Q = Flow (cfs)

A = cross section area (sq. ft.)

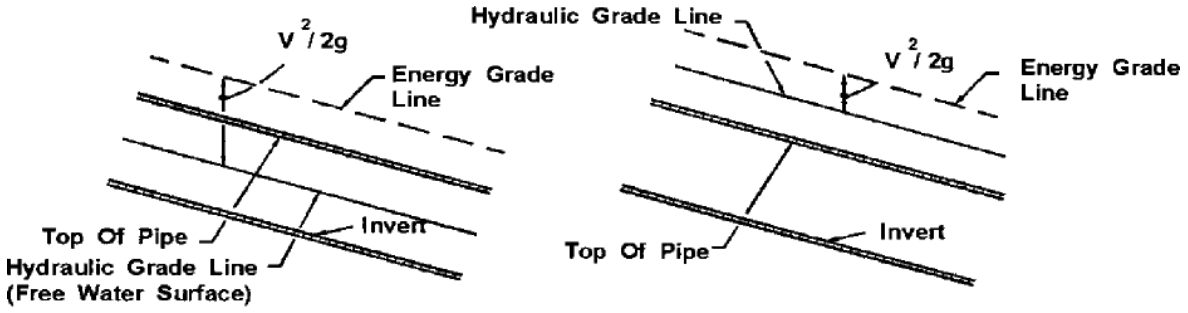
n = roughness coefficient of conduit

R = hydraulic radius = A/W_P (ft.)

W_P = wetted perimeter (ft.)

S_f = frictional slope of conduit (ft./ft.)

The "n" value to be used in Manning's Formula shall conform to Table 3.17 for design purposes. Any other "n" value shall be based on generally accepted engineering principles.



a. Open Channel Flow b. Pressure Flow

FIGURE 3.7 HYDRAULIC AND ENERGY GRADE LINE IN A CONDUIT (SOURCE FHWA HEC NO. 22)

TABLE 3.17 MANNING'S ROUGHNESS COEFFICIENT

TYPE OF CULVERT	ROUGHNESS OR CORRUGATION	MANNING'S N
Concrete Pipe	Smooth	0.013
Concrete Box	Smooth	0.013
Corrugated Metal Pipe, Pipe-Arch - Unpaved	2-2/3 by 1/2 inch Annular	0.024
Corrugated Metal Pipe, Pipe-Arch - Unpaved	3 by 1 inch Annular	0.027

3.3.4.2.4: Hydraulic Grade Line and Energy Grade Line

The HGL is the water surface of an open channel or the water surface of a conduit with partial flow. For a conduit with pressure flow, the HGL would be the level of water surface that would rise within a vertical tube at any point along the conduit.

The EGL is an imaginary line that is the measure of total energy along the open channel or conduit carrying water. This total energy includes elevation head, velocity head, and pressure head. The EGL is a velocity head ($V^2/2g$) above the HGL. The EGL is always increasing in the upstream conduit. The EGL should not be above the finished grade, or top of curb, at any point along the conduit.

3.3.4.2.5: Storm Dain Outfalls

The outfall for the storm drain system should discharge into a natural low, existing storm drainage system, or a channel. The start of the EGL for the storm drain system begins at the outfall. The design engineer should determine the tail water for the downstream drain to find the impact on the proposed outfall. There are two conditions for determining the starting point for the HGL at the outfall. The tail water may be above the critical depth of the outfall conduit or between the critical depth and invert of the outfall conduit. The starting point for the HGL should be either the tail water elevation or the average of critical depth plus the height of the storm drain conduit, whichever is greater. The design engineer will consider an exit loss at the outlet.

If the outfall of the storm drain system is into a river, stream, or creek, the design engineer should consider the coincidental probability of the peaks of both systems occurring at the same time. The tail water for the receiving stream should be checked with the peak of the storm drain system.

3.3.4.2.6: Energy Losses

The energy losses for the storm drain system include frictional, exit, entrance, bend, and manhole and junction losses. These losses add to the hydraulic gradient along the storm drain system.

3.3.4.2.6.1: Pipe Friction Losses

The frictional loss from the conduit is one of the losses. The head loss due to friction is determined by the following formulas:

$$H_f = S_f(L) \qquad \text{[Equation 3.41]}$$

H_f = Friction Loss (ft)

S_f = Friction slope (ft./ft.)

L = Length of pipe (ft.)

Should the conduit have partial flow, then the frictional slope will match the pipe slope. For pressure flow or full flow of the conduit, the formula below can be used to determine the frictional slope of the conduit:

$$Sf = [Q(n/1.49)/(AR^{2/3})]^2 \qquad \text{[Equation 3.42]}$$

S_f = Friction slope (ft./ft.)

Q = flow (cfs.)

n = roughness coefficient of conduit

A = cross section area (sq. ft.)

R = hydraulic radius = A/W_P (ft.)

W_P = wetted perimeter (ft.)

3.3.4.2.6.2: Exit Losses

The exit loss from the storm drain outlet should be determined by the following formula:

$$H_o = 1.0[(V_o^2/2g) - (V_d^2/2g)] \qquad \text{[Equation 3.43]}$$

H_o = Exit loss (ft.)

V_o = Velocity of outfall conduit (fps)

g = Acceleration due to gravity (ft./s^2 (32.2 ft./s^2))

V_d = Velocity of downstream channel (fps)

3.3.4.2.6.3: Bend Losses

This loss is for a bend located in the conduit run and not at a junction or manhole structure.

Use the following formula to determine the bend loss:

$H_b = 0.0033(\Delta)(V^2/2g)$ [Equation 3.44]

H_b = Bend loss (ft.)

Δ = Angle of curvature in degrees

V = Velocity of the conduit (fps)

g = Acceleration due to gravity (ft./s^2 (32.2 ft./s^2))

3.3.4.2.6.4: Transition Losses

These losses are used where box culverts have a transition in width, height, or both width and height. The energy loss for expansions or contraction in open channel or partial flow must use the following formulas:

$$H_c = K_c[(V1^2/2g) - (V2^2/2g)]$$

$$H_e = K_e[(V1^2/2g) - (V2^2/2g)]$$

[Equation 3.45]

H_c = Contraction loss (ft.)

H_e = Expansion loss (ft.)

K_c = Contraction coefficient (0.5 K_e)

K_e = Expansion coefficient

V1 = Velocity upstream of transition (fps)

V2 = Velocity downstream of transition (fps)

g = Acceleration due to gravity (ft./s^2 (32.2 ft./s^2))

Energy loss for expansions or contraction in pressure flow conditions will use the following formulas:

$$H_c = K_c(V1^2/2g)$$

$$H_e = K_e[(V1^2/2g)$$

[Equation 3.46]

H_c = Contraction loss (ft.)

H_e = Expansion loss (ft.)

K_c = Contraction coefficient (Table 3.19)

K_e = Expansion coefficient (Table 3.18 and Table 3.20)

V1 = Velocity upstream of transition (fps)

V2 = Velocity downstream of transition (fps)

g = Acceleration due to gravity (ft./s^2 (32.2 ft./s^2))

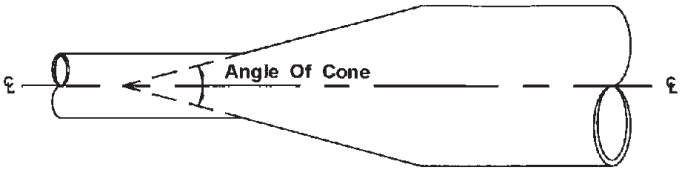


FIGURE 3.8 ANGLE OF CONE FOR CHANGE IN PIPE DIAMETER (SOURCE FHWA HEC 22)

TABLE 3.18 (SOURCE FHWA HEC 22)

TYPICAL VALUES FOR K_E FOR GRADUAL ENLARGEMENT OF PIPES IN NON-PRESSURE FLOW

	ANGLE OF CONE						
D2/D1	10°	20°	45°	60°	90°	120°	180°
1.5	0.17	0.40	1.06	1.21	1.14	1.07	1.00
3	0.17	0.40	0.86	1.02	1.06	1.04	1.00

TABLE 3.19 (SOURCE ASCE MANUALS AND REPORTS OF ENGINEERING PRACTICE NO. 77)

TYPICAL VALUES OF K_c FOR SUDDEN PIPE CONTRACTIONS

D2/D1	K_c
0.2	0.5
0.4	0.4
0.6	0.3
0.8	0.1
1.0	0.0

D2/D1 = Ratio of diameter of smaller pipe to larger pipe.

Table 3.20 (Source ASCE Manuals and Reports of Engineering Practice No. 77)

Values of K_E for Determining Loss of Head due to Sudden Enlargement in Pipes

	Velocity, V1, in feet per second												
D2 /D1	2.0	3.0	4.0	5.0	6.0	7.0	8.0	10.0	12.0	15.0	20.0	30.0	40.0
1.2	0.11	0.10	0.10	0.10	0.10	0.10	0.10	0.09	0.09	0.09	0.09	0.09	0.08
1.4	0.26	0.26	0.25	0.24	0.24	0.24	0.24	0.23	0.23	0.22	0.22	0.21	0.20
1.6	0.40	0.39	0.38	0.37	0.37	0.36	0.36	0.35	0.35	0.34	0.33	0.32	0.32
1.8	0.51	0.49	0.48	0.47	0.47	0.46	0.46	0.45	0.44	0.43	0.42	0.41	0.40
2.0	0.60	0.58	0.56	0.55	0.55	0.54	0.53	0.52	0.52	0.51	0.50	0.48	0.47
2.5	0.74	0.72	0.70	0.69	0.68	0.67	0.66	0.65	0.64	0.63	0.62	0.60	0.58
3.0	0.83	0.80	0.78	0.77	0.76	0.75	0.74	0.73	0.72	0.70	0.69	0.67	0.65
4.0	0.92	0.89	0.87	0.85	0.84	0.83	0.82	0.80	0.79	0.78	0.76	0.74	0.72
5.0	0.96	0.93	0.91	0.89	0.88	0.87	0.86	0.84	0.83	0.82	0.80	0.77	0.75
10.0	1.00	0.99	0.96	0.95	0.93	0.92	0.91	0.89	0.88	0.86	0.84	0.82	0.80
∞	1.00	1.00	0.98	0.96	0.95	0.94	0.93	0.91	0.90	0.88	0.86	0.83	0.81

D2/D1 = ratio of diameter of larger pipe to smaller pipe
V1 = Velocity in smaller pipe (upstream of transition)

Table 3.21 (Source ASCE Manuals and Reports of Engineering Practice No. 77)

Values of K_E for Determining Loss of Head due to Gradual Enlargement in Pipes.

	Angle of Cone										
D2/D1	2°	6°	10°	15°	20°	25°	30°	35°	40°	50°	60°
1.1	0.01	0.01	0.03	0.05	0.10	0.13	0.16	0.18	0.19	0.21	0.23
1.2	0.02	0.02	0.04	0.09	0.16	0.21	0.25	0.29	0.31	0.35	0.37
1.4	0.02	0.03	0.06	0.12	0.23	0.30	0.36	0.41	0.44	0.50	0.53
1.6	0.03	0.04	0.07	0.14	0.26	0.35	0.42	0.47	0.51	0.57	0.61
1.8	0.03	0.04	0.07	0.15	0.28	0.37	0.44	0.50	0.54	0.61	0.65
2.0	0.03	0.04	0.07	0.16	0.29	0.38	0.46	0.52	0.56	0.63	0.68
2.5	0.03	0.04	0.08	0.16	0.30	0.39	0.48	0.54	0.58	0.65	0.70
3.0	0.03	0.04	0.08	0.16	0.31	0.40	0.48	0.55	0.59	0.66	0.71
∞	0.03	0.05	0.08	0.16	0.31	0.40	0.49	0.46	0.60	0.67	0.72

D2/D1 = ratio of diameter of larger pipe to smaller pipe
Angle of cone is the angle in degrees between the sides of the tapering section

TABLE 3.22 (SOURCE ASCE MANUALS AND REPORTS OF ENGINEERING PRACTICE NO. 77)

VALUES OF K_E FOR DETERMINING LOSS OF HEAD DUE TO SUDDEN CONTRACTION.

	VELOCITY, V1, IN FEET PER SECOND												
D2/D1	2.0	3.0	4.0	5.0	6.0	7.0	8.0	10.0	12.0	15.0	20.0	30.0	40.0
1.1	0.03	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.05	0.05	0.60
1.2	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.08	0.08	0.08	0.09	0.10	0.11
1.4	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.18	0.18	0.18	0.18	0.19	0.20
1.6	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.25	0.25	0.24
1.8	0.34	0.34	0.34	0.34	0.34	0.34	0.33	0.33	0.32	0.32	0.32	0.29	0.27
2.0	0.38	0.38	0.37	0.37	0.37	0.37	0.36	0.36	0.35	0.34	0.33	0.31	0.29
2.2	0.40	0.40	0.40	0.39	0.39	0.39	0.39	0.38	0.37	0.37	0.35	0.33	0.30
2.5	0.42	0.42	0.42	0.41	0.41	0.41	0.40	0.40	0.39	0.38	0.37	0.34	0.31
3.0	0.44	0.44	0.44	0.43	0.43	0.43	0.42	0.42	0.41	0.40	0.39	0.36	0.33
4.0	0.47	0.46	0.46	0.46	0.45	0.45	0.45	0.44	0.43	0.42	0.41	0.37	0.34
5.0	0.48	0.48	0.47	0.47	0.47	0.46	0.46	0.45	0.45	0.44	0.42	0.38	0.35
10.0	0.49	0.48	0.48	0.48	0.48	0.47	0.47	0.46	0.46	0.45	0.43	0.40	0.36
∞	0.49	0.49	0.48	0.48	0.48	0.47	0.47	0.47	0.46	0.45	0.44	0.41	0.38

D2/D1 = ratio of diameter of larger pipe to smaller pipe
V1 = Velocity in smaller pipe (downstream of transition)

3.3.4.2.6.5: Junction Losses

For junction losses the design engineer can use the losses found in the University of Missouri Engineering Bulletin No. 41 "Pressure Changes at Storm Drain Junctions." The bulletin was a result of flume model testing.

The conduit junction losses within this section is for the connection of a lateral pipe to a larger storm drain trunk line without an access manhole. The following formula is a form of the momentum equation.

$$H_j = \{[(Q_o V_o) - (Q_i V_i) - Q1V1\cos\theta_j)]/[0.5g(A_o + A_i)]\} + h_i - h_o$$

[Equation 3.47]

H_j = Junction loss (ft.)

Q_o, Q_i, Q1 = Outlet, inlet, and lateral flows (cfs)

V_o, V_i, V1 = Outlet, inlet, and lateral velocity

h_o, h_i = Outlet and inlet velocity head (ft.)

A_o, A_i = Outlet and inlet cross sectional area ($ft.^2$)

θj = Angle between the inflow trunk line and lateral pipe

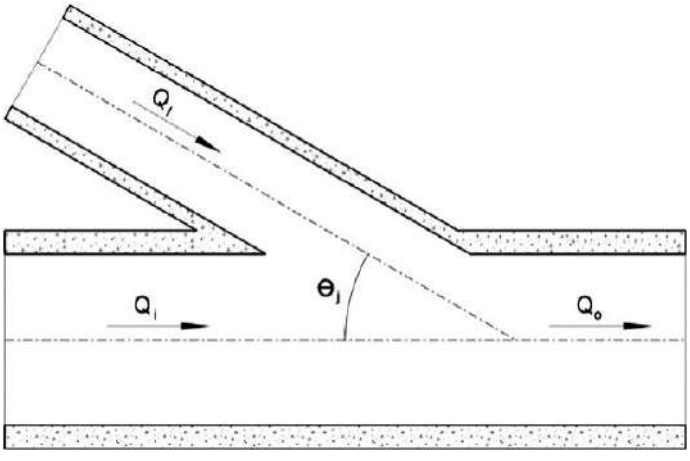


FIGURE 3.9 INTERIOR ANGLE FOR PIPE JUNCTION WITHOUT MANHOLE (SOURCE FHWA HEC 22)

3.3.4.2.6.6: Inlet and Manhole Losses

The conduit junction losses within this section are for the connection of a lateral pipe(s) to a larger storm drain trunk line with an access manhole or junction box.

There are a number of ways to determine the losses within a junction. It is up to the design engineer to determine the appropriate loss method.

Missouri Charts

The instructions and charts from the University of Missouri Engineering Bulletin No. 41 "Pressure Changes at Storm Drain Junctions" is provided in the Appendix B of this manual as a reference for the design engineer should there be special configuration of manholes and junction boxes within the storm drain system. The University of Missouri Engineering Bulletin No. 41 "Pressure Changes at Storm Drain Junctions" was the results of flume model testing.

FHWA Inlet and Access Hole Energy Loss

FHWA has been developing and refining the methods to determine the energy losses within access manholes (junction box) and inlets. The effort has been supported by research and laboratory analysis to improve the methods. These methods calculate the energy level through the manhole.

The FHWA method follows the following three steps. For more information on this method see reference FHWA HEC-22 Urban Drainage Design Manual.

STEP 1: Determine an initial access hole energy level (E_{ai}) based on inlet control (weir and orifice) or outlet control (partial and full flow) equations.

STEP 2: Adjust the initial access hole energy level based on benching, inflow angle(s), and plunging flows to compute the final calculated energy level (E_a).

STEP 3: Calculate the exit loss from each inflow pipe and estimate the energy grade line (EGL_o), which will then be used to continue calculations upstream.

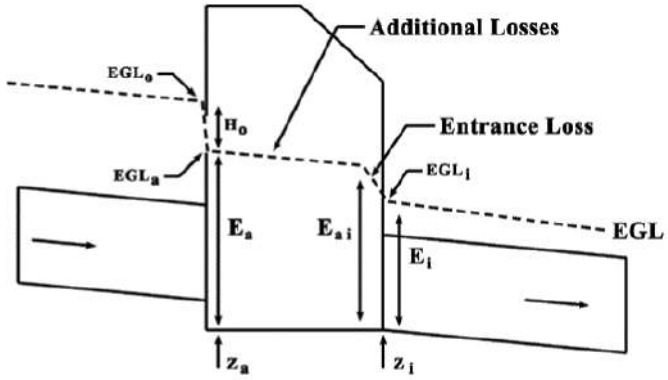


FIGURE 3.10 SKETCH OF FHWA ACCESS HOLE METHOD (SOURCE FHWA HEC-22, 3RD EDITION)

Energy Loss Method (FHWA HEC-22, 2nd Edition)

A number of modeling programs use this method to calculate the energy loss for a junction.

When the inflow conduit invert is above the water surface elevation in the manhole, then the energy loss method does not apply to this upstream conduit.

$$H_{ah} = K(V_o^2/2g)$$
$$K = K_o CD\ C_d CQ\ C_p CB \qquad \text{[Equation 3.48]}$$

H_{ah} = Energy loss head

K = adjusted loss coefficient

K_o = initial head loss coefficient based on relative access hole size

CD = correction factor for pipe diameter (pressure Flow only)

C_d = correction factor for Flow depth

CQ = correction factor for relative Flow

C_p = correction factor for plunging Flow

CB = correction factor for benching

V_o = velocity of outlet pipe

K_o: The initial head loss coefficient is based on the relative access hole size and the angle of deflection between the inflow and outflow conduits.

[Equation 3.49]

$K_o = .1(b/D_o)(1-\sin\theta) + 1.4(b/D_o)^{1.5}\sin\theta$

K_o = initial head loss coefficient based on relative access hole size

θ = angle between the inflow and outflow pipes (**Figure 3.11**)

b = access hole or junction diameter

D_o = outlet pipe diameter

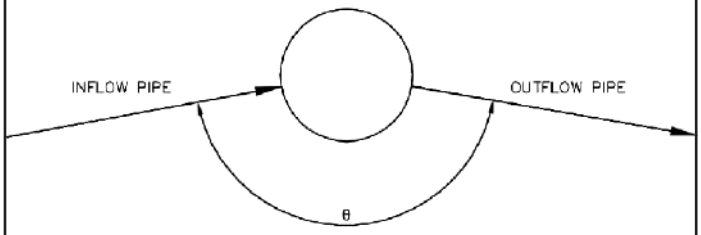


FIGURE 3.11 DEFLECTION ANGLE

CD: When the depth of flow within the manhole relative to the diameter of the outlet conduit ratio, $_{daho/Do,}$ is greater than 3.2, the correction factor for the conduit diameter is as follows.

$$CD = (D_o/D_i)^3 \quad \text{[Equation 3.50]}$$

CD = correction factor for pipe diameter (pressure Flow only)

D_o = outgoing pipe diameter

D_i = inflowing pipe diameter

Cd: The correction factor for flow depth is used when the depth of flow within the manhole relative to the diameter of the outlet conduit ratio, $_{daho/Do,}$ is less than 3.2. If the ratio is greater than 3.2, then the value of C_d is set to 1. The correction factor for flow depth is as follows.

$$C_d = .5(daho/D_o)^{.6} \quad \text{[Equation 3.51]}$$

C_d = correction factor for Flow depth

daho = water depth in access hole above the outlet pipe invert

D_o = outlet pipe diameter

CQ: The correction factor for relative flow is a function of the angle of the incoming flow and the percentage of flow coming in through the conduit of interest. The correction factor will be different for each upstream conduit. The correction factor for relative flow is applicable where the conduits are approximately the same elevation, otherwise the value of CQ is equal to 1.

$$CQ = (1 - 2\sin\theta)(1-Q_i/Q^o)^{.75} + 1 \quad \text{[Equation 3.52]}$$

CQ = correction factor for relative Flow

θ = the angle between the inflow and outflow pipes (**Figure 3.12**)

Q_i = Flow in the inflow pipe

Q_a = Flow in the outflow pipe

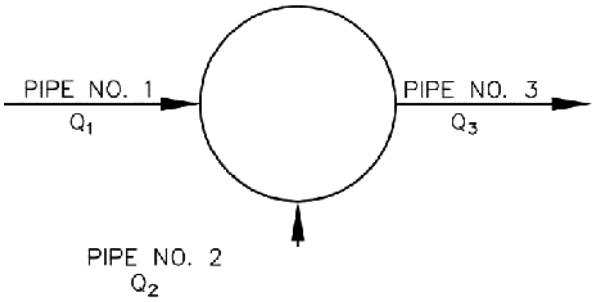


FIGURE 3.12 RELATIVE FLOW EFFECT

C_p: The correction factor for plunging flow is where the inflow one of the conduits plunges into the water surface within the manhole. This correction factor is applied to the inflow conduit and the outflow are at the bottom of the manhole and when $_{h>daho.}$ Flows from a grate inlet or curb opening are considered plunging flow. If there are no plunging flow within the manhole, the correction factor is set to 1.

$$Cp = 1 + 0.2(h/D_o)((h - daho)/D_o) \quad \text{[Equation 3.53]}$$

C_p = correction for plunging flow

h = vertical distance of plunging Flow from the Flow line of the higher elevation inlet pipe to the center of the outflow pipe

D_o = outlet pipe diameter

daho = water depth in access hole relative to the outlet pipe invert

CB: The correction factor for benching in the manhole is obtained from Table 3.23. The benching directs flow through the manhole. See Figure 3.13.

TABLE 3.23 CORRECTION FOR BENCHING

	CORRECTION FACTORS, CB	
BENCH TYPE	SUBMERGED*	UNSUBMERGED**
Flat or Depressed Floor	1.00	1.00
Half Bench	0.95	0.15
Full Bench	0.75	0.07

*pressure Flow, daho/Do ≥ 3.2
**free surface Flow, daho/Do ≤ 1.0

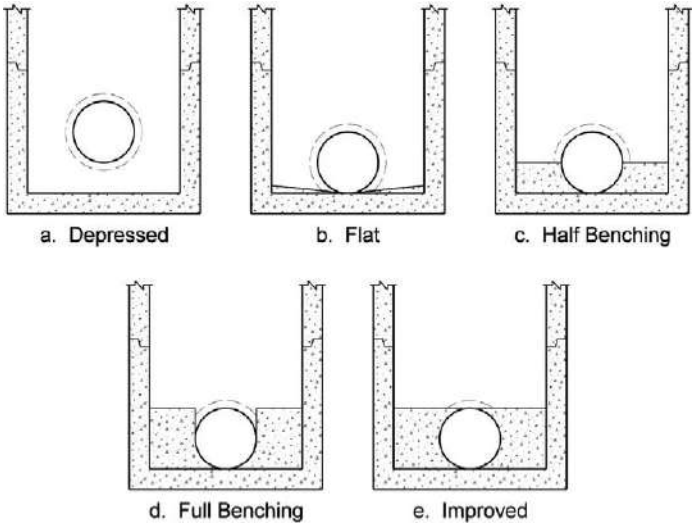


FIGURE 3.13 MANHOLE BENCHING METHODS (SOURCE FHWA HEC-22, 3RD EDITION)

3.3.4.3: DESIGN GUIDELINES

3.3.4.3.1: Time of Concentration and Discharge

The rational formula should be used to determine the discharges through the storm drain system. The time of concentration is the time required for water to travel from the most hydraulically distant point in the watershed to the point under consideration. The time of concentration is comprised of overland flow, sheet flow, and gutter flow to the first inlet. Then from the first inlet there is additional time in the underground drainage system to the design points which should be considered. With the total time of concentration to the point of interest in the storm drain system the designer can select the intensity value for use in the rational formula.

3.3.4.3.2: Velocity and Grade Considerations

The minimum cleaning velocity for a storm drain line is three (3) fps for a 5-year event is recommended. Use the following formula to determine the minimum slope required for a velocity of three (3) fps.

$$S = K_u[(nV)/D^{0.67}]^2 \quad \text{[Equation 3.54]}$$

S = pipe slope (ft./ft.)

K_u = 2.87

n = Manning's N

V = Cleaning velocity (ft./sec.)

D = diameter of conduit (ft.)

The maximum velocity for a storm drain line in a twenty-five (25)-year event should Table 3.24. Velocities above the maximum shall be approved by the Director of Engineering.

TABLE 3.24 MAXIMUM VELOCITY

TYPE	MAXIMUM PERMISSIBLE VELOCITY
Laterals	No limit
Main trunk lines	15 fps

The minimum slope for the storm drain line is 0.3% or as approved by the Director of Engineering.

3.3.4.3.3: Pipe/Box Size and Placement

No storm drains shall be less than eighteen (18) inches in diameter. For laterals and at driveway and roadway crossings, the minimum pipe/culvert size shall be eighteen (18) inches in diameter unless the option for multiple smaller sized culverts is approved by the Director of Engineering. A minimum (15) inches in diameter culvert pipe may be used for driveways accessing a county road section when required within the city limits or by the County Engineer in the County or ETJ.

If a storm drain accepts storm water from offsite area, the storm drain should not be placed below/underneath buildings or structures unless approved by the Director of Engineering. If a proposed building or structure is over an existing storm drain, the storm drain should be relocated around the exterior of the building or structure.

Minimum cover over pipes and boxes should be maintained to make sure the structural stability of the conduit under live and impact loads. Pipes shall have a minimum cover per the manufactures design requirements.

3.3.4.3.4: Multiple Conduits Spacing and Placement

Should multiple parallel precast box culverts be used for a storm drain line, the spacing between adjacent boxes shall be six (6) inches. An increase in this dimension will require additional consideration of the fill material between the boxes.

Should multiple conduits, other than multiple parallel precast box culverts mentioned above, be used, the minimum spacing between conduits should be one (1) foot to allow for the compaction of backfill around the conduits.

Flow equalizers may be needed between multiple conduits to equalize the HGL where laterals or other conduits tie into the drainage system. The equalizer opening should have the same area as the incoming conduit.

3.3.4.3.5: Access Spacing

The maximum distance between access points in a storm drain line shall be five hundred (500) feet.

3.3.4.3.6: Manholes

Manholes to be used on box culvert storm drain lines shall have a manhole ring with a locking or bolted cover and with an inside diameter of thirty (30) inches for maintenance access.

3.3.4.3.7: Junction Boxes

Junction boxes shall be constructed at locations of laterals, changes in grade or alignment of pipes. The riser should have a manhole ring with a locking or bolted cover and with an inside diameter of thirty (30) inches for maintenance access. At the spring line of the pipe, the inside wall of the junction box must be a minimum of the outside diameter of the pipe, plus six (6) inches on each side of the pipe. If the pipe is at a skew to the junction box wall, additional distance is required. When an upstream conduit is smaller than the downstream conduit, it is preferable to match conduit soffits, unless the upstream conduit needs to miss a conflict such as a utility or minimum conduit cover.

3.3.4.3.8: Materials and Specifications

3.3.4.3.8.1: Pipe Material

The pipe material must have a minimum service life of fifty (50) years.

Reinforced concrete pipe is preferred. The use of HDPE or PVC, or corrugated metal pipe will not be allowed crossing under public streets or within public street ROW unless approved by the Director of Engineering. If use is allowed by the Director of Engineering, corrugated metal pipe must be checked for corrosion resistance. Asphalt lining or bituminous interior coated corrugated metal pipe will not be allowed.

3.3.4.3.8.2: Minimum Structural Loads

The minimum live load should be HS 20 for streets and E 80 for railways. Heavier live load may be needed in special cases, and the design engineer should determine the required live load.

3.3.4.3.8.3: Mud Slab

A mud slab is a base slab of low strength concrete used to level up or stabilize the bottom of an excavation for the placement of multiple inlets, multiple boxes, or other structures. The mud slab is from two (2) to six (6) inches thick but may be thicker if needed.

3.3.4.3.9: Outfalls

The outfall of a storm drain system should be to an existing low or proposed channel. The discharge velocity from the outfall should not cause erosion to the existing low or proposed channel. Velocity controls should be used when erosion is possible of the existing low or proposed channel. The outfall of the storm drain should be positioned in the existing low or proposed channel in the downstream direction to reduce the turbulence and erosion. The design

engineer should meet with the Engineering Department to discuss a solution, if a defined low does not exist for the discharge of the outfall. Should the discharge from an outfall cross a sidewalk area, discharge will not be allowed over the sidewalk. A channel section will be provided under the sidewalk.

3.3.4.3.9.1: Velocity Controls

Energy dissipation at the outlet may be required to prevent erosion of the channel bottom and banks. The use of baffle blocks, USBR Type VI impact basin, Contra Costa Basin, rock riprap basin, and rock riprap aprons may be used to reduce the velocity of the discharge from the storm drain conduit. See Chapter 10.4.3 for use of different energy dissipators.

The velocity at the end of the outlet structure should be a maximum of six (6) ft/sec. Sandy soils may require a discharge velocity less than six (6) ft/sec. The design engineer should be aware of the types of soils at the outfall location and design accordingly.

3.3.4.3.10: French Drains

French drains are used to control ground water or surface water. The French drain consists of a perforated pipe with a fabric sock around the exterior of the pipe to keep soil particles from entering the pipe. The pipe is installed in a trench filled with gravel.

A project may encounter a perched water table that will impact the street design section. A French drain may be needed to intercept the ground water that will impact the street section. The French drain should be placed in the parkway between the curb and property line, and should outfall into a drainage inlet, pipe, or channel. The outfall should not drain onto the street, as this could cause street failure or cause an unsafe condition.

3.3.4.4: MAINTENANCE CONSIDERATIONS

A few items to consider during the design of a storm drain system would be the minimum cleaning velocity to keep sediment in suspension during a storm, access points along the trunk line for ease of maintenance personnel to clean and inspect the system, and access to outfall or intake structures for cleaning and inspection. See Chapter 4.12 for additional guidance on maintenance standards.

3.3.5: OPEN CHANNELS

3.3.5.1: General

Open channels for use in a major drainage system have significant advantages related to cost, capacity, multiple use for recreational and aesthetic purposes and potential for detention storage. Disadvantages include right of way needs and maintenance costs. Careful planning and design are needed to minimize the disadvantages and to increase the benefits.

The general classifications for open channels are: (1) Natural channels, which include all watercourses that have been carved by nature through erosion; and (2) New or altered channels, which are constructed or existing channels that have been significantly altered by human effort. New or altered channels can be lined with grass, concrete, mortared rocks or other materials. The channels should be designed for the 25-year storm with provisions for the 100-year storm within dedicated easements or right of way.

3.3.5.1.1: Natural Channels

The ideal natural channel has the following benefits:

a. Velocities are usually low, resulting in longer concentration times and lower downstream peak flows.

b. Maintenance needs are usually low because the channel is somewhat stabilized.

c. The channel provides a desirable green belt and recreational area add in significant social benefits.

3.3.5.1.2: New or Altered Channels

Grass channels are the most desirable of the various types of new channels for the following reasons:

a. The grass can stabilize the body of the channel.

b. The grass consolidates the soil mass of the bed.

c. The grass controls the movement of soil particles along the channel bottom.

Concrete lined channels are designed to protect the channel body from the erosive potential of high velocities. In addition to concrete-lined channels, other methods to combat erosive velocities in channels may be available and should be submitted to the Director of Engineering and Capital Improvements for review.

3.3.5.1.3: Section 404 Permit

When a project to modify a natural channel is proposed, the design engineer should check the requirements of Section 404 of the Clean Water Act. If required, a permit should be obtained from the U.S. Army Corps of Engineers by the design engineer.

3.3.5.2: Open Channel Hydraulics

An open channel is a conduit in which water flows with a free surface. The classification of open channel flow is made according to the change in flow depth with respect to time and space.

Flow in an open channel is said to be "steady" if the depth of flow does not change or if it can be assumed to be constant during the time interval under consideration. The flow is "unsteady" if the depth changes with time.

Open channel flow is said to be "uniform" if the depth of flow is the same at every section of the channel. A uniform flow may theoretically be steady or unsteady, depending on whether or not the depth changes with time. The establishment of unsteady uniform flow requires that the water surface fluctuate with time while remaining parallel to the channel bottom. Since it is impossible for this condition to occur within a channel, steady uniform flow is the fundamental type of flow treated in open channel hydraulics.

Flow is "varied" if the depth of flow changes along the length of the channel. Varied flow may be either steady or unsteady. Since unsteady uniform flow is rare, the term "unsteady flow" is used to designate unsteady varied flow exclusively.

Varied flow may be further classified as either "rapidly" or "gradually" varied. The flow is rapidly varied if the depth changes abruptly over a comparatively short distance; otherwise, it is gradually varied. Rapidly varied flow is also known as a local phenomenon; an example of which is the hydraulic jump.

With these varying conditions, open channel hydraulics can be very complex, encompassing many different flow conditions from steady uniform flow to unsteady rapidly varied flow. Most of the problems in stormwater drainage involve uniform, gradually varied or rapidly varied flow situations. In this section, the basic equation and computational procedures for uniform, gradually varied and rapidly varied flows are presented.

3.3.5.2.1: Uniform Flow

For a given channel condition of roughness, discharge and slope, there is only one (1) possible depth for maintaining a uniform flow. This depth is referred to as normal depth.

The Manning's Equation is used to determine the normal depth for a given discharge.

$$Q = (1.49/n)\, A\, R^{2/3}\, S^{1/2} \qquad \text{[Equation 3.55]}$$

Where,

Q = Total discharge, cfs

n = Roughness coefficient

A = Cross-sectional area of channel, ft^2

R = Hydraulic radius of channel, feet ($R=A/P$)

S = Slope of the frictional gradient, ft/ft

P = Wetted perimeter, feet

Uniform flow is more often a theoretical abstraction than an actuality. True uniform flow is difficult to find in nature or to obtain in the laboratory. The engineer must be aware of the fact that uniform flow computations provide only an approximation of what will occur but that such computations are usually adequate and useful and, therefore, necessary for planning.

The computation of normal depth for trapezoidal sections can be performed by using Figure D.21 in Appendix D of this manual.

3.3.5.2.2: Gradually Varied Flow

The most common example of gradually varied flow in urban drainage systems occurs in the backwater of bridge openings, culverts, storm sewer inlets and channel constrictions. Under these conditions, gradually varied flow will be created and the flow depth will be greater than normal depth in the channel. Backwater techniques would need to be applied to determine the water surface profile.

Calculations of water surface profiles can be accomplished by using standard backwater methods or acceptable computer routines, which take into consideration all losses due to changes in velocity, drops, bridge openings and other obstructions in open channels.

There are several acceptable methods for backwater calculations. The most common hand calculation method for prismatic channels and irregular-uniform channels is the Standard Step Method. The

most widely used backwater analysis computer program is HEC-RAS, developed by the U.S. Army Corps of Engineers. This program can compute water surface profiles for natural and new channels.

3.3.5.2.3: Rapidly Varied Flow

Rapidly varied flow is characterized by abrupt changes in the water surface elevation for a constant flow. The change in elevation may become so abrupt that the flow profile is virtually broken, resulting in a state of high turbulence. Some common causes of rapidly varied flow in urban drainage systems are side-spill weirs, weirs and spillways of detention basins.

3.3.5.3: Manning's Roughness Coefficients

3.3.5.3.1: Existing and Natural Channels

Because several primary factors affect the roughness coefficient, a procedure has been developed to estimate this value, n. By this procedure, the value of n may be computed by:

$$n = (n_0 + n_1 + n_2 + n_3 + n_4)m \qquad \text{[Equation 3.56]}$$

Where n_0 is a basic n value for a straight, uniform, smooth channel in the natural materials involved, n_1 is a value added to n_0 to correct for the effect of surface irregularities; n_2 is a value for variations in shape and size of the channel cross section; n_3 is a value for obstructions; n_4 is a value for vegetation and flow conditions; and m is a correction factor for meandering of the channel. Proper values of n_0 to n_4 and m may be selected from Table 3.25 according to the given conditions.

In selecting the value of n_1, the degree of irregularity is considered smooth for surfaces comparable to the best attainable for the materials involved; minor for good dredged channels, slightly eroded or scoured side slopes of canals or drainage channels; moderate for fair to poor dredged channels, moderately sloughed or eroded side slopes of canals or drainage channels; and severe for badly sloughed banks of natural streams, badly eroded or sloughed sides of canals or drainage channels, and unshaped, jagged and irregular surfaces of channels excavated in rock.

In selecting the value of n_2, the character of variations in size and shape of cross section is considered gradual when the change in size or shape occurs gradually; alternating occasionally when large and small sections alternate occasionally or when shape changes cause occasional shifting of main flow from side to side; and alternating frequently when large and small sections alternate frequently or when shape changes cause frequent shifting of main flow from side to side.

The selection of the value of n_3 is based on the presence and characteristics of obstructions such as debris deposits, stumps, exposed roots, boulders and fallen and lodged logs. One should recall that conditions considered in other steps must not be re-evaluated or double-counted in this selection. In judging the relative effect of obstructions, consider the following: the extent to which the obstructions occupy or reduce the average water area, the obstruction characteristics (sharp-edged or angular objects induce greater turbulence than curved, smooth-surfaced objects) and the position and spacing of obstructions transversely and longitudinally in the reach under consideration.

Table 3.25 Computation of Composite Roughness Coefficient for Excavated and Natural Channels

$N = (N_0 + N_1 + N_2 + N_3 + N_4)M$

Channel Conditions		Values
n_0 Material Involved	Earth	0.020
	Rock Cut	0.025
	Fine Gravel	0.024
	Coarse Gravel	0.028
n_1 Degree of Irregularity	Smooth	0.000
	Minor	0.005
	Moderate	0.010
	Severe	0.020
n_2 Relative Effect of Channel Cross Section	Gradual	0.000
	Alternating	0.005
	Occasionally	0.013
	Alternating Frequently	
n_3 Relative Effect of Obstructions	Negligible	0.000
	Minor	0.013
	Appreciable	0.025
	Severe	0.050
n_4 Vegetation	Low	0.008
	Medium	0.018
	High	0.038
	Very High	0.075
m Degree of Meandering	Minor	1.000
	Appreciable	1.150
	Severe	1.300

Source: Chow, V.T. *Open Channel Hydraulics*, 1959.

In selecting the value of n4, the degree of effect of vegetation is considered in the following way:

a. **Low** for conditions comparable to the following: (a) dense growths of flexible turf grasses or weeds, of which Bermuda and blue grasses are examples, where the average depth of flow is two (2) to three (3) times the height of vegetation and (b) supple seedling tree switches, such as willow, cottonwood or salt cedar where the average depth of flow is three (3) to four (4) times the height of the vegetation.

b. **Medium** for conditions comparable to the following: (a) turf grasses where the average depth of flow is one (1) to two (2) times the height of vegetation, (b) stemmy grasses, weeds or tree seedlings with moderate cover where the average depth of flow is two (2) to three (3) times the height of vegetation and (K) brush growths, moderately dense, similar to willows one (1) to two (2) years old, dormant season, alongside slopes of a channel with no significant vegetation along the channel bottom, where the hydraulic radius is greater than two (2) feet.

c. **High** for conditions comparable to the following: (a) turf grasses where the average depth of flow is about equal to the height of vegetation, (b) dormant season — willow or cottonwood trees eight (8) to ten (10) years old, intergrown with some weeds and brush, where none of the vegetation is in foliage, where the hydraulic radius is greater than two (2) feet, and (c) growing season — bushy willows about one (1) year old intergrown with some weeds in full foliage alongside slopes, no significant vegetation along channel bottom, where hydraulic radius is greater than two (2) feet.

d. **Very high** for conditions comparable to the following: (a) turf grasses where the average depth of flow is less than ½ the height of vegetation, (b) growing season — bushy willows about one (1) year old, intergrown with weeds in full foliage alongside slopes, or dense growth of cattails along channel bottom, with any value of hydraulic radius up to ten (10) or 15 feet and (c) growing season— trees intergrown with weeds and brush, all in full foliage, with any value of hydraulic radius up to ten (10) or 15 feet.

In selecting the value of m, the degree of meandering depends on the ratio of the meander length to the straight length of the channel reach. The meandering is considered minor for ratios of 1.0 to 1.2, appreciable for ratios of 1.2 to 1.5, and severe for ratios of 1.5 and greater.

In applying the above method for determining the n value, several things should be noted. The method does not consider the effect of suspended and bed loads. The values given in Table 3.25 were developed from a study of some 40 to 50 cases of small and moderate channels. Therefore, the method is questionable when applied to large channels whose hydraulic radii exceed 15 feet. The method applies only to unlined natural streams, floodways, and drainage channels and shows a minimum value of 0.02 for the n value of such channels. The minimum value of n in general, however, may be as low as 0.012 in lined channels and as low as 0.008 in artificial laboratory flumes.

3.3.5.3.2: New or Altered Channels

The Manning's Roughness Coefficients (n) for new or altered channels are shown in Table 3.26.

Table 3.26 Minimum Roughness Coefficients of New or Altered Channels

Type of Channel and Description	Manning's Coefficients
1. **Grass Lined**	
a. Bermuda (with regular mowing)	0.040
b. St. Augustine (with regular mowing)	0.045
c. Native grasses and vegetation not mowed regularly	0.060
2. **Concrete**	
a. Concrete lined (rough finish)	0.020
b. Concrete lined (smooth finish – culverts)	0.015
c. Concrete rip-rap (exposed rubble)	0.025
3. **Gabion**	0.035
4. **Rock-cut**	0.025

Source: 1. Chow, V.T. Open Channel Hydraulics. 1959
2. WRC Engineering, Inc. Boulder County Storm Drainage Criteria Manual. 1984

3.3.5.4: Design Requirements

Channel design involves the determination of the channel cross-section required to accommodate a given design discharge. The design requirements for open channels are discussed in the sections below and apply to channels or waterways that are proposed to be modified or constructed.

3.3.5.4.1: Grass-Lined Channels and Waterways

Key parameters in grass-lined channel or waterway design include permissible velocity, roughness coefficient, side slope, curvature, bottom width, and freeboard. The grass species selected shall conform to the City of San Marcos Standards and Standard Specifications, as appropriate, and must be suitable for permanent application based upon the anticipated operation and maintenance of the channel or waterway.

a. **Velocity.** The maximum permissible velocity for the one hundred (100) year storm is six (6) feet per second and includes all transitions to or from channels and waterways with similar or different materials. In all cases, the velocity for the one hundred (100) year storm must be non-erosive. The minimum permissible velocity for the two (2) year storm is two (2) feet per second.

b. **Roughness Coefficient.** The roughness coefficients selected shall be based on the degree of retardance of vegetation. Table 3.25 provides minimum Manning's Coefficients for channel design. The roughness coefficient shall be adjusted to reflect the relationship between the depth of flow and the typical height of the design vegetation, especially for shallow depths of flow, as well as other factors affecting channel conveyance.

c. **Slope.** The flow line slope of the channel shall be a minimum of two (2) percent unless the velocity for the two (2) year storm flow is greater than two (2) feet per second, in which case the channel slope may be a minimum of one (1) percent. Compliance with this requirement must take into account the variation in channel flow due to distributed inflows to the channel. A reinforced concrete pilot channel must be used if the channel slope is less than one (1) percent. The pilot channel must be at least four (4) feet wide, two (2) inches deep, and be capable of withstanding vehicular loading. Any grass-lined portion of the channel bottom must have a slope of at least two (2) percent from that portion to the concrete-lined pilot channel. However, no open channel flow line slope may be less than one-half (0.5) percent.

d. **Side Slopes.** Side slopes shall be three (3) to one (1) or flatter.

e. **Curvature.** The center line curvature shall have a minimum radius of twice the top width of the one hundred (100) year storm flow.

f. **Bottom Width.** The minimum flat bottom width of the channel is three (3) feet.

g. **Freeboard.** All grass-lined channels shall be designed to convey the one hundred (100) year storm event. The freeboard for the channel shall be the velocity head for the one hundred (100) year storm.

3.3.5.4.2: Concrete-Lined Channels

Concrete-lined channels may be needed in channel reaches where the velocities are excessive (See Section 3.3.5.4.1) or where the channel characteristics require such use.

a. **Velocity.** In concrete-lined channels the probability of achieving supercritical flow is greatly increased. The designer must take care to insure against the possibility of unanticipated hydraulic jumps forming in the channel in considering the 25- and 100-year storms. Flow with a Froude number equal to one (1) is unstable and should be avoided. If supercritical flow does occur, then freeboard and superelevation must be determined. In addition, all channels carrying supercritical flow shall be continuously lined with reinforced concrete.

b. **Roughness Coefficient.** Table 3.25 provides the Manning's Coefficients for concrete-lined channels.

c. **Freeboard.** Adequate channel freeboard shall be provided for the 100-year storm in reaches where supercritical flow occurs by Equation 3.57 or using the energy grade line, whichever is less.

$$H_{FB} = 2.0 + 0.025V\,(d)^{1/3} \quad \text{[Equation 3.57]}$$

Where,

H_{FB} = Freeboard height, ft

V = Velocity, ft/sec

d = Depth of flow, ft

Freeboard shall be in addition to superelevation, standing waves and/or other water surface disturbances. Concrete side-slopes shall be extended to provide freeboard. Freeboard shall not be obtained by the construction of levees.

d. Superelevation. Superelevation of the water surface shall be determined at all horizontal curves which deviate more than 45 degrees off the projected centerline. An approximation of the superelevation at a channel bend can be obtained from the following equation:

$$h = V^2 T_w / g r_c \quad \text{[Equation 3.58]}$$

Where,

h = Superelevation, ft

V = Flow velocity, ft/sec

T_w = Top width of channel, ft

R_c = Centerline radius of curvature, ft

g = Acceleration due to gravity, ft/sec^2

The freeboard shall be measured above the superelevation water surface.

e. Side Slopes. Since concrete lined channels do not require slope maintenance, the side slopes may be as steep as vertical with appropriate structural methods applied.

f. Slope. The flow line slope of the channel shall be no less than one-half (0.5) percent and must also be sufficient to produce a velocity for the two (2) year storm flow of at least two (2) feet per second. Compliance with this requirement must take into account the variation in channel flow due to distributed inflows to the channel.

3.3.5.4.3: Other Channels

Channels composed of materials other than vegetation or concrete shall be designed so that sediment deposition does not occur for the two (2) year storm (except for channel drop structures and energy dissipaters as approved by the City) and velocities for the one hundred (100) year storm are not erosive, using methods as approved by the Director of Engineering and Capital Improvements.

3.3.5.5: Channel Drop Structures

The function of a drop structure is to reduce channel velocities by allowing for flatter upstream and downstream channel slopes. Two commonly used drop structures are shown in Figure D.22 in Appendix D of this manual.

The flow velocities in the upstream and downstream channels of the drop structure need to satisfy the permissible velocities allowed for channels. The design parameters for the sloping channel drop and the vertical channel drop are given below.

3.3.5.5.1: Sloping Channel Drop

a. Approach Apron. A minimum ten (10) foot long riprap apron should be constructed immediately upstream of the drop to protect against the increasing velocities and turbulence which result as the water approaches the sloping portion of the drop structure. The same riprap and bedding design should be used as specified for the portion of the drop structure immediately downstream of the drop.

b. Chute. The chute shall have roughened faces and shall be no steeper than 2:1. The length, L, of the chute depends upon the hydraulic characteristics of the channel and drop. For a unit discharge, q, of 30 cubic feet per second per foot, L would be about 15 feet, that is, about ½ of the q value. The L should not be less than ten (10) feet, even for low q values.

c. Downstream Apron. The length of the downstream apron shall be sized according to Table 3.27 and shall be constructed of reinforced concrete or riprap depending on structural requirements.

TABLE 3.27 LENGTH OF DOWNSTREAM APRON

MAXIMUM UNIT DISCHARGE, Q (CFS/FT)	LENGTH OF DOWNSTREAM APRON, L_B (FT)
0-14	10
15	15
20	20
25	20
30	25

Source: City of Austin, Watershed Engineering Division

3.3.5.5.2: Vertical Channel Drops

The design criteria for the vertical channel drop is based upon the height of the drop and the normal depth and velocity of the approach and exit channels. The channel must be prismatic throughout, from the upstream channel through the drop to the downstream channel.

The steepest allowable sideslope for the riprap stilling basin is 4:1. The riprap should extend up the side slopes to a depth equal to one (1) foot above the normal depth projected upstream from the downstream channel. The maximum fall allowed at any one drop structure is four (4) feet from the upper channel bottom to the lower channel bottom.

A description of the drop structure and the design procedure, going from upstream to downstream, is given below and shown on Figure D.22 in Appendix D of this manual.

a. **Approach Channel:** The upstream and downstream channels will normally be grass-lined trapezoidal channels.

b. **Approach Apron:** A minimum ten (10) foot long riprap apron is provided upstream of the drop to protect against the increasing velocities and turbulence which result as the water approaches the vertical drop.

c. **Downstream Apron:** The riprap stilling basin is designed to force the hydraulic jump to occur within the basin and is designed for essentially zero scour.

3.3.5.6: Energy Dissipaters

Energy dissipaters are used to dissipate excessive kinetic energy in flowing water that could promote erosion. An effective energy dissipater must be able to retard the flow of fast moving water without damage to the structure or to the channel below the structure.

Impact-type energy dissipaters direct the water into an obstruction that diverts the flow in many directions and in this manner, dissipates the energy in the flow. Baffled outlets and baffled aprons are two (2) impact-type energy dissipaters.

Other energy dissipaters use the hydraulic jump to dissipate the excess head. In this type of structure, water flowing at a higher than critical velocity is forced into a hydraulic jump, and energy is dissipated in the resulting turbulence. Stilling basins are this type of dissipater, where energy is diffused as flow plunges into a pool of water.

Generally, the impact-type of energy dissipater is considered to be more efficient than the hydraulic jump-type. Also, the impact-type energy dissipater results in smaller and more economical structures.

The design of energy dissipaters is based on the empirical data resulting from a comprehensive series of model structure studies by the U.S. Bureau of Reclamation, as detailed in its book Hydraulic Design of Stilling Basins and Energy Dissipaters. Two (2) impact-type energy dissipaters are briefly explained here.

3.3.5.6.1: Baffled Apron (U.S. Bureau of Reclamation Type IX)

Baffled aprons are used to dissipate the energy in the flow at a drop. They require no initial tailwater to be effective, although channel bed scour is not as deep and is less extensive when the tailwater forms a pool into which the flow discharges. The chutes are constructed on a slope that is 2:1 or flatter and extends below the channel bottom. Backfill is placed over one (1) or more bottom rows of baffles to restore the original streambed elevation. When scour or downstream channel degradation occurs, successive rows of baffle piers are exposed to prevent excessive acceleration of the flow entering the channel. If degradation does not occur, the scour creates a stilling pool at the downstream end of the chute, stabilizing the scour pattern. The simplified hydraulic design of the baffled apron is shown in Figure D.23 in Appendix D of this manual.

The general rules of hydraulic design of a baffled apron are as follows:

a. **Design Discharge.** The chute should be designed for the full capacity expected to be passed through the structure. The maximum unit discharge may be as high as 60 cfs per foot for the 100-year storm.

b. **Chute Entrance.** The flow entering into the chute should be well distributed laterally across the width of the chute. The velocity should be well below the critical velocity, preferably the value shown in the curve D of Figure D.23 in Appendix D of this manual. The curve C in Figure D.23 in Appendix D of the manual is the critical velocity in a rectangular channel, Vc=(gq)1/3.

c. **Chute Design.** The chute is usually constructed on a 2:1 slope. The upstream end of the chute floor should be joined to the horizontal floor by a curve to prevent excessive vertical contraction of the flow. The upstream face of the first row should be no more than one (1) foot (vertically) below the high point of the chute.

   I. Based on the results of U.S. Bureau of Reclamation experiments, the greatest tendency to overtop the training walls occurs in the vicinity of the second and third rows of baffles. To prevent this overtopping, a partial baffle (1/3 to 2/3 of the width of a full baffle) should be placed against the training walls in the first row. This will place a space of the same width adjacent to the walls in the second row. Alternate rows are then made identical (i.e., rows 1, 3, 5, 7, etc., are identical; Rows 2, 4, 6, 8, etc., are identical). Four (4) rows of baffles are necessary to establish the expected flow pattern at the base of the chute.

   II. The height of the training walls on the chute should be three (3) or more times the baffle height, measured normal to the chute floor. Several rows of baffle piers are usually constructed below the channel grade to establish full control of the flow. At least one (1) row of baffles should be buried in the backfill which is used to restore the original bottom topography.

d. **Heights and Spacing of Baffle Pier.** Baffle pier height, H, should be about 0.8 D_c to 0.9 D_c, as shown in Curve B in Figure D.23 in Appendix D of this manual. D_c is the critical depth in a rectangular channel and determined by:

$D_c = (q^2/g)^{1/3}$ (Eq. 4-1-24)

Baffle pier widths and spaces should be equal, up to 1.5H but no less than H. The slope distance between rows of baffle piers should be 2H, twice the baffle height.

1.1.1.1.2: Baffled Outlet

Baffled outlets are used to dissipate the discharge energy from flow in a pipe. They are normally used at outlets from detention ponds or storm drainage systems. The baffles are intended to decrease the discharge velocities and subsequent erosion of the receiving system. The design detail for a baffled outlet can be found in the City of Austin Standards.

3.3.5.7: Structure Aesthetics

The design of hydraulic structures in the urban environment requires an approach not encountered elsewhere because appearance must be an integral part of the design. The treatment of the exterior appearance should not be considered of minor importance.

Parks. Hydraulic structures should not detract from the pleasures enjoyed in an urban park. Furthermore, parks and green belts may later be developed in an urban area in which the structure will play a dominant environmental role.

Play Areas. An important consideration is that drainage structures often are an attraction for neighborhood children. It is almost impossible to make drainage works inaccessible to children, and therefore what is constructed should be made as safe as is reasonably possible. Hazards to children's safety should be avoided whenever possible.

Concrete Surface Treatment. The use of textured concrete presents a pleasing appearance and removes form marks. Exposed aggregate concrete is also attractive but may require special control of the aggregate used in the concrete.

Rails and Fences. The use of rails and fences along concrete walls provides a pleasing topping to an otherwise stark wall yet provides a safety measure against the hazard of falling from an unprotected wall.

3.3.6: BRIDGES AND CULVERTS

3.3.6.1: General

The function of a drainage culvert is to pass the 100-year design storm flow without causing excessive backwater or overtopping of the structure and without creating excessive downstream velocities. The designer shall keep energy losses and discharge velocities within allowable limits when selecting a structure that will meet these requirements. The design storm flow shall be determined by the hydrologic methods as set forth in Section 3.2.1 of this manual. The system shall accommodate the runoff from 25-year and 100-year frequency storms. The rules for manhole spacing also apply to culverts. The Federal Highway Administration is a good source of information for the design of culverts and bridges, including the HY-8 software, Hydraulic Design Series publications, and Hydraulic Engineering Circulars. The Army Corps of Engineers' HEC-RAS software is also a good tool for analyzing systems that contain culverts or bridges.

3.3.6.2: Culvert Headwalls and Endwalls

3.3.6.2.1: General

The normal functions of properly designed headwalls and endwalls are to anchor the culvert in order to prevent movement due to hydraulic and soil pressures, to control erosion and scour resulting from excessive velocities and turbulence and to prevent adjacent soil from sloughing into the waterway opening. All headwalls shall be constructed of reinforced concrete and may be either straight-parallel, flared or warped. They may or may not require aprons, as determined by site conditions. Headwalls should be aligned with the direction of the receiving flow when discharging into a waterway. Precast headwalls and endwalls may be used if all other criteria are satisfied; generally precast headwalls/endwalls are available for smaller culverts (18 and 24 inches diameter).

3.3.6.2.2: Conditions at Entrance

The operating characteristics of a culvert may be completely changed by the shape or condition at the inlet or entrance. Therefore, design of culverts must involve consideration of energy headlosses that may occur at the entrance. Entrance headlosses may be determined by the following equation:

$$H_e = K_e (V_2^2 - V_1^2)/2g \qquad \text{[Equation 3.59]}$$

Where,

H_e = Entrance headloss, feet

V_2 = Velocity of flow in culvert, ft/sec

V_1 = Velocity of flow approaching culvert, ft/sec

K_e = Entrance loss coefficient as shown in Table 3.28

g = Acceleration due to gravity

TABLE 3.28 VALUES OF CULVERT ENTRANCE LOSS COEFFICIENTS

TYPE OF ENTRANCE	ENTRANCE COEFFICIENT, K_e
Pipe	
Headwall (no wingwalls)	
Grooved edge	0.20
Rounded edge (0.15D radius)	0.15
Rounded edge (0.25D radius)	0.10
Square edge (cut concrete and CMP)	0.40
Headwall with 45° wingwalls	
Grooved edge	0.20
Square edge	0.35
Headwall with parallel wingwalls spaced 1.25D apart	
Grooved edge	0.30
Square edge	0.40
Beveled edge	0.25
Projecting Entrance (no headwall or wingwalls)	
Grooved edge (RCP)	0.25
Square edge (RCP)	0.50
Sharp edge, thin walls (CMP)	0.90
Sloping Entrance (no headwall or wingwalls)	
Mitered to conform to slope	0.70
Flared-end section	0.50
Box, Reinforced Concrete	
Headwall Parallel to Embankment (no wingwalls)	
Square edge on sides of opening	0.50
Rounded on 3 edges to radius of 1/12 barrel dimension	0.20
Wingwalls at 30° to 75° to barrel axis	
Square edged at crown	0.40
Crown edge rounded to radius of 1/12 barrel dimension	0.20
Wingwalls at 10° to 30° to barrel axis	
Square edged at crown	0.50
Wingwalls parallel (extension of culvert walls)	
Square edged at crown	0.70
RCP: Reinforced Concrete Pipe CMP: Corrugated Metal Pipe	

Note: The entrance loss coefficients are used to evaluate the culvert or drain capacity operating under outlet control.
Source: WRC Engineering, Inc. Boulder County Storm Drainage Criteria Manual, 1984.

3.3.6.2.3: Type of Headwall

The common types of headwall entrances are shown in Figure D.21 in Appendix D of this manual but are not limited to the designs shown there. The following guidelines can be used in the selection of the type of headwall. Approach velocities are measured immediately upstream of the headwall under normal operating conditions.

a. Parallel Headwall.

1. Approach velocities are low (below six (6) feet per second).
2. Backwater pools are permitted.

b. Flared Headwall.

3. Approach velocities are between six (6) and ten (10) feet per second.
4. Ample right of way or easement is available.

The wings of flared walls should be located with respect to the direction of the approaching flow, not the culvert axis as in Figure 24 in Appendix D of this manual.

c. Warped Headwall.

5. Approach velocities are between eight (8) and 20 feet per second.

Warped headwalls are effective with aprons to accelerate flow through the culvert.

3.3.6.2.4: Debris Fins

For conditions where more than one (1) box culvert is required, the upstream face of the structure shall incorporate debris deflector fins to prevent debris buildup. For multiple-pipe situations installations of debris fins may be used but are not required.

The debris fin is an extension of the interior walls of a multiple-box culvert. The wall thickness shall be designed to satisfy structural requirements and reduce impact and turbulence to the flow.

A debris fin is constructed to the height of the culvert. A fin length of one and one-half (1.5) times the height of the box culvert is required. Since the debris fins are subject to the same erosive

forces as bridge piers, care must be taken in the design of the footing. A toewall at the upstream end of the debris fin and the apron is recommended. Figure D.26 in Appendix D of this manual depicts the conceptual design for debris deflector fins. It should be noted that alternate types of wingwalls can be used other than the parallel shown in Figure D.26.

3.3.6.2.5: Trash racks and Safety End Treatments

If trash racks or safety end treatments are to be used, appropriate clogging factors must be applied. TxDOT provides some guidance for the selection of clogging factors. "Design of Small Dams" by US Bureau of Reclamation (1987) suggests a trash rack clogging factor of 50%.

3.3.6.3: Culvert Discharge Velocities

Placement of a culvert crossing in a channel produces rapid changes in flow regime that can present erosion hazards both upstream and downstream of the culvert location. Design of the culvert should incorporate features that lessen these impacts to the receiving channel to the greatest extent possible. The following concepts should be considered in the design process. These erosion hazards are discussed in greater detail in the Federal Highway Administration (FHWA), Hydraulic Engineering Circular No. 14, "Hydraulic Design of Energy Dissipaters for Culverts and Channels":

- Whenever possible, the culvert axis should match the natural channel alignment upstream and downstream. Matching the channel alignment with the culvert axis prevents a bend in the channel that would be subject to erosion.
- Depressed entrances should be avoided. If capacity or depth of cover forces the use of a depressed entrance, the upstream apron should be designed to prevent progressive degradation of the upstream channel. Additionally, the potential for deposition should be considered.
- The local conditions within the channel reach should be evaluated to determine if there is degradation present. In a degrading channel, headcuts can migrate upstream and compromise the integrity of the culvert. If channel degradation is anticipated, the design should accommodate for future erosion.
- At a culvert outlet, exit velocities should be minimized to the greatest extent practical. Channel erodibility and local scour potential should be evaluated and taken into account. Due to the dynamic nature of alluvial channels, a flexible armoring system is preferred in streams subject to erosion. Where degradation is expected, the volume of riprap should be increased to account for loss of base level downstream of the culvert outlet. The rock riprap used in this standard detail must be designed by the engineer for the specific hydraulic conditions present. Where energy dissipation is needed at a culvert outlet, a rock riprap basin is preferred over rigid structures (see HEC-14 for design guidelines).
- If multiple boxes or culverts are necessary, different flowline elevations for each structure should be evaluated. The culverts in the center of the channel should be lower to match with the existing natural channel with the outlying culverts raised up to more closely coincide with the natural channel "terrace" elevation. This will help to minimize sedimentation in the outlying culverts, help preserve the integrity of the channel system, and reduce maintenance costs. A culvert may be depressed below the channel flowline, such that a natural channel bottom is maintained, as long as n-values and cross-sectional area are adjusted appropriately when modeling conveyance.

Riprap armoring is preferred; however, if site conditions necessitate a concrete apron, the minimum apron length which provides transition from a culvert outlet to an open channel shall be calculated from the following equation:

$$L = 0.2\ VD \qquad \text{[Equation 3.60]}$$

Where,

L = Apron length, feet

V = culvert discharge velocity, ft/sec

D = height of box culvert or diameter of pipe culvert, feet

3.3.6.4: Selection of Culvert Size and Flow Classification

Laboratory tests and field observations show that there are two (2) major types of culvert flow: (1) flow with inlet control, and (2) flow with outlet control. Under inlet control the cross-sectional area of the barrel, the inlet configuration or geometry and the amount of headwater are the factors affecting capacity. Outlet control involves the additional consideration of the tailwater in the outlet channel and the slope, roughness and length of barrel. Under inlet control conditions, the slope of the culvert is steep enough so that the culvert does not flow full and the tailwater does not affect the flow. If using software to perform culvert calculations, keep in mind the limitations of the modeling program.

3.3.6.4.1: Culvert Hydraulics

3.3.6.4.1.1: Inlet Control Condition.

Inlet control for culverts may occur in two (2) ways:

1. **Unsubmerged:** The headwater is not sufficient to submerge the top of the culvert opening and the culvert inlet slope is supercritical. The culvert inlet acts like a weir (Condition A, Figure D.26 in Appendix D of this manual).

2. **Submerged:** The headwater submerges the top of the culvert but the pipe does not flow full. The culvert inlet acts like an orifice (Condition B, Figure D.26 in Appendix D of this manual).

The discharge capacity for several culvert materials, shapes, and inlet configurations under inlet control conditions are presented in the nomographs of the Federal Highway Administration. The nomographs are recommended for use in all inlet-control culvert calculations.

3.3.6.4.1.2: Outlet Control Condition.

There are three (3) types of outlet control culvert flow conditions:

1. The headwater submerges the culvert opening, and the culvert outlet is submerged by the tailwater. The culvert will flow full (Condition A, Figure D.26 in Appendix D of this manual).

2. The headwater submerges the culvert opening, the culvert outlet is not submerged by the tailwater (Condition B or C, Figure D.26 in Appendix D of this manual).

3. The headwater is insufficient to submerge the top of the culvert opening. The culvert slope is subcritical and the tailwater depth is lower than critical depth for the culvert (Condition D, Figure D.26 in Appendix D of this manual).

The capacity of a culvert for outlet control is calculated using Bernoulli's Equation, which is based on the conservation of energy principle. In the application of this equation, an energy balance is determined between the headwater at the culvert inlet and the tailwater at the culvert outlet. This balance is a function of inlet losses, friction losses and velocity head (See Figure D.27 in Appendix D of this manual).

Please refer to the publications of the Federal Highway Administration for design calculations.

3.3.6.4.2: Depths of Tailwater and Headwater.

In culverts flowing with outlet control, tailwater is an important factor in computing both the headwater depth and the hydraulic capacity of a culvert. Thus, in many culvert designs, it becomes necessary to determine tailwater depth in the outlet channel.

Much engineering judgment and experience are needed to evaluate possible tailwater conditions during storms. A field inspection should be made to check on downstream controls and to determine water stages. Tailwater is often controlled by a downstream obstruction or by water stages in another stream.

An approximation of the depth of flow in a natural stream (outlet channel) can be made by using Manning's equation in the channel with normal flow condition (see Section 3.3.5.2.1, "Uniform Flow"). If the water surface in the outlet channel is established by downstream controls, a backwater analysis is required (see Section 3.3.5.2.2, "Gradually Varied Flow").

Please refer to the publications of the Federal Highway Administration for required designs.

A series of nomographs for various culvert materials and shapes have been developed by the Federal Highway Administration and the various pipe manufacturers. The nomographs include those for inlet control conditions, outlet control conditions and the critical depth for pipes of different shapes.

3.3.6.4.3: Design Procedures

The Federal Highway Administration has published guidance for the hydraulic design of culverts. Their guidance includes design methodology, check lists, design charts, and tables which shall be used to perform culvert designs that meet the City's performance criteria. In addition, the FHWA distributes a free program titled HY-8 Culvert Hydraulic Analysis Program that uses the FHWA methodology for analysis and design of culverts. This program includes example problems to guide the user on culvert design.

3.3.6.5: Hydraulic Considerations in Bridge Design

3.3.6.5.1: General

For bridges and culverts crossing local streets, runoff from the fully developed 100-year frequency storm shall not produce a headwater elevation at the roadway greater than either twelve (12) inches above the roadway crown elevation or any top of upstream curb elevation, whichever is lower.

For bridges and culverts crossing streets other than a local street, runoff from the fully developed 100-year frequency storm shall not produce a headwater elevation at the roadway greater than six (6) inches above the roadway crown elevation or six (6) inches above any top of upstream curb elevation, whichever is lower.

Several hydraulic parameters should be considered in bridge design. These considerations should include, but should not be limited to, the following:

a. Channel transitions into and out of the bridge opening.

b. Overall length and height of bridge.

c. Cross-sectional opening of bridge.

d. Location of the bridge opening relative to the main channel.

e. Bridge alignment relative to general flow of main channel i.e., "skewed" crossing.

f. Number of crossings or bridge openings.

g. Other obstructions to flow, i.e., piers, abutments, deck width and clearances.

h. Design flows for bridge opening to pass.

i. Any freeboard requirements for channel design.

3.3.6.5.2: Types of Flow for Bridge Design

Three (3) types of flow caused by bridge construction on a flood plain are shown in Figure D.28 in Appendix D of this manual. The three flow types are described below:

a. Type I Flow. Referring to Figure D.28A in Appendix D of this manual, it can be observed that normal water surface is above critical depth at all points. This has been labeled Type I, or subcritical flow, the type usually encountered in practice. The backwater expression for Type I flow is obtained by applying the conservation of energy principle between cross-sections 1 and 4.

b. Type IIA Flow. There are at least two (2) variations of Type II flow which will be described here as Types IIA and IIB. For Type IIA flow, Figure D.28B in Appendix D of this manual, normal water surface in the unconstricted channel again remains above critical depth in the constriction. Once critical depth is penetrated, the water surface upstream from the constriction, and thus the backwater, becomes independent of conditions downstream (even though the water surface returns to normal stage at cross-section 4).

c. Type IIB Flow. The water surface for Type IIB flow, Figure D.28C in Appendix D of this manual, starts out above both normal water surface and critical depth upstream, passes through critical depth in the constriction and then returns to normal. The return to normal depth can be rather abrupt as in Figure D.28C in Appendix D of this manual, taking place in the form of a poor hydraulic jump, since normal water surface in the stream is above critical depth.

d. Type III Flow. In Type III Flow, Figure D.28D in Appendix D of this manual, the normal water surface is below critical depth at all points and the flow throughout is supercritical. This is an unusual case requiring a steep gradient but such conditions do exist, particularly in mountainous regions. Theoretically, backwater should not occur for this type, since the flow throughout is supercritical. It is more than likely that an undulation of the water surface will occur in the vicinity of the constriction, as indicated on Figure D.28D in Appendix D of this manual.

A more thorough and complete discussion of these parameters and preliminary design procedures are presented in Chapters 1 and 11 of Hydraulics of Bridge Waterways by U.S. Department of Transportation Federal Highway Administration, Second Edition, September, 1973.

3.3.6.5.3: Modeling Hydraulic Conditions

The most commonly used backwater program for modeling hydraulic conditions at existing or proposed bridge crossings is the U.S. Army Corps of Engineers HEC-RAS.

CHAPTER 4. RESOURCES

Edwards Aquifer Map Viewer
https://www.tceq.texas.gov/gis/edwards-viewer.html

Watershed Protection Plan Application
https://www.sanmarcostx.gov/DocumentCenter/Home/View/5039

United States Department of Agriculture Natural Resources Conservation Service Web Soil Survey
https://websoilsurvey.sc.egov.usda.gov/App/WebSoilSurvey.aspx

City of San Marcos Design and Construction Guide
http://www.sanmarcostx.gov/367/Design-Construction-Guide

City of Austin Manuals (Drainage Criteria and Environmental Criteria)
https://library.municode.com/TX/Austin/codes/Drainage_Criteria_Manual
https://library.municode.com/TX/Austin/codes/Environmental_Criteria_Manual

Texas Commission on Environmental Quality Stormwater General Permit for Construction Activities
https://www.tceq.texas.gov/permitting/stormwater/construction

Texas Commission on Environmental Quality Complying with the Edwards Aquifer Rules: Technical Guidance on Best Management Practices
https://www.tceq.texas.gov/publications/rg/rg-348

San Antonio River Authority Low Impact Development Technical Guidance Manual
https://www.sara-tx.org/wp-content/uploads/2015/05/Full-LID-Manual.pdf

The Center for Urban Forest Research
https://www.fs.fed.us/psw/topics/urban_forestry/

I-Tree Tools for Urban and Rural Forestry Analysis
https://www.itreetools.org/

APPENDIX A. EROSION AND SEDIMENT CONTROL BMPS

1: INTRODUCTION.....A:2

2: SELECTION AND DESIGN.....A:3
2.1: TEMPORARY PRACTICES A:3
2.2: PERMANENT PRACTICES A:4

3: MAINTENANCE REQUIREMENTS.....A:5

4: INSPECTIONS.....A:5

1: INTRODUCTION

Erosion is a natural process that occurs when soil particles are dislodged and displaced from their original location. This can be caused by water (running over the soil or splash erosion from rain), ice, wind, or gravity. Although erosion is a natural feature, it is greatly accelerated by construction projects. In fact, construction activity is the largest direct source of human-caused erosion. In addition to natural erosion factors, erosion can also be caused by construction vehicles at construction sites.

Erosion caused by new development projects can cause a build-up of sediment and fine particles downstream. Deposition of sediment can choke vegetation, diminish water quality, and reduce the hydraulic capacity of culverts and waterways. Sediment laden runoff and exposed soil therefore must be minimized to the greatest extent practicable.

Maintaining the soil cover at a site is the best way to prevent erosion and downstream pollution during construction. It is easier and cheaper to maintain soil cover than to design and install treatment devices. When soil exposure cannot be prevented (which is often the case on construction sites), temporary erosion and sediment control best management practices (BMPs or practices) must be installed. These practices are installed prior to and during construction of a site to prevent erosion from occurring and, where required, prevent sediment from leaving the site boundaries.

To prevent erosion post-construction, designers should utilize the permanent structural post-construction erosion and sediment control practices outlined in this appendix. However, it is important to implement adequate sustainable site design practices which are appropriate for the ecoregion where the project is occurring as well. These include practices such as preserving natural drainage patterns, limiting disturbance on steeps slopes, avoiding unnecessary cut/fill, minimizing soil compaction and protecting areas with highly erodible soils Drainage objectives can best be accomplished by leaving portions of a subdivision in an underdeveloped and natural state and located to receive runoff from the developed areas for purposes of un-channelized, overland flow. In addition, a sustainable site design should include limiting the creation of impervious surfaces — particularly connected impervious cover — which will increase the volume and velocity of runoff during rain events and increase the likelihood of post-construction erosion (see Section 3 and Appendix B of this manual for more information on selecting sustainable site design practices).

The following is a list of those soils located within the San Marcos area that have a high potential for erosion based on soil conservation service data:

- Branyon clay
- Houston black clay
- Heiden clay
- Anhalt clay
- Ferris clay
- Lewisville silty clay
- Sunev silty clay loam
- Seawillow clay loam
- Denton silty clay
- Bular clay loam
- Altoga silty clay
- Oakalla soils
- (m)Tinn clay
- Oakalla silty clay loam
- Oris soils
- Austin-Castephen complex
- Tarpley clay
- Castephen clay loam
- Purves clay
- Gruene clay
- Doss silty clay
- Krum clay
- Pedernales pine sandy loam

As described in Chapter II of the manual, erosion and sediment controls are required for all regulated construction and development projects. Erosion and sediment control practices must be displayed on the final plan set used for construction. Design of these devices will be verified by the City of San Marcos during the construction review phase. Some projects may require compliance with the TPDES Stormwater Construction General Permit TXR 150000. A copy of the General Permit and associated forms can be found by using the link in the Resources section of this manual.

2: SELECTION AND DESIGN

Erosion and sediment control devices can be divided into three categories:

- Diversion
- Flow spreading/velocity reduction
- Detention/sedimentation/filtration

Determining which erosion and sediment control devices to use on a site depends on a variety of factors, regardless of whether the devices are structural or non-structural. Several of these criteria are identified below.

- Upstream drainage area.
- Site drainage patterns.
- Whether or not upstream flow is concentrated or sheet flow.

Specific restrictions associated with each of the approved erosion and sediment control facilities can be found in the City of Austin Environmental Criteria Manual (see the link in Resources).

The effectiveness of erosion and sediment control devices is dependent upon their design, installation, and maintenance. One of the most common design mistakes is to miscalculate the drainage area leading to an erosion or sediment control device. Assuming a drainage area that is much smaller than the actual drainage area can cause a device to get overloaded with sediment and/or cause a washout of contaminated water. This can destroy the device and damage downstream water bodies. Proper design of a facility is an integral part to their function.

Installation of an erosion and sediment control device is also important for the function of a device. For instance, devices should be installed along the contour or with a constant top elevation. Otherwise, runoff will flow to the low spot within the barrier and can leave the site without proper treatment. Proper installation should be verified by construction inspectors to ensure proper function of a facility.

2.1: TEMPORARY PRACTICES

Temporary erosion and sediment control BMPs are installed before and during construction. These measures are not to be removed until vegetation is established and the construction area is stabilized. Erosion control measures should be installed at a site to prevent erosion from occurring. Often the most effective erosion control measures is to reduce the limits of clearing and grading and phasing a construction project so that a minimum volume of soil is exposed at any one time will reduce the potential for erosion.

There are a variety of methods that can be used to slow down runoff velocities, direct water to stabilized areas and provide surface protection to limit erosion. For example:

- Interceptor Swale
- Diversion Dike
- Pipe Slope Drain
- Polyacrylamide (PAM) Soil Stabilizers
- Outlet Stabilization
- Level Spreader
- Subsurface Drain
- Temporary Vegetation
- Blankets and Matting
- Hydraulic Mulch
- Sod
- Dust Control

When erosion does occur, sediment control BMPs will be required to capture stormwater runoff and remove the sediment from the runoff. Sediment control BMPs improve water quality by settling and/or filtering runoff. The majority of sediment control BMPs act by allowing sediment to settle out of the stormwater.

Typically, devices should be installed along the contour or with a constant top elevation. This will prevent runoff concentrating at a low spot within the barrier. Where larger volumes of runoff need to be captured, sediment traps and basins can be installed.

Sediment control devices should be designed based on the volume of flow they will receive. A sample list of sediment control BMPs can be found below.

- Construction Exit
- Silt Fence (Interior and Exterior)

- Triangular Filter Dike
- Berms (such as Rock Berms, High Service Rock Berms, Brush Berms, and Sand Bag Berms)
- Vegetative Buffer Strips
- Inlet Protection
- Sediment Trap
- Sediment Basin
- Filter Rolls
- Dewatering Operations
- Spill Prevention
- Utility Line Crossings
- Concrete Washout

Temporary detention/sedimentation structures must be designed to withstand the force and velocity from a 10-year frequency storm without failing. Larger storms shall be bypassed via stabilized conveyances. Those devices that employ sedimentation must provide the storage volume for the runoff from a 2-year, 24-hour storm under compacted site conditions. The sedimentation basins must be designed such that drawdown time is 72 hours via surface skimmers. The design must include considerations for overflows to ensure that the device and its detention pool remain intact.

Detention/sedimentation structures should not be sited in natural drainage channels, draws or ravines that are directly connected to off-site drainage features like creeks, rivers, ponds or recharge features. In particular, this means that silt fences should not be used to control concentrated or channelized flow and sedimentation basins shall not be constructed in natural draws because failures of the earthen retaining system are often catastrophic to the downstream receiving waters.

Many temporary erosion and sediment control practices can be designed and installed to later function as permanent facilities.

Information about designing these facilities, as well as installation and maintenance, can be found in the Edwards Aquifer Design Manual.

2.2: PERMANENT PRACTICES

Permanent structural erosion and sediment control devices are intended to remain on a site after construction.

There are several permanent E&SC features that can be installed at a construction site based on need. They include:

- Diversions
- Grass-lined Swales
- Level Spreaders
- Stone Riprap
- Gabions
- Subsurface Drains
- Land Grading
- Grade Stabilization Structure (Paved Chute or Flume)

and all prevent future sediment runoff. These controls can be structural or permanent surface stabilization. Permanent controls are designed for less frequent (larger) design storms than temporary measures in order to ensure that they remain effective throughout their life. Velocities exiting the stormwater controls must be non-erosive. For the City of San Marcos, these controls should be designed for the 25-year, 24 hour storm event. Flows from storm events between 25-year and 100-years should remain within rights-of-way or drainage easements.

Permanent vegetation must be established throughout the areas disturbed by construction to prevent post-construction erosion. Vegetation should be selected based upon the proposed land use, desired aesthetic or landscaping and ecoregion (see Appendix B). Maintenance of the final site should be considered when selecting vegetation material (see *San Antonio River Authority Low Impact Development Technical Guidance Manual* for a list of acceptable plant materials). In addition, native species should be used at a site. Vegetation can be established by means of seeding, sodding, sprigging, seed drilling or soil retention blankets. Permanent vegetation installation should occur within seven days after the completion of construction activities, or after each phase of construction.

Construction inspections should be performed by a design professional of record or a certified inspector with specific training and experience on E&SC device construction.

3: MAINTENANCE REQUIREMENTS

Maintenance of sediment control devices are integral to their performance. Contractors are required to periodically remove sediment buildup from E&SC BMPs to allow them to continue functioning properly. Damaged BMPs must be repaired or replaced upon discovery. Otherwise, sediment can wash out from facilities and cause damage downstream.

Maintenance of erosion and sediment control facilities is also vital to their functionality. Sediment must be periodically removed from the facilities so that washouts do not occur. In addition, damaged or destroyed facilities must be repaired or replaced upon discovery.

The Responsible Official will monitor stormwater discharges from these activities to evaluate the adequacy of the temporary erosion and sedimentation control measures. The Responsible Official may require the person performing the activity to use additional controls if the Responsible Official determines that the controls used by the person are inadequate to protect water quality.

Detailed instructions on maintenance for each of the recommended erosion and sediment control devices can be found in the TCEQ's Complying with the Edwards Aquifer Rules: Technical Guidance on Best Management Practices RG-348 Manual (see the link in Resources).

4: INSPECTIONS

To maintain the effectiveness of E&SC devices, regular inspection is essential. Generally, inspection and maintenance can be categorized as routine and as needed. Routine activities, performed regularly (e.g., monthly) ensure that the devices are in good working order. Routine inspection is an efficient way to prevent potential nuisance situations from developing and reduce the need for repair or maintenance. Routine inspection also reduces the chance of degrading the quality of the effluent by identifying and correcting potential problems regularly. Property maintenance personnel should be instructed to inspect E&SC devices during their normal routines.

In addition to routine inspections, as-needed inspection/maintenance of all practices should be performed after any event or activity that could cause damage, particularly after every large storm event.

Routine and as-needed inspections consist of technical and non-technical activities as summarized below:

- Inspect the general conditions of the E&SC practice and areas directly adjacent.
- Maintain access to the site including the inlets, side slopes (if applicable), forebay (if one exists), device area, outlets, emergency spillway, and so on.
- Examine the overall condition of vegetation.
- Eliminate any possibility of public hazards (vector control, unstable public access areas).
- Check the conditions of inflow points, pretreatment areas (if they exist), and outlet structures.
- Inspect and maintain the inlet and outlet regularly and after large storms.
- Ensure that the pretreatment areas meet the original design criteria.
- Check the encroachment of undesirable plants in vegetated areas. This could require more frequent inspections in the growing season.
- Inspect water quality improvement components. Specifically, check the stormwater inflow, conveyance, and outlet conditions.
- Inspect hydrologic functions such as maintaining sheet flow where designed, ensuring functional pretreatment, maintaining adequate design storage capacity, and verifying proper operation of outlet structures.
- Check conditions downstream of the E&SC device to ensure that flow is properly mitigated below the facility (e.g., excessive erosion, sedimentation).

In every inspection, whether routine or as needed, the inspector should document whether the E&SC device is performing correctly and whether any damage has occurred to it since the last inspection. Ideally, the inspector will also identify what should be done to repair the E&SC device if damage has occurred. Documentation is very important in maintaining an efficient inspection and maintenance schedule, providing evidence of ongoing inspection and maintenance, and detecting and reporting any necessary changes in overall management strategies.

APPENDIX B. LOW IMPACT DEVELOPMENT PRACTICES

B.1: LOW IMPACT DEVELOPMENT STORMWATER CONTROL MEASURES B:3
B.1.1: INTRODUCTION B:3
B.1.2: INSPECTIONS B:7
B.1.3: REFERENCES B:7

B.2: SITE PLANNING/SCM SELECTION B:8
B.2.1: EXAMPLE LID CONCEPTUAL DESIGN B:10

B.3: RAIN GARDENS AND BIORETENTION FACILITIES..... B:31
B.3.1: DESCRIPTION B:31
B.3.2: SELECTION CRITERIA B:33
B.3.3: DESIGN SPECIFICATIONS B:33
B.3.4: INSPECTION CRITERIA B:49
B.3.5: RECOMMENDED MAINTENANCE B:50
B.3.6: REFERENCES B:52

B.4: POROUS PAVEMENT B:54
B.4.1: DESCRIPTION B:54
B.4.2: SELECTION CRITERIA B:55
B.4.3: DESIGN B:55
B.4.4: CONSTRUCTION CONSIDERATIONS B:66
B.4.5: INSPECTION CRITERIA B:67
B.4.6: RECOMMENDED MAINTENANCE B:69
B.4.7: REFERENCES B:69

B.5: RAINWATER HARVESTING..... B:71
B.5.1: DESCRIPTION B:71
B.5.2: SELECTION CRITERIA B:72
B.5.3: DESIGN SPECIFICATIONS B:72
B.5.4: CONSTRUCTION CONSIDERATIONS B:82
B.5.5: INSPECTION CRITERIA B:82
B.5.6: OPERATION AND MAINTENANCE B:83
B.5.7: REFERENCES B:84

B.6: ADDITIONAL LID SCMS B:85
B.6.1: VEGETATED FILTER STRIPS B:85
B.6.2: VEGETATED SWALES B:86
B.6.3: GREEN ROOFS B:88
B.6.4: CONSTRUCTED WETLANDS AND WET PONDS B:90
B.6.5: PROPRIETARY SYSTEMS B:91

B.7: TREATMENT TRAINS B:92

B.8: COMMON DESIGN ELEMENTS B:93
B.8.1: GEOTECHNICAL INVESTIGATION B:93
B.8.2: CURB CUTS B:93
B.8.3: STABILIZATION AND ENERGY DISSIPATION B:97
B.8.4: UNDERDRAIN DESIGN B:98
B.8.5: TRASH RACKS B:100
B.8.6: DIVERSION STRUCTURES B:101
B.8.7: IMPERMEABLE LINERS AND HYDRAULIC RESTRICTION BARRIERS B:102
B.8.8: OUTLET STRUCTURES B:104
B.8.9: UTILITIES B:105
B.8.10: CONNECTIVITY B:105
B.8.11: ADA REQUIREMENTS B:107
B.8.12: REFERENCES B:107

B.1: LOW IMPACT DEVELOPMENT STORMWATER CONTROL MEASURES

In order for Low Impact Development (LID) practices to perform effectively, basic guidelines need to be followed in their selection, design, construction, and maintenance. This appendix is written to guide professionals through the design process. Detailed requirements are provided for the three most widely used LID practices appropriate for the San Marcos area; bioretention, permeable pavement, and rainwater harvesting. Lastly, this appendix discusses several other structural controls in brief – green roofs, constructed wetlands, and proprietary systems – and provides links to resources for more information if projects in the San Marcos region wish to implement these practices.

LID practices use natural features to slow and filter stormwater runoff. Project characteristics will define which LID Stormwater Control Measures (SCMs) are applicable. When determining the appropriate LID requirements, project managers must consider characteristics such as site location, existing topography and soils, and planning elements. These characteristics and their impacts on design are important because LID SCMs are permanent features that can affect other project elements; therefore, it is critical to conduct thorough site assessments to avoid the need for redesign later. Incorporating LID early in the site design stage could reduce the need for and cost of traditional drainage infrastructure by reducing the amount of stormwater to be conveyed off-site.

Though the LID toolbox is unlimited, this appendix focuses on the above structural tools as they are most appropriate for the San Marcos region. Further, many of these practices are most effective at reducing both runoff volume and pollutant loads.

B.1.1: INTRODUCTION

LID features employ principles such as preserving and recreating natural landscape features, minimizing effective imperviousness, and employing processes of infiltration, filtration, storage, evaporation and detention of stormwater runoff. LID features can be structural or non-structural. By implementing LID principles and practices, water can be managed in a way that reduces the impact of built areas and promotes the natural movement of water within an ecosystem or watershed. Applied on a broad scale, LID can maintain or restore a watershed's hydrologic and ecological functions. These principles, and the strategies described below can be of the most benefit when used together, often in a linked series of practices, referred to as the treatment train approach.

B.1.1.1: Types

This appendix will describe in detail both non-structural and structural methods for reaching LID goals. Non-structural sustainable site design practices should come first and include practices such as keeping existing trees on-site, minimizing compaction of earth that inhibits water infiltration, and planting trees and other vegetation in areas where none exists. Retaining existing tree cover and vegetated areas helps infiltrate and evapotranspire stormwater runoff while intercepting large amounts of rainfall that would otherwise enter waterways as runoff. These site design approaches should be considered early in the design process and are detailed in Section B.2

B.1.1.1.1: Structural LID Measures

Structural LID practices mimic natural features to convey, treat, and infiltrate stormwater runoff. A variety of structural LID measures exist, including:

- Rain Gardens and Bioretention facilities
- Porous Pavement
- Rainwater Harvesting
- Vegetated Filter Strips
- Vegetated Swales
- Green Roofs
- Constructed Wetlands and Wet Ponds
- Proprietary Systems

Integrating LID measures into site design is a key component of integrated site design and stormwater management. This appendix details how to integrate LID measures into site design, determination of which LID measures work for which sites, and how to design structural LID measures. Designers should utilize this Appendix to determine the best LID measures to use on their sites. Structural LID measures require detailed engineering design in order to meet performance requirements. Additional information on detailed design and sizing is included in Section III-B-2 of this manual.

B.1.1.2: Functionality

LID strategies are often implemented at three scales: the region or large watershed area, the community or neighborhood, and the

site or block. Different stormwater approaches are used at different scales to afford the greatest degree of protection to waterbodies because the influences of pollution are often found at all three scales.

This manual focuses on LID strategies at the site scale. In that context, LID is best described as a process where you plan, design, construct and manage each development site to protect, or restore, the natural hydrology (the scientific study of the properties, distribution and effects of the water on the earth's surface) of the site so that the overall integrity of the watershed is protected. By creating a "hydrologically" functional landscape, you can reduce the potentially negative impacts of storm water quantity and quality.

B.1.1.3: Benefits

Future development will continue to create challenges for maintaining and improving water quality in San Marcos' waterbodies. Nationwide, the past few decades of stormwater management have resulted in the current convention of control-and-treatment strategies. These traditional strategies are largely engineered, end-of-pipe practices that have been focused on controlling peak flow rate (i.e. flood control) and reducing suspended solids loadings. However, these practices fail to address the widespread and cumulative hydrologic modifications within the watershed that increase stormwater volumes and runoff rates, and cause excessive erosion and stream channel degradation. Existing practices also fail to adequately treat for other pollutants of concern, such as nutrients, pathogens, and metals.

LID can alleviate some of these challenges posed by development. Note that environmental and ancillary benefits typically are not measured as part of development projects, nor are they measured as part of pilot or demonstration projects, because they can be difficult to isolate and quantify.

Incorporating Low Impact Design into site planning involves designing landscapes that are multi-functioning. In addition to providing flood control and water quality benefits, LID facilities create habitat for natural resources, improve air quality and aesthetics, and may offer recreational opportunities.

B.1.1.1.1: Environmental Benefits

LID measures can reduce both the volume of runoff and the pollutant loadings discharged into receiving waters. LID measures result in pollutant removal through settling, filtration, adsorption, and biological uptake.

The National Pollutant Removal Performance Database, compiled by the Center for Watershed Protection (www.cwp.org), is a good resource for examining pollutant removal data from multiple monitoring sites across the country.

Reductions in pollutant loadings to receiving waters also improve habitat for aquatic and terrestrial wildlife and enhance recreational uses. Reducing pollutant loadings can decrease stormwater and drinking water treatment costs by decreasing the need for regional stormwater management systems and expansions in drinking water treatment systems.

Vegetated LID measures can provide air quality benefits, particularly those that incorporate trees. Trees absorb air pollutants, notably carbon dioxide (CO_2) but also nitrogen dioxide (NO_2), ground-level ozone (O_3), sulfur dioxide (SO_2), and particulate matter that is 10 μm or smaller (PM-10). +-Vegetation's ability to sequester carbon can help to meet greenhouse gas emission goals by contributing to a carbon sink (CNT 2010).

Trees create shade that reduce indoor air temperatures and reduce the demand for energy for cooling. This yields direct cost savings to electricity consumers and, through reduced electricity demand, reduces air pollution emissions from electricity generation. Reduced emissions of air pollution benefits human health through lowered incidence and severity of respiratory ailments and reduces costs associated with air quality regulation compliance (ECONorthwest 2011).

LID measures that include trees and other vegetation can reduce the urban heat island effect, which is the phenomenon of urban area temperatures that are several degrees higher than surrounding rural land uses. The U.S. EPA (2012b) indicates that annual mean air temperature can be 1.8 °F to 5.4 °F higher in urban centers and up to 22 °F higher in the evening. Tree cover does not absorb heat like pavements do, and trees reduce temperatures through shading and evapotranspiration. Reducing urban heat islands through tree planting achieves energy reduction (reduced need for cooling, along with the ancillary benefits described above) and can reduce the incidence and severity of heat-related illnesses.

B.1.1.2.2: Protection of Downstream Water Resources

LID measures can be used to protect water resources that are downstream in the watershed. These practices can help to prevent or reduce hydrologic impacts on receiving waters, reduce stream channel degradation from erosion and sedimentation, improve water

quality, increase water supply, and enhance the recreational and aesthetic value of our natural resources.

Other potential benefits include reduced incidence of illness from contact recreation activities such as swimming and wading, more robust and safer seafood supplies, and reduced medical treatment costs.

B.1.1.3.3: Groundwater Recharge

LID measures also can be used to infiltrate runoff to recharge groundwater. Growing water shortages nationwide increasingly indicate the need for water resource management strategies designed to integrate stormwater, drinking water, and wastewater programs to maximize benefits and minimize costs. Development pressures typically result in increases in the amount of impervious surface and volume of runoff. Infiltration practices can be used to replenish groundwater and increase stream base flow. Adequate base flow to streams during dry weather is important because low groundwater levels can lead to greater fluctuations in stream depth, flows, and temperatures, all of which can be detrimental to aquatic life.

B.1.1.4.4: Water Quality Improvements and Reduced Treatment Costs

Keeping water clean is almost always less expensive than cleaning it up. The Trust for Public Land noted Atlanta's tree cover has saved more than $883 million by preventing the need for stormwater retention facilities[1]. A study of 27 water suppliers conducted by the Trust for Public Land and the American Water Works Association found a direct relationship between forest cover in a watershed and water supply treatment costs. According to the study, approximately 50 to 55% of the variation in treatment costs can be explained by the percentage of forest cover in the source area. The researchers found that for every 10% increase in forest cover in the source area, treatment and chemical costs decreased approximately 20%, up to about 60% forest cover.[2] In other words, communities with higher percentages of forest cover had lower treatment costs.

B.1.1.5.5: Habitat Improvements

Innovative stormwater management techniques like LID or conservation design can be used to improve natural resources and wildlife habitat, or avoid expensive mitigation costs. Conservation design strategies set aside large parcels of undisturbed land that will not be developed but rather left as open space or as a conservation easement.

Aquatic habitat improvements can also be seen from LID measures as the quality, volume, rate, and temperature entering receiving waterbodies is more closely associated with pre-development conditions.

B.1.1.6.6: Land Value and Quality of Life Benefits

Many of the direct and indirect benefits of LID are derived from improved land value - through improved aesthetics, additional lot yield, or property protection – and quality of life benefits. These latter benefits are some of the most difficult to quantify, yet are also some of the most important for a community as LID techniques can help brand a community, provide multiple amenities, and provide for an improved landscape and sense of place.

B.1.1.7.7: Reduced Downstream Flooding

LID measures can be used to reduce downstream flooding through the reduction of peak flows and the total amount or volume of runoff. Flood prevention reduces property damage and can reduce the initial capital costs and the operation and maintenance costs of stormwater infrastructure. Strategies designed to manage runoff on-site or as close as possible to its point of generation can reduce erosion and sediment transport as well as reduce flooding and downstream erosion. As a result, the costs for cleanups and streambank restoration can be reduced or avoided altogether. The use of LID techniques also can help protect or restore floodplains, which can be used as park space or wildlife habitat.[3]

B.1.1.8.8: Real Estate Value/Property Tax Revenue

Homeowners and property owners are willing to pay a premium to be located next to or near aesthetically pleasing amenities like water features, open space, and trails. Some stormwater treatment systems can be beneficial to developers because they can serve as a "water" feature or other visual or recreational amenity that can be used to market the property. These designs should be visually attractive and safe for the residents and should be considered an integral part of planning the development. Various LID projects and smart growth studies have shown that people are willing to pay

1 The Trust for Public Land. *The Economic Benefits of Land Conservation*. (San Francisco, CA: Trust for Public Land, 2007).

2 Trust for Public Land and American Water Works Association. *Protecting the Source* (San Francisco, CA: Trust for Public Land, 2004).

3 The Trust for Public Land. *The Economic Benefits of Land Conservation*. (San Francisco, CA: Trust for Public Land, 2007).

more for clustered homes than conventionally designed subdivisions. Further, many studies have shown examples where developers and subsequent homeowners have received premiums for proximity to attractive stormwater management practices.[4]

B.1.1.9.9: Lot Yield

LID measures typically do not require the large, contiguous areas of land that are usually necessary when traditional stormwater controls like ponds are used. In cases where LID measures are incorporated on individual house lots and along roadsides as part of the landscaping, land that would normally be dedicated for a stormwater pond or other large structural control can be developed with additional housing lots.

For more information on the cost-benefits of LID, visit www.texaslid.org for a bibliography of cost-benefit resources and case studies.

B.1.1.10.10: Aesthetic Value

LID techniques are usually attractive features because landscaping is an integral part of the designs. Designs that enhance a property's aesthetics using trees, shrubs, and flowering plants that complement other landscaping features can be selected. The use of these designs may increase property values or result in faster sale of the property due to the perceived value of the "extra" landscaping.

Green infrastructure that includes attractive vegetation can improve property aesthetics, which can translate into increased property values (more information is in Section 1.1.3.6, Land Value and Quality of Life Benefits). This vegetation also provides habitat for urban wildlife, particularly birds and insects, even at small scales of implementation. Larger-scale facilities that include public access, such as constructed wetlands, offer recreational opportunities (e.g., fishing, bird-watching) as well as habitat for wildlife and water quality/quantity improvements.

B.1.1.11.11: Public Spaces/Quality of Life/Public Participation

Placing water quality practices on individual lots provides opportunities to involve homeowners in stormwater management and enhances public awareness of water quality issues. An American Lives, Inc., real estate study found that 77.7 percent of potential homeowners rated natural open space as "essential" or "very important" in planned communities.[5]

B.1.1.4: Limitations

Non-structural LID measures can be limited in their effectiveness to treat stormwater runoff. Constructing impervious area will cause an increase in stormwater runoff volume and can potentially decrease the downstream water quality. Incorporating non-structural LID measures helps to mitigate for the increase in impervious area, but it cannot alleviate the increase alone. Non-structural LIDs often require structural LID measures to be fully effective.

As with traditional SCMs, the effectiveness of structural LID measures is dependent upon their design, installation, and maintenance. Proper design ensures that a structural LID practice will function appropriately based on the characteristics of the surrounding landscape. Proper installation and maintenance ensures that the LID practice will not fail and cause erosion and contamination downstream.

B.1.1.5: Operation and Maintenance

The major goal of SCM operation and maintenance is to ensure that the SCM is meeting the specified design criteria for stormwater flow rate, volume, and water quality control functions. If structural LID systems are not properly maintained, SCM effectiveness can be reduced, resulting in water quality impacts. The design professional should provide an O&M manual with the construction documents or final as-built plans. It is important that routine maintenance and any need-based repairs for a structural SCM be completed according to schedule or as soon as practical after the problem is discovered. Deferred SCM maintenance could result in detrimental effects on the landscape and increased potential for water pollution and local flooding.

Training should be included in program development to ensure that maintenance staff has the proper knowledge and skills. Most structural SCM maintenance work—such as mowing, removing trash and debris, removing sediment, and the like—is non-technical and is already performed by property maintenance personnel. More specialized maintenance training might be needed for more complex systems.

4 USEPA, *Economic Benefits of Runoff Controls* (Washington, DC: U.S. Environmental Protection Agency, Office of Water, 1995).

5 National Park Service, *Economic Impacts of Protecting Rivers, Trails, and Greenway Corridors: A Resource Book* (National Park Service, 1995).

Non-structural LID measures, such as reduction of impervious cover and impervious area disconnect, are typically part of the design stage, but are not constructed. Operation and maintenance of many non-structural LID measures are therefore not included as part of the site development construction phase.

B.1.2: INSPECTIONS

To maintain the effectiveness of structural SCMs, regular inspection is essential. Generally, SCM inspection and maintenance can be categorized as routine and as needed. Routine activities, performed regularly (e.g., monthly) ensure that the SCM is in good working order and continues to be aesthetically pleasing. Routine inspection is an efficient way to prevent potential nuisance situations from developing and reduce the need for repair or maintenance. Routine inspection also reduces the chance of degrading the quality of the effluent by identifying and correcting potential problems regularly. Property maintenance personnel should be instructed to inspect SCMs during their normal routines.

In addition to routine inspections, as-needed inspection/ maintenance of all SCMs should be performed after any event or activity that could damage the SCM, particularly after every large storm event. Post-storm inspections should occur after the expected drawdown period for the SCM, when the inspector can determine if the SCM is draining correctly.

Routine and as-needed SCM inspections consist of technical and non-technical activities as summarized below:

- Inspect the general conditions of the SCM and areas directly adjacent.
- Maintain access to the site including the inlets, side slopes (if applicable), forebay (if one exists),
- Examine the overall condition of vegetation.
- Eliminate any possibility of public hazards (vector control, unstable public access areas).
- Check the conditions of inflow points, pretreatment areas (if they exist), and outlet structures.
- Inspect and maintain the inlet and outlet regularly and after large storms.
- Ensure that the pretreatment areas meet the original design criteria.
- Check the encroachment of undesirable plants in vegetated areas. This could require more frequent inspections in the growing season.
- Check conditions downstream of the SCM to ensure that flow is properly mitigated below the facility (e.g., excessive erosion, sedimentation).

In every inspection, whether routine or as needed, the inspector should document whether the SCM is performing correctly and whether any damage has occurred to the SCM since the last inspection. Ideally, the inspector will also identify what should be done to repair the SCM if damage has occurred. Documentation is very important in maintaining an efficient inspection and maintenance schedule, providing evidence of ongoing inspection and maintenance, and detecting and reporting any necessary changes in overall management strategies.

B.1.3: REFERENCES

CNT (Center for Neighborhood Technology). 2010. *The Value of Green Infrastructure: A Guide to Recognizing its Economic, Environmental, and Social Benefits*. http://www.cnt.org/repository/gi-values-guide.pdf. Accessed September 28, 2018.

ECONorthwest. 2011. *Economic Benefits of Green Infrastructure: Chesapeake Bay Region*. ECONorthwest, Eugene, OR.

B.2: SITE PLANNING/SCM SELECTION

When initially performing site layouts, the designer should evaluate where traditional and LID SCMs should be utilized. Evaluating which practices are optimal for a site prior to the design phase, rather than after, has many benefits. Primarily, performing this evaluating concurrent with the design will reduce the environmental footprint of a site, while retaining and enhancing the owner or developer's purpose and vision.

Incorporating sustainable site design into the site planning process yields high benefits for the overall site. For instance, incorporating this design early in the site design stage can reduce construction costs for the project. There are several goals that should be accomplished when obtaining sustainable site design. They include:

- Prevent stormwater impacts rather than having to mitigate for them;
- Manage stormwater (quantity and quality) as close to the source as possible and minimize the use of large or regional collection and conveyance;
- Preserve natural areas, healthy soils, native vegetation and reduce the impact on watershed hydrology;
- Use natural drainage pathways as a framework for site design;
- Reduce soil compaction during construction to maintain infiltration capacities of the soil;
- On disturbed sites, to increase the organic content of the soil to maximize its water retention capacity and improve success of re-vegetation;
- Minimize the amount of disturbance to existing, mature stands of vegetation;
- Utilize simple, non-structural methods for stormwater management that are lower cost and lower maintenance than structural controls;
- Reduce capacity requirements for detention SCMs by incorporating LID SCMs early in the planning process;
- Create a multifunctional landscape which considers construction and maintenance implications; and
- Use appropriate plant species and communities for the eco-region and the designed media.

Sustainable site design incorporates approaches to new and redevelopment projects which reduce impacts on watersheds by conserving natural areas, and better integrating stormwater management. The aim of sustainable site design is to reduce the environmental "footprint" of the site while retaining and enhancing the owner/developer's purpose and vision for the site. Many of the sustainable site design concepts employ non-structural on-site treatment that can reduce the cost of infrastructure while maintaining or even increasing the value of the property relative to conventional designed developments.

Creating a site plan that integrates LID and SCM practices can be accomplished using a nine step process. The nine steps are covered in three stages of the design. A graphical representation of the steps is shown in Figure B.2.1.

The nine steps towards creating an optimal site plan can be sectioned into three phases. The first phase is the site assessment phase. That is followed by the preliminary site design, and finally, the final design.

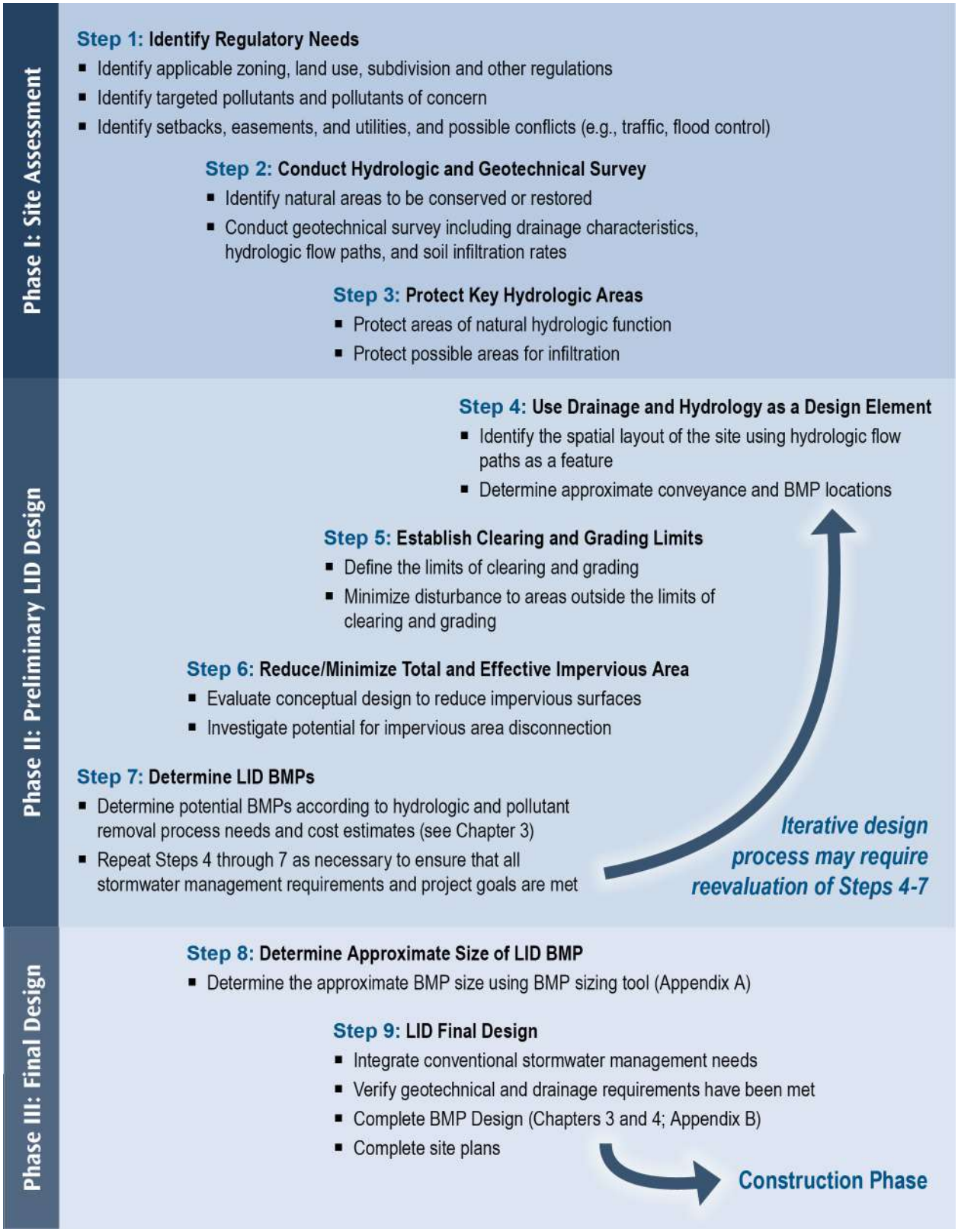


FIGURE B.2.1 STEPS TO DEVELOP A SITE PLAN WITH INTEGRATED STORMWATER MANAGEMENT.

B.2.1: EXAMPLE LID CONCEPTUAL DESIGN

To illustrate the step in the site planning and design process, a series of images was created to show users of this manual what type of information is useful. The images follow the steps and show how the site planning process changes with each step. Starting with Figure B.2.2 below, the images build upon each other to describe the process of developing a LID compliant residential development in San Marcos.

B.2.1.1: Phase I – Site Assessment

The first phase of site planning is composed of the site assessment. Steps 1 through 3 below define the site assessment process.

B.2.1.1.1: Step One: Identify Regulatory Requirements

Stormwater management SCMs must be consistent with the applicable federal, state, and local regulations. A submittal requirement checklist for drainage has been prepared by the City of San Marcos and is available online. Please contact COSM Engineering and Capitol Improvement Department for the latest version of the checklist. Each of the items below should be evaluated by the site planning and design team.

Identify applicable zoning land use, subdivision, and other local regulations. San Marcos' zoning requirements can be found online through the Planning and Development Services website.

Identify setbacks, easements and utilities. The San Marcos Municipal Code provides the basic regulations regarding the size and scale of development, such as permitted density, setbacks and structure height on the basis of the applicable zoning code. Setbacks will restrict the buildable area.

Planning and assessment must also include identifying easements on the site. Easements that could be present are a road or sidewalk (right-of-way) easement; a public utility easement that allows a utility to run gas, water, sewer or power lines through a private property; or a railway easement; drainage and flood maintenance easements or SCM maintenance areas. Local utilities departments (e.g., electric, wastewater) should be consulted to determine

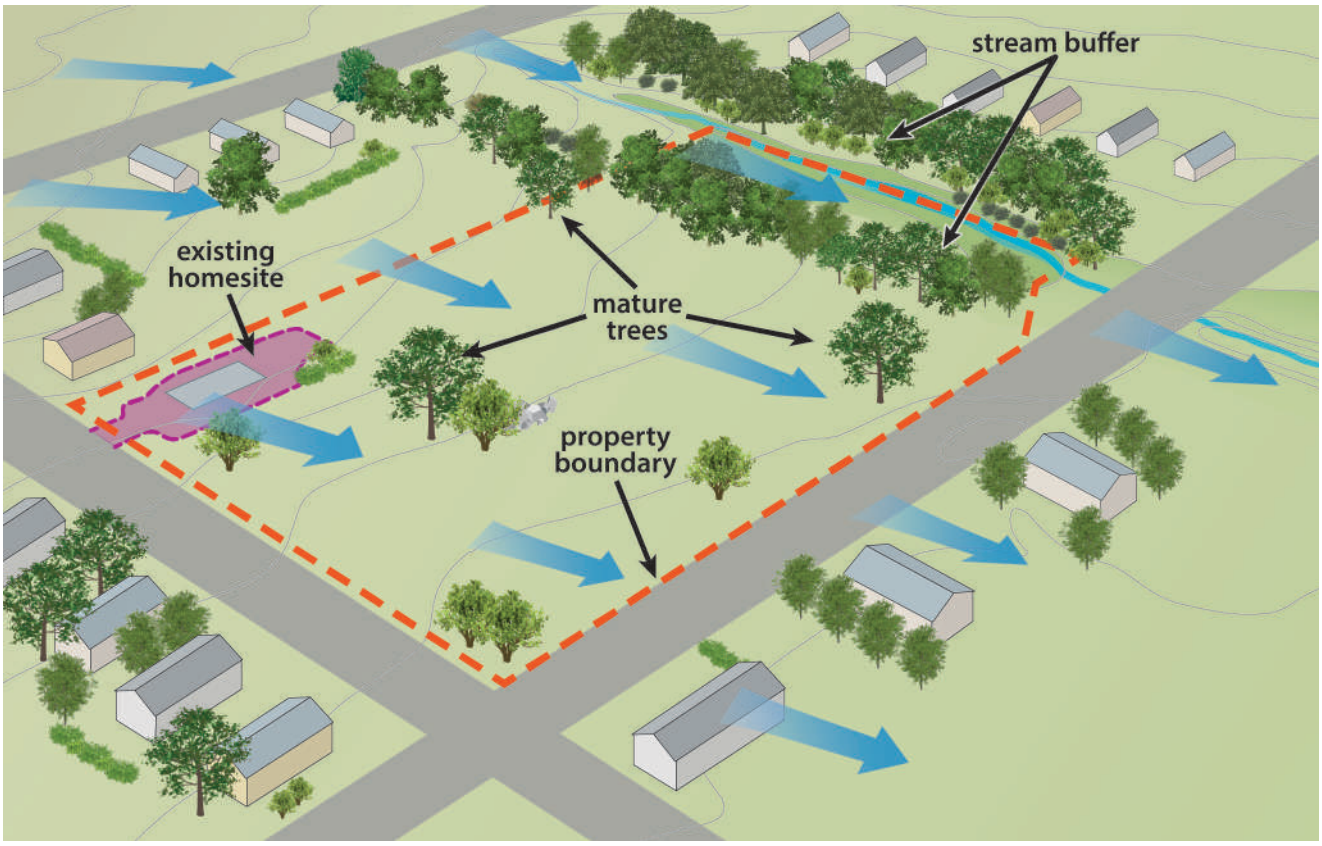


FIGURE B.2.2 EXAMPLE RESIDENTIAL DEVELOPMENT CONCEPTUAL SITE FOR LID AND DETENTION.

whether utilities are above or below ground and the required distance that site disturbance should be maintained from any utilities present. Easements on a site can be determined by consulting as-built drawings and records research. Easements should be included on site drawings.

Identify applicable local environmental regulations. Chapter 6 of the San Marcos Development Code specifies the types of environmental regulations which are applicable to various parts of the City.

Identify areas of contaminated soils at brownfield sites. A brownfield is a property, the expansion, redevelopment, or reuse of which may be complicated by the presence or potential presence of a hazardous substance, pollutant, or contaminant. Contaminated areas may need to be capped with an impervious surface and/or avoided for the purposes of SCM infiltration.

After all of the site assessment steps have been completed, all of the physical requirements (setbacks, easements, etc.) should be displayed on the site plan. By doing so, designers can visualize which parts of the site should not be used, and which are available for construction and for stormwater management facilities. An example site plan with setbacks shown is displayed in Figure B.2.3.

B.2.1.2.2: Step Two: Conduct Hydrologic and Geotechnical Survey

Hydrologic and geotechnical features are critical elements used in determining which SCMs can be used at a site. There are several components to completing the hydrologic and geotechnical investigations.

Conduct geotechnical survey, including drainage characteristics, hydrologic flow paths, and soil infiltration rates. Any project that includes LID measures should include a soil evaluation or geotechnical investigation. A licensed engineer (P.E.) with geotechnical expertise, a licensed geologist, engineering geologist, hydrogeologist, or other licensed professional acceptable to the local jurisdiction should perform a detailed evaluation of soils, shallow ground water and bedrock conditions. A soil evaluation including soil infiltration testing is intended to identify and protect soils that provide greater infiltration as potential locations for LID SCMs. The presence and

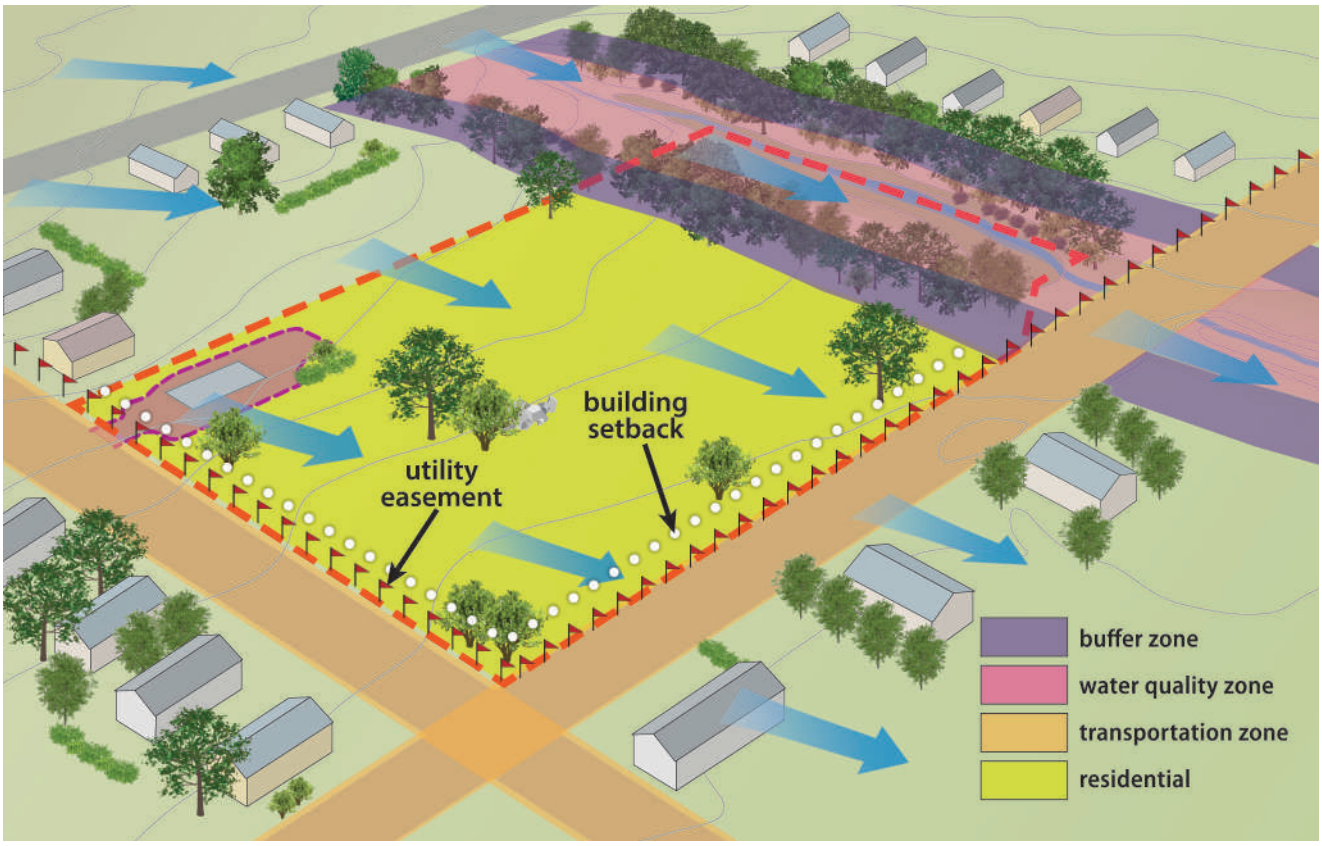


FIGURE B.2.3 IDENTIFY APPLICABLE ZONING REQUIREMENTS, UTILITY EASEMENTS, AND SITE SETBACKS.

depth to the seasonal water table or shallow bedrock should also be identified, which will influence the type of SCM that can be installed at a site. In addition, natural drainage characteristics and hydrologic flow paths should be identified. These features can be used in the design and protected in future steps to maintain the site's natural drainage characteristics.

Areas of a site with hydrologic soil group A and B soils, such as sands and sandy loam soils, should be conserved as much as possible and these areas should ideally be incorporated into undisturbed natural or open space areas. Conversely, buildings and other impervious surfaces should be located on those portions of the site with the least permeable soils. Similarly, areas on a site with highly erodible or unstable soils should be avoided for land disturbing activities and buildings to prevent erosion and sedimentation problems as well as potential future structural problems. These areas should be left in an undisturbed and vegetated condition.

Determine the eco-region the project is in and what specific considerations for LID practice which may be necessary. The eco-regions of the San Marcos area can help inform LID design decisions by placing in context, hydrology, geology, soil types, rainfall patterns, and plant communities. Consideration of the different landscape characteristics assists in items as preliminary as BMP placement or choice, to farther reaching items such as long-term maintenance. For example, residential development in the Edwards Plateau would require minimizing development near karst topography and riparian river crossings due to limited soil layers and steep slopes that increase erosion possibilities and pose a threat to water quality. Additionally, from a regulatory standpoint, implementation of LID in certain portions of the Edwards Plateau will require design modifications as described below. In areas that have more significant soil profiles and gradual slopes such as the Blackland Prairie, infiltration rates will differ from that of the nearby Edwards Plateau eco-region. One way to ensure preservation of San Marcos' waterbodies is to implement non-structural LID measures such as conservation design, preservation of existing resources, such as healthy riparian buffers, or disconnection of impervious surfaces. Further, when using vegetated LID measures, appropriate species selection should also be considered for each of the eco-regions, particularly when using native soils.

It is important to understand the local eco-regions in San Marcos before proceeding with LID design and implementation because their characteristics all impact LID implementation. It is also important to review current regulatory guidance for the Edwards Aquifer regions in San Marcos before proceeding with LID implementation due to limitations and requirements on infiltration. Nevertheless, the full suite of LID measures can be implemented in all areas of San Marcos and will provide substantial benefit for the management of stormwater. Maintenance requirements of LID measures will be the same regardless of the underlying geology or physiographic region. Considering the eco-regions throughout design, construction, and maintenance processes will ensure that LID implementation will be long-lasting, safe, and cost- effective.

San Marcos Eco-Regions Eco-regions are distinct zones based on the analysis of patterns and composition of characteristics such as geology, physiography, vegetation, climate, soils, land use, wildlife, and hydrology. Of the USDA/EPA 85 Level III eco-regions in the continental United States, two are represented in San Marcos: the Edwards Plateau and the Blackland Prairie (Figure B.2.4). The geographical juxtaposition of these two distinct eco-regions results in a great diversity of landscapes and species throughout the area and constitutes the iconic Central Texas landscape. Awareness of the characteristics of these eco-regions is essential for LID design, success and longevity.

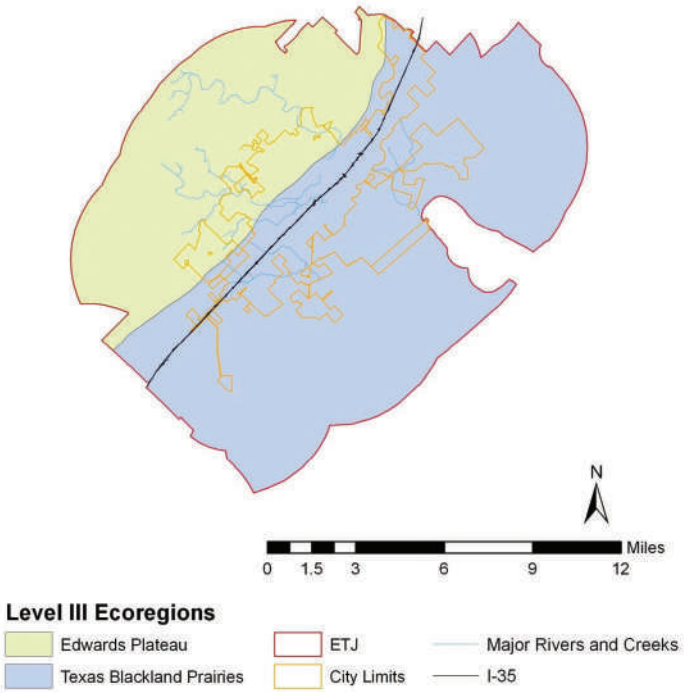


FIGURE B.2.4 SAN MARCOS ECO-REGIONS

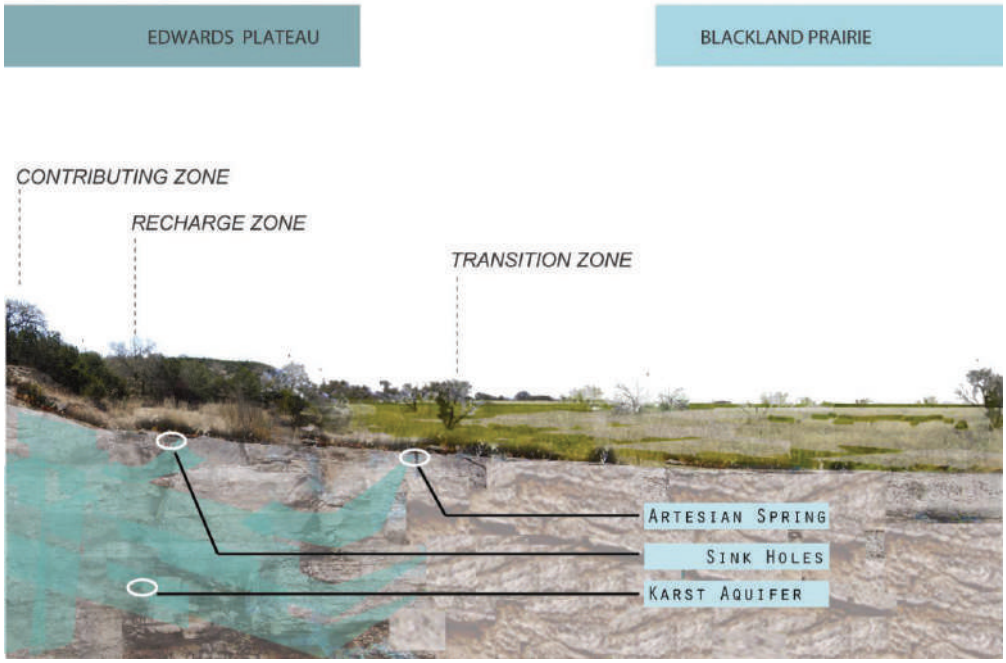


FIGURE B.2.5 TYPOLOGY OF EDWARDS PLATEAU AND BLACKLAND PRAIRIE REGIONS (ADAPTED FROM AND COURTESY OF CALPOLY STUDIO 606)

Blackland Prairie Landscape Region.The Blackland Prairie is a part of the Great Plains that once ran from the Waco area along what is now the Interstate 35 corridor to the Red River (Diamond and Smeins 1993). The gently rolling plains of this region possess soils that are mostly fine-textured, dark, calcareous, and productive clays. Historically, the dominant vegetation would have been almost exclusively tallgrass species such as little bluestem, big bluestem, yellow Indiangrass, and tall dropseed with trees and other woody species primarily limited to low lying wet areas and creek and river bottoms. In lowlands and mesic zones, dominant grasses were eastern gamagrass and switchgrass. Stream bottoms were wooded with bur oak, Shumard oak, sugar hackberry, elm, ash, eastern cottonwood, and pecan (Griffith, G.E., et. al., 2004).

It is part of the true prairie ecosystem and as such is dominated by perennial tallgrass species and, in healthy systems, almost entirely lacking trees except along riparian corridors (Riskind and Collins 1975). Researchers have documented at least six distinct vegetative communities making up the Blackland Prairie (Collins et al. 1975). Currently, most of the prairie has been converted to cropland, non-native pasture, and increasingly urban and industrial uses around the larger metropolitan regions such as Dallas, Waco, Austin, and San Antonio. Of the original 15 million acres, less than one percent of the original vegetation still exists and what remains is dispersed throughout the region in small parcels.

Edwards Plateau Landscape Region The Edwards Plateau eco-region is largely a dissected limestone plateau that is hillier to the south and east where it is easily distinguished from bordering ecological regions by a sharp fault line, shown in Figure B.2.5. Because of the karst topography and underground drainage, the region possesses a network of clear, cool perennial streams. The shallow rocky soils enable both grasses and tree species to become established, forming a region dominated by juniper oak savanna and mesquite-oak savanna, both of which would have had sideoats grama, little bluestem, and muhly grasses commonly in the herbaceous understory. The region was developed through ranching due to the difficulty of the land to plow and is predominantly used for grazing cattle, sheep, goats, exotic game animals, and native wildlife (Griffith, G.E., et. al., 2004). Today, much of the Edwards Plateau is dominated by poor quality forbs and grasses with juniper woodland being the dominant plant habitat of the region (TPWD).

B.2.1.3.3: Step Three: Protect Natural Features and Key Hydrologic Areas

Utilizing hydrology to develop a site plan is an integral part to creating an LID-focused design. Preserving the hydrology of a site includes reviewing existing hydrologic flow paths and protecting infiltrating soils. Natural hydrologic functions of the site should be preserved to the maximum extent practicable. Incorporating these techniques into a site design results in a hydrologically functional landscape that can slow runoff rates, protect receiving waters, and reduce the total volume of runoff.

Identify natural areas to be conserved or restored. Site planners and designers should consider how to use existing natural features of the site in an effort to retain natural hydrologic functions and potentially reduce the cost of drainage infrastructure. Identifying natural or sensitive areas is an integral factor in defining the site area for development and placing site needs and features in the context of the overall watershed. It is important to remember that there are a variety of physiographic regions in San Marcos as the city lies within both the Blackland Prairie and Edwards Plateau eco-regions. These regions differ substantially from other nearby eco-regions in geology, soil characteristics, slopes, infiltration rates, and risks of contamination of groundwater from infiltrated runoff (see Appendix G).

To enhance a site's ability to reduce sources of pollutants and runoff, natural areas that can infiltrate stormwater should be identified in the site design process and conserved or restored. These areas can intercept stormwater without engineered controls, thereby reducing the amount of runoff and the size and extent of drainage infrastructure. Such natural features can result in cost savings due to decreased infrastructure costs.

The following are fundamental principles encouraging conservation and restoration of natural areas:

- Minimize site grading and the area of disturbance by isolating areas where construction will occur (See Step 5). Doing so will reduce soil compaction from construction activities. Additionally, reduced disturbance can be accomplished by increasing building density or height.
- When possible, the site should be planned to conform to natural landforms and to replicate the site's natural drainage pattern. Building roads and sidewalks on the existing contour ensures that natural flow paths and hydrology continue to function.
- An essential factor in optimizing a site layout includes conserving natural soils and vegetation, particularly in sensitive areas such as habitats of sensitive species, wetlands, existing trees, hillsides, conservation areas, karst features, and existing water bodies. Such areas can be used as natural features in site planning to avoid or reduce potential effects of development. Wetlands, for example, provide habitat for several sensitive species, and off-site mitigation does not always provide the same type or quality of habitat.
- In areas of disturbance, topsoil can be removed before construction and replaced after the project is completed. When handled carefully, such an approach limits the disturbance to native soils and reduces the need for additional (purchased) topsoil later.
- Impervious areas (e.g., square footage of parking lots, sidewalks, and roofs) should be minimized by designing compact, taller structures; narrower streets; and using underground or underbuilding parking. In the example shown in Figure B.2.6, the natural and sensitive areas that should be considered for protection during development are identified on the site map, riparian areas, high-quality vegetation. In much of the San Marcos area, especially in the Edwards Escarpment eco-region steep slopes should also be protected as required by the SMDC.

The figure B.2.6 shows the same site, but with the natural features displayed. In addition, sites for geotechnical surveys are displayed.

Protect areas of natural hydrologic function. Hydrologic site features should be preserved to the maximum extent practicable. Hydrologic site features that should be protected include riparian areas, floodplains, stream buffers, and wetlands. These elements are vital to the health of an ecosystem. Preserving them on site allows stormwater to infiltrate and be treated naturally.

Preservation of natural features includes techniques to foster the identification and preservation of natural areas that can be used in the protection of water resources. Whether a large contiguous area is set aside as a preservation zone or certain smaller areas have been identified as appropriate for preservation, protecting established vegetation (existing trees, shrubs, grasses, and other flora) can help reduce revegetation requirements, reduce long-term

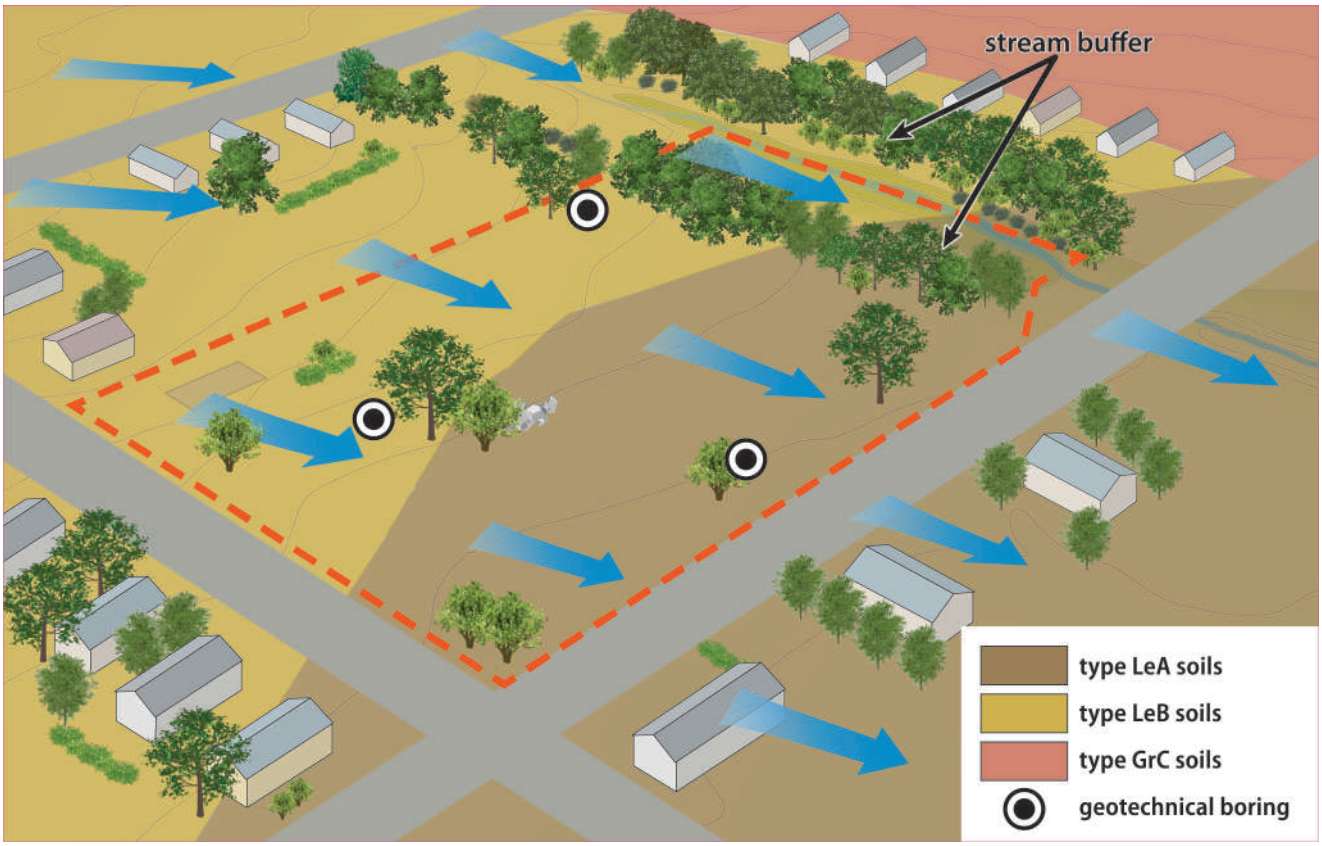


FIGURE B.2.6 PROTECT NATURAL AND SENSITIVE AREAS (WETLANDS, NATIVE TREE GROVES, STEEP HILLSIDE) AND CONDUCT GEOTECHNICAL SURVEY TO CHARACTERIZE INFILTRATION CAPACITY OF SOILS.

erosion, preserve habitat, protect water and land resources, and maintain a healthy ecosystem. Other benefits include:

- An immediate finished "aesthetic" that does not require time to establish;
- Increased stormwater infiltration due to the ability of mature vegetation to process higher quantities of storm water runoff than newly seeded areas;
- Reduced runoff velocity, quantity, and erosion rates (by intercepting rainfall, promoting infiltration, and lowering the water table through transpiration among others);
- Provides a buffer against noise and visual disturbance during construction;
- Provides fully developed habitat for wildlife;
- Reduced construction costs; and
- Usually requires less maintenance (e.g., irrigation, fertilizer) and land clearing labor and costs than planting new vegetation.

In order to reach these benefits, it is important to first identify and preserve sensitive areas that affect hydrology. A site assessment is the process whereby the design team conducts an in-depth evaluation of the overall environmental conditions of the proposed development or redevelopment prior to detailed site design. Natural conservation areas are typically identified using mapping and field reconnaissance assessments. Areas proposed for protection should be delineated early in the planning stage, long before any site design, clearing or construction begins.

Floodplains

Development in floodplain areas can reduce the ability of the floodplain to convey stormwater, potentially causing safety problems or significant damage to the site in question, as well as to both upstream and downstream properties. Ideally, the entire 100-year full build out floodplain should be avoided for clearing or building activities and should be preserved in a natural undisturbed state (See Figure B.2.6). It is understood that this is not possible in all of

San Marcos since the Blanco River has such a large floodplain and development has already occurred in this zone. Where possible, future development should stay out of these and other local floodplains. If this is not feasible, some options include:

- Improving and restoring riparian buffers where needed;
- Restoring and preserving the natural resources and functions of floodplains;
- Diverting high flows around developed areas;
- Acquiring land in a floodplain in order to preserve open space and permanently relocate buildings;
- Treating land to hold as much rain as possible where it falls, so it can infiltrate the soil instead of running off (employing LID techniques);
- Controlling runoff from areas under development outside the floodplain;
- Altering channels to make them more efficient, so overbank flooding will be less frequent; or
- Transferring development rights to areas outside the floodplain (TDR).[6]

However, please note that none of these singular measures alone will prevent floodplain areas from being flooded during extreme storm events. The FEMA National Flood Hazard Layer (NFHL) is a digital database of the latest floodplain risk maps and can be accessed at https://www.fema.gov/national-flood-hazard-layer-nfhl. A viewer is available to check the location of a site relative to the floodplain. The digital data can also be downloaded for use by site designers to confirm the location of the floodplain on a specific site and determine if the floodplain accurately represents the elevations on the site. The designer should verify the risk associated with building near a floodplain and must follow all City of San Marcos floodplain design criteria as defined in Chapter 3, Article 9 of the Development Code.

Once identified, preservation areas should be incorporated into site development plans and clearly marked on all construction and grading plans to ensure that construction activities are kept out of these areas and that native vegetation is kept in an undisturbed state. The boundaries of each conservation area should be mapped by carefully determining the limit which should not be crossed by construction activity.

6 Please visit the Federal Emergency Management Agency for more guidance on flooding issues.

Buffers

A riparian buffer is a special type of natural conservation area along a stream, wetland or shoreline where development is restricted or prohibited. The primary function of buffers is to protect and physically separate these waterbodies from future disturbance or encroachment. If properly designed, a buffer can provide stormwater management functions, can act as a right-of-way during floods, and can sustain the integrity of water resource ecosystems and habitats. The City of San Marcos protects the riparian buffers along streams with water quality and buffer zones. Further information on water quality and buffer zone limits can be found in Section §6.2.2.2 of the SMDC. Figure B.2.7 is an example of how the water quality and buffer zones are aligned with the creek/river centerline.

Riparian buffers should be continuous and not interrupted by impervious areas that would allow stormwater to concentrate and flow into the stream without first flowing through the buffer. Existing forested riparian buffers should be maintained. Where no wooded buffer exists, reforestation should be encouraged. Proper restoration should include all layers of the forest plant community, including trees, understory, shrubs and groundcover. Water quality and buffer zone restrictions can be found in the Development Code §6.2.3.2.

The development of LID features within riparian buffers should be allowed as long as the development does not adversely impact the riparian area. Ideally, all buffers should remain in their natural state. However, some maintenance is periodically necessary, such as:

- Planting to minimize concentrated flow;
- Removal of invasive or exotic plant species when these species are detrimental to the vegetated buffer; and
- Removal of diseased or damaged trees.

A review of the hydrologic features on-site provides many benefits to the site-planning process. In addition to examining the availability of natural features for stormwater quality treatment, evaluating the direction of flow on site is a vital step in the site assessment phase. Determining the downstream points on a site can help the designer determine the final site layout. Doing so allows SCMs to be placed downstream of the impervious area that they need to treat. Laying out the site while knowing the overall hydrologic properties and preserving natural features saves both time and cost for the SCM treatment requirements.

Further information on the buffer zone boundaries can be found in Section §6.2.2.2 of the Development Code.

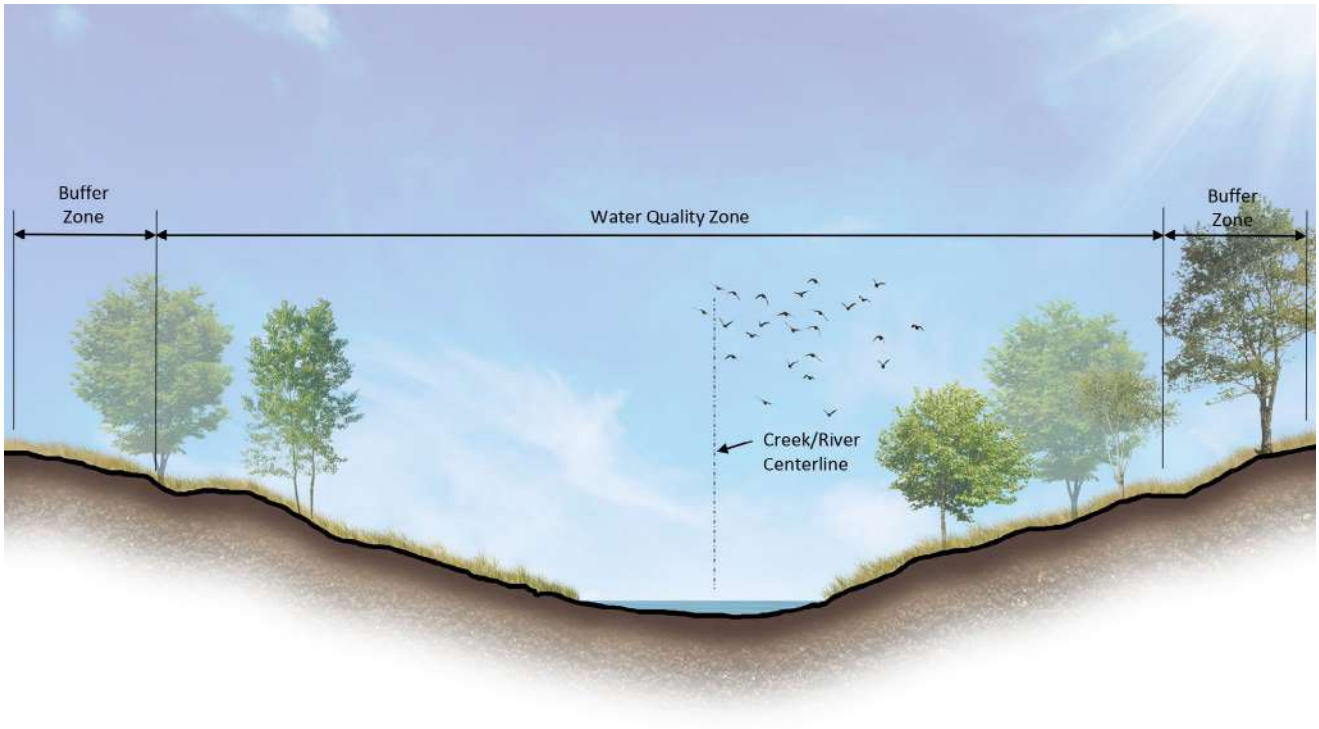


FIGURE B.2.7 EXAMPLE CROSS-SECTION OF A CREEK/RIVER AND THE ASSOCIATED BUFFERS.

Protect possible areas for infiltration. Second only to flow regimes in ensuring proper hydrology, healthy soils or media often serve as essential elements for achieving LID functions and providing source control for stormwater treatment. For example, upper soil layers are conducive to slowly filtering and storing stormwater, allowing unit processes such as infiltration, sorption, evapotranspiration, and surface retention to occur.

LID measures along sections of the San Marcos River, where gravelly sediments deposited by the river have substantial infiltration rates, will prove effective and beneficial. This riparian area allows the maximum performance benefit by both infiltrating all the runoff and eliminating surface discharge. Infiltration rates will vary from site to site, so characterization of the infiltration capacity at the proposed location should be undertaken. Some of these river deposits in the upland extend into the Edwards recharge and contributing zones (generally areas west of Post Road and Hunter Road) and TCEQ rules requiring liners for certain types of facilities will apply.

However, LID measures in the riparian areas do not need to be structural in nature. Often in these riparian areas the water table can be close to the surface. In this case, healthy (and large, where possible) buffers and strips will be very effective LID measures. Further, the use of infiltration-based LID structural controls located higher in the watershed can aid in stream base flow recharge, further helping the success of these riparian areas.

Identify areas with steep slopes. Development on slopes with a grade of 15% (7:1) or greater should be avoided if possible to limit soil loss, erosion, excessive stormwater runoff, and the degradation of surface water. Excessive grading should be avoided on all slopes, as should the flattening of hills and ridges. Steep slopes should be kept in an undisturbed natural condition to help stabilize hillsides and soils. On slopes greater than 25% (4:1), no development, regrading, or stripping of vegetation should be considered. When developing a new site, where possible, it is beneficial to work with the site's topography and natural drainage ways.

Figure B.2.8 below details areas of the site that should be conserved. Using the information collected in the Step 2 soil evaluation, more specific locations of soils with greater infiltration rates that are near or on hydrologic flow paths should be protected to avoid or limit hydrologic impacts. As an example, Figure B.2.8 indicates the key hydrologic areas that should be considered for protection. The blue, area identified as an area for possible infiltration, should be separated from other site features by surrounding it with construction fencing to prevent access and avoid compaction. In addition, the areas having a natural hydrologic function either through storage or conveyance should be protected. (also see Figure B.2.10 in setting site clearing and grading limits).

With the conclusion of Phase I, the initial site assessment has been completed. The decisions made regarding LID and SCM practices during the site assessment process should be documented to ensure that if changes are required in future Phases II and III, the original design ideas are available for reference. That helps ensure that LID concepts and SCMs are considered during every component of project site planning. Phase II of site planning, described below, results in a preliminary design plan.

B.2.1.2: Phase II – Preliminary Design

At the conclusion of Phase II of the site design, a preliminary site design will be completed. Following the steps outlined below will yield a site plan that incorporates many features of low impact development and includes best management practices. The steps are often iterative, and may need to be repeated several times for an optimal result.

B.2.1.1.1: Step Four: Use Drainage and Hydrology as Design Elements

Delineate drainage areas throughout the site. Drainage areas are defined by geographical terrain and man-made drainage systems. Drainage area delineations are critical to determining how much stormwater reaches a point and therefore how to size a stormwater

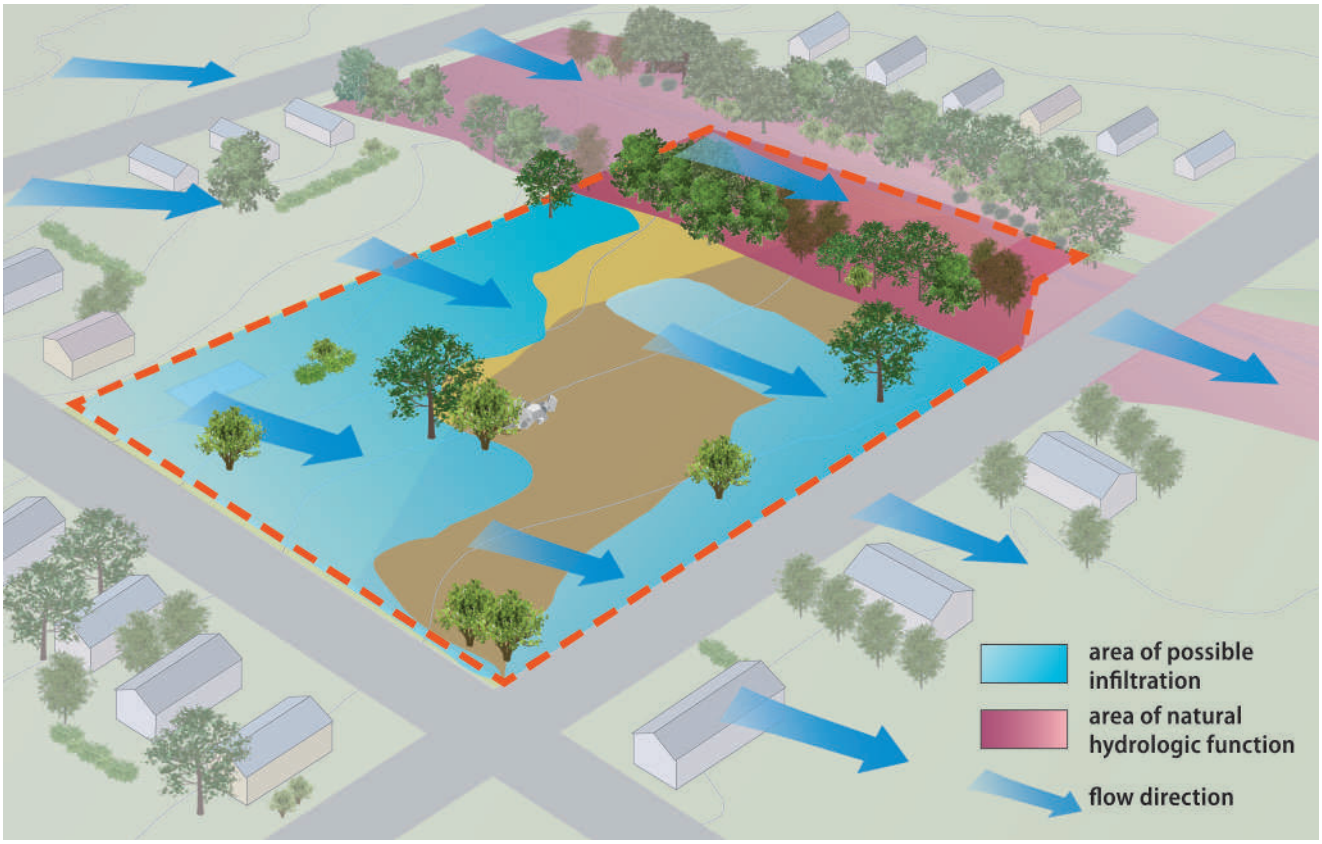


FIGURE B.2.8 IDENTIFY AND PROTECT KEY HYDROLOGIC AREAS, SUCH AS INFILTRATING SOILS (BLUE AREA) AND RIPARIAN BUFFER (PURPLE AREA)

control measure. Data utilized to delineate drainage areas can come from multiple sources, including:

- High accuracy LiDAR based contours, as generated by public or private agencies;
- On-the-ground topographic survey data;
- Historical topography maps, including USGS Quad. Maps for pre-development conditions;
- Roadway construction plans;
- Aerial photos;
- Underground infrastructure plans;
- As-built plans for a site.

All basin delineations should consider previously defined drainage basins as found by the regions data sets or previously approved drainage studies. Errors that are found during this investigation should be corrected.

Basin delineations defined by computer software should be reviewed carefully. Software including AutoCAD, Microstation, ESRI-GIS and others have the capability to define basins. These basins are created by source data such as a Digital Elevation Model (DEM), a Triangular Irregular Network (TIN) or Raster grid files. The data set should be detailed enough to define the basin; it may require the use of break lines or fault lines to create certain features. Generally when DEM or Raster is used to generate basin delineation the resulting basin will create a jagged or zigzagged basin boundary. The design engineer should verify that this resulting basin has the correct level of accuracy for the individual study.

Identify the spatial layout of the site using hydrologic flow paths as a feature. Natural hydrologic functions (e.g., flow paths) should be included as a fundamental component of the preliminary design. Naturally present functions should be retained, or if that is not an option, replicate natural functions with appropriate SCM placement. See Step Three for further details.

Determine approximate conveyance and SCM locations. Natural hydrologic functions, including interception, depression storage, and infiltration, should be distributed throughout the site to the extent possible. In conserving predevelopment and retrofit hydrology, runoff volume, peak runoff rate, flow frequency and duration, and water quality control must be considered. Rainfall abstractions are the physical processes of interception, evaporation, transpiration, infiltration, and storage of precipitation.

Runoff flow frequency and duration should try to mimic predevelopment conditions by implementing practices to minimize runoff volume and rate. Both SCM and LID measures also provide pollutant removal processes that enhance water quality treatment for the designed treatment volume.

By setting the development footprint back from natural drainage features, the drainage can retain its hydrologic functions and its water quality benefit to the watershed as shown in the example in Figure B.2.8, assuming that runoff from the contributing watershed is mitigated to predevelopment conditions.

Spatial layout should use the natural landforms and hydrologic flow paths identified in Step 2 as a major design element of the site. Common LID elements using that premise include designing open drainage systems to function as both treatment and conveyance devices. Impervious elements such as parking lots, roadways, and sidewalks can be designed on the existing contours to minimize effects on the natural hydrologic flow path.

Selecting the appropriate structural SCMs for a project area should be on the basis of site-specific conditions (e.g., land availability, slope, soil characteristics, climate condition, and utilities) and stormwater control targets (e.g., peak discharge, runoff volume, or water quality targets). In the example shown in Figure B.2.9, areas are identified that will be developed for residential lots, parking, and sidewalk footprints. The figure also indicates ideal locations where LID SCMs can be placed. In this instance, the courtyard in between the lots makes an ideal location for bioretention facilities. The SCMs can be incorporated into the natural drainage paths to function as conveyance and treatment LID SCMs. A geotechnical investigation indicates that the hydrologic soil group is Group B, indicating that infiltration SCMs are allowed on the site without an underdrain. Note that bioretention facilities can also meet landscaping requirements and create features that enhance and beautify the site.

Information on designing each of these types of SCMs can be found in Sections 3 and 4.

B.2.1.2.2: Step Five: Establish Clearing and Grading Limits

Clearing and grading should be kept to the minimum amount required for construction in order to minimize total impacts to the site.

Define the limits of clearing and grading on the site plan. Limits of clearing and grading refer to the total site area that is to be developed, including all impervious and pervious areas. The area

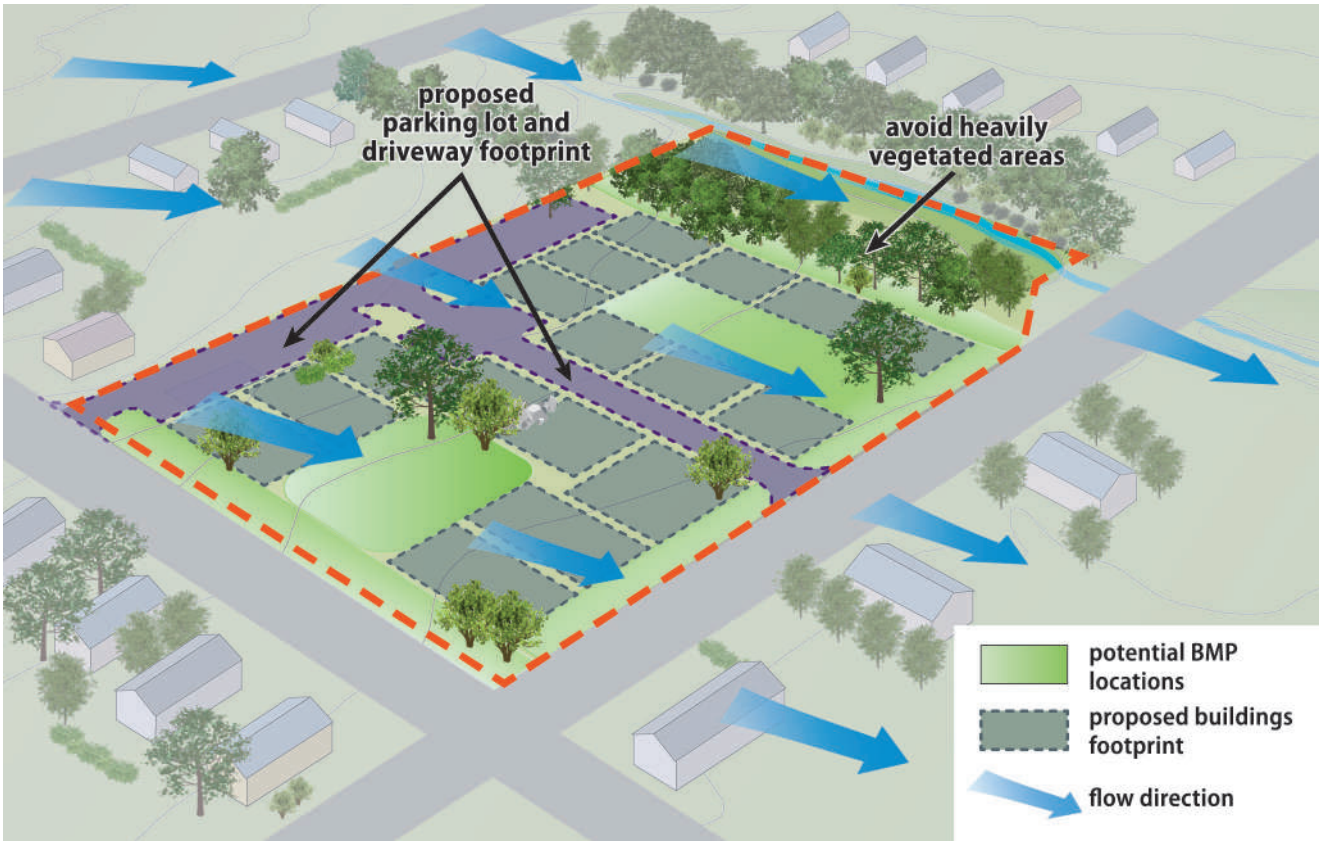


FIGURE B.2.9 IDENTIFY IDEAL LOCATIONS FOR LID IMPLEMENTATION ACCORDING TO SITE CONDITIONS.

of development ideally should be in less sensitive locations with respect to hydrologic function and should be outside protected areas and areas containing setback regulations, easements, and utilities.

Minimize disturbance to areas outside the limits of clearing and grading. Disturbance to areas that are designated as those outside clearing and grading limits should be minimized. Any disturbance across a site should be treated with erosion and sediment control devices. Reducing the overall disturbance helps to preserve natural features and diminish the cost of designing and constructing a site.

Once a site is under construction, minimal disturbance methods should be used to limit the amount of clearing and grading that takes place on a development site, preserving the undisturbed vegetation and natural hydrology of a site. A limit of disturbance (LOD) should be established based on the maximum disturbance zone. These maximum distances should reflect reasonable construction techniques and equipment needs together with the physical situation of the development site such as slopes or soils. LOD distances may vary by type of development, size of lot or site, and by the specific development feature involved.

Not only should these natural conservation areas be protected during construction, but they should also be managed after occupancy by a responsible party able to maintain the areas in a natural state in perpetuity. Typically, conservation areas are protected by legally enforceable deed restrictions, conservation easements, and a maintenance agreement.

Site fingerprinting refers to site clearing and development with minimal disturbance of existing vegetation and soils. Such techniques include reducing paving and compaction of highly permeable soils, minimizing the size of construction easements and material storage areas, site clearing and grading to avoid tree removal, delineating and flagging the smallest site disturbance area possible, and maintaining existing topography to the extent possible. Figure B.2.10 illustrates the use of orange construction fencing to preserve the natural features, drainage pathways, and maintain infiltration on suitable soils at the example site as identified in previous steps.

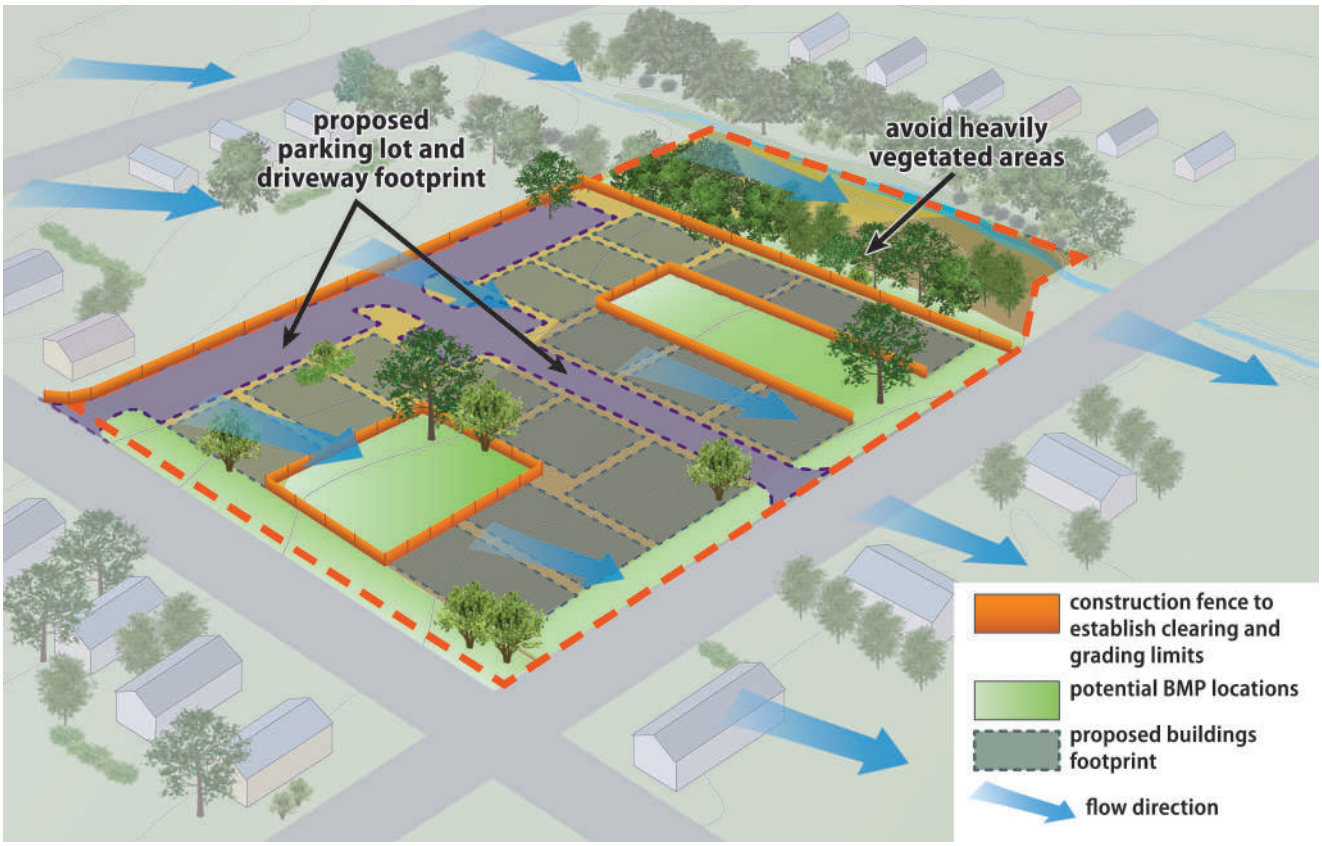


FIGURE B.2.10 ESTABLISH GRADING ENVELOPE TO PROTECT NATURAL AREAS AND INFILTRATING SOILS.

B.2.1.3.3: Step Six: Reduce/Minimize Impervious Cover

Rainfall that does not infiltrate or pool where it falls results in runoff. As the imperviousness of the site increases, runoff also increases. For each acre of impervious cover and every inch of rainfall, approximately 27,150 gallons of stormwater are generated. Predevelopment runoff, measured as a runoff coefficient or the ratio of runoff volume to the total amount of rainfall, can be maintained by compensating for increases in impervious areas, soil compaction, and the loss of abstraction through planning and design. Such tools can be used to also manage the peak runoff rate and volume and protect water quality.

Evaluate conceptual design to reduce impervious surfaces. One of the most effective LID measures is the reduction of overall impervious surfaces. Reducing the construction of impervious surfaces reduces the volume of stormwater runoff created. Minimizing runoff at a site reduces the size of SCMs required. Therefore, reducing the overall surface area for impervious areas is one of the most cost-effective methods of reducing surface runoff and required treatment.

Street layouts often can be designed to reduce the extent of paved areas, and street widths can be narrowed to decrease the total impervious area as long as applicable street design criteria are satisfied. Eliminating curbs and gutters along streets and including curb cuts around parking areas, where consistent with city standards and where appropriate, can promote drainage to on-site pervious areas and decrease directly connected area considerably. Other options include replacing curbs and gutters with roadside vegetated swales and directing runoff from the paved street or parking areas to adjacent LID facilities. Such an approach for alternative design can reduce the overall capital cost of the site development while addressing stormwater quantity and quality issues and improving the site's aesthetic values.

Investigate potential for impervious area disconnection. Diverting stormwater runoff from impervious areas such as rooftops and pavement to adjacent pervious areas can be used to infiltrate stormwater runoff and to reduce flow rates (shown in Figure B.2.11). Proper design can align pervious surfaces with building drainage. Such a technique is also referred to as impervious area disconnection.



FIGURE B.2.11 DISCONNECTION AT PORTLAND STATE UNIVERSITY, OREGON, WHERE VEGETATED AREAS ARE DISTRIBUTED THROUGHOUT THE IMPERVIOUS COURTYARD AREA.



FIGURE B.2.12 DOWNSPOUT DIRECTED TO A RAIN GARDEN AFTER PASSING OVER A STONE SPLASH PAD.

Disconnection of impervious surfaces and downspouts can help maximize the function of the LID measures. Disconnection is a low-cost, effective non-structural control which can reduce total runoff volume, increase the time of concentration and promote infiltration. The first step in disconnection is to identify the source of runoff and understand how it will be managed once disconnection occurs. In addition, well-conceived use of disconnection methods can reduce overall project costs by reducing or eliminating the need for more expensive structural practices.

In addition, the amount of runoff and associated pollutants from a project can be reduced by disconnecting impervious surfaces. By disconnecting impervious areas and directing the flow to infiltration basins or designated buffer areas, a portion of additional runoff that would contribute to stormwater runoff is infiltrated close to the source instead. Further, the runoff that would potentially carry pollutants from the site to surface water instead gets treated and helps recharge groundwater.

Disconnection methods should be incorporated at the planning and design level. However, the designer and reviewer should note that these methods must be used in unison with the design of other stormwater conveyance and treatment practices. The use of these disconnection methods does not relieve the designer or reviewer from following the standard engineering practices associated with safe conveyance of stormwater runoff and good drainage design.

Downspout Disconnection

Rooftops with exterior drains for the gutter (the normal configuration for most residential structures) is one of the easiest disconnection practices to implement. These downspouts should be directed to landscaped portions of the site rather than driveways or sidewalks unless the driveway is constructed of pervious paving materials as shown in Figure B.2.13. It is not common, but driveways can be crowned so that a portion of the runoff is directed to vegetated areas, rather than the street.

In addition to directing downspouts to vegetated areas, roof runoff may also be directed to cisterns and other rain barrels for later consumption, or even to depressed storage or other underground storage areas. Further, this runoff may be directed through a treatment train system as described below and demonstrated in Figure B.2.14. Some design considerations include:

- Slowing down the water after it leaves the downspout if the volume and velocity is high (as shown with a splash pad configuration in Figure B.2.12;
- Keeping the disconnected runoff away (10' minimum) from other impervious surfaces to reduce the chance for re-connection;
- Not placing the disconnected runoff into a steep slope area which could cause erosion and concentrate flows; and



FIGURE B.2.13 DOWNSPOUTS DIRECTED TO PERMEABLE PAVEMENT ON DRIVEWAY (COURTESY OF MONTGOMERY COUNTY, MD).

- Directing the runoff into features specifically designed to receive (and either store, soak, treat, or convey) this runoff.

In Figure B.2.12, this downspout disconnection is part of a larger treatment train system, as disconnection techniques often are. Once the rooftop runoff is directed into the splash pad area, it flows off the side into a bioretention system as pictured in Figure B.2.14 below. Excess water not infiltrated or evaporated enters the overflow grate shown in the bottom of the middle image. From here, the runoff is carried away from the building underneath the sidewalk and into a second treatment area, a larger bioretention system adjacent to the parking lot, shown at right.

To reduce the storage and conveyance requirements, the directly connected impervious area of the site should be minimized to the extent practicable. That can be accomplished by increasing the building density by increasing the vertical extent and minimizing the horizontal extent. Impervious area disconnect can also include using permeable features instead of impermeable including permeable pavement for walkways, trails, patios, parking lots, and alleys; and constructing streets, sidewalks, and parking lot aisles to the minimum width necessary. Information regarding the design of permeable pavement can be found in Section B.4.

Figure B.2.15 illustrates the inclusion of pervious areas and locations of potential impervious area disconnect to demonstrate locations where that can be achieved.

In Figure B.2.15 opportunities for imperviousness reduction and runoff disconnection were identified for the site. The sidewalk surrounding the lots was disconnected by routing runoff to the pervious landscaped areas surrounding the lots (shown in yellow).

Determine if conservation planning area design is a viable option. Per Section 4.4.3.7 of the San Marcos Development Code, developers in San Marcos can establish a Planning Area District in order to assign a conservation character to a development project rather than using conventional subdivision design. Contact the Planning Department or go to http://sanmarcostx.gov/DocumentCenter/View/3202 for more information about where these Districts are allowed.



FIGURE B.2.14 TREATMENT TRAIN APPROACH: DISCONNECTED DOWNSPOUT RUNOFF ENTERS A SERIES OF VEGETATED CONTROLS.

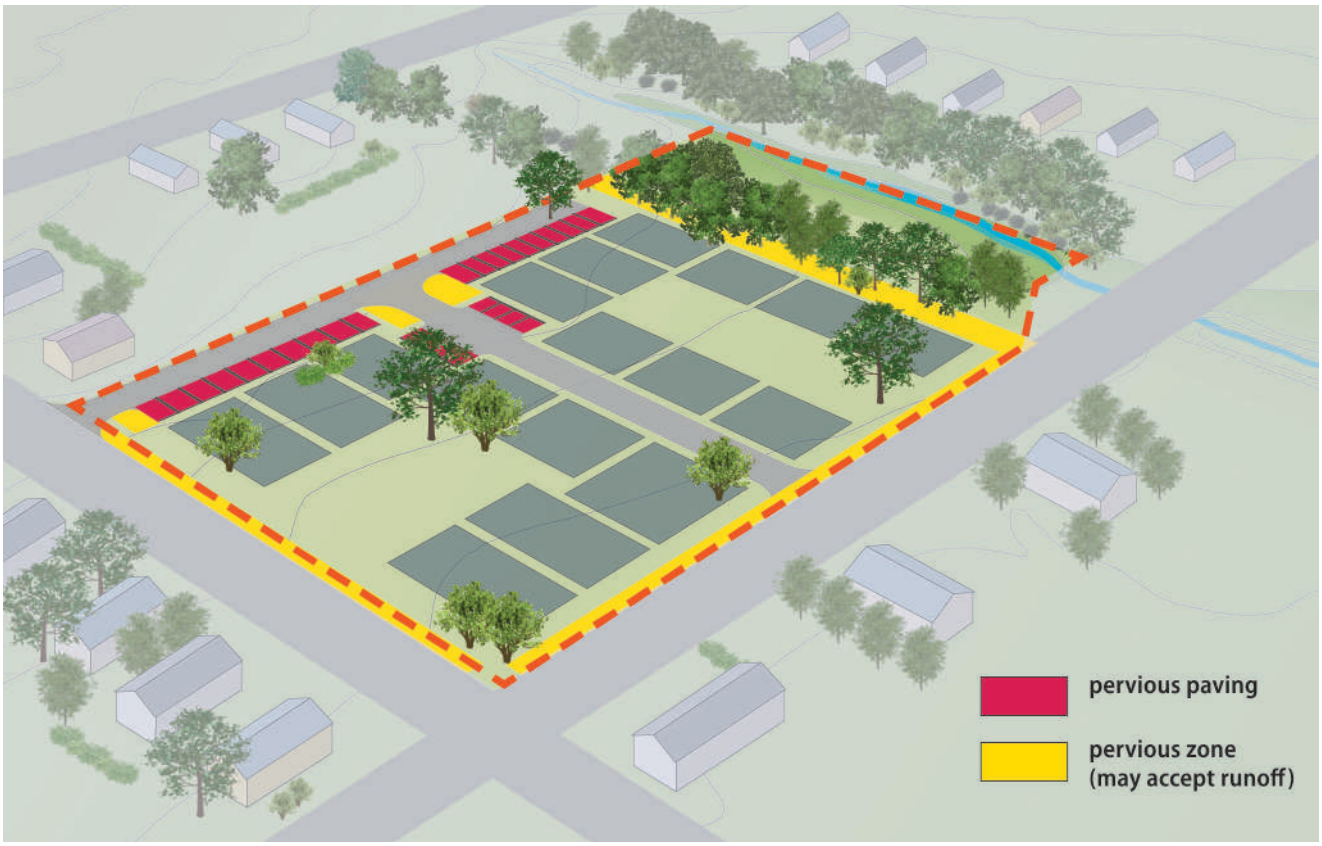


FIGURE B.2.15 SITE EXAMPLE DEMONSTRATING PLACEMENT OF PERVIOUS MATERIAL (RED) AND OPPORTUNITIES TO MINIMIZE CONNECTED IMPERVIOUS AREA.

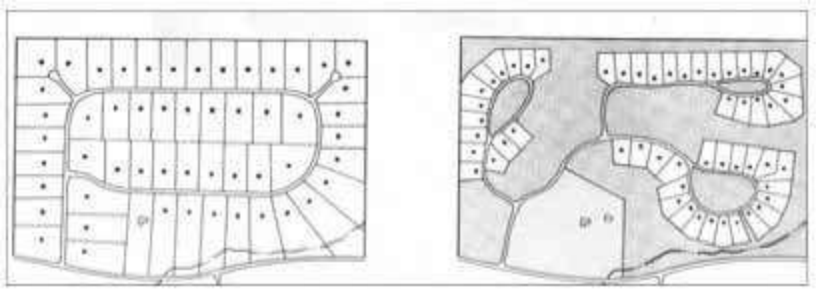


FIGURE B.2.16 CONVENTIONAL DESIGN (LEFT) AND CONSERVATION DESIGN (RIGHT) (COURTESY TOWN OF PINE PLAINS, NY).

Conventional subdivisions have evolved over the past 50 years and generally involve larger lot development, clearing and grading of significant portions of a site, wider streets and larger cul-de-sacs, enclosed drainage systems for stormwater conveyance, and large "hole-in-the-ground" detention basins (see left image of Figure B.2.16).

The intention of Conservation Planning Area, also known as open space design or cluster development, is to preserve large areas of environmentally sensitive or prime agricultural lands while providing for clustered residential development in appropriate areas. The planning approach includes laying out the elements of a development project in such a way that the site design takes advantage of a site's natural features, preserves the more sensitive areas, and identifies

any site constraints and opportunities to prevent or reduce impacts. Techniques include:

- Preservation of undisturbed areas, stream buffers, farmland, native ecosystems;
- Reduction in clearing and grading;
- Working with the site's topography and natural drainage ways;
- Locating projects in less sensitive areas; and
- Clustering development.

Natural conservation areas are typically identified through a site assessment. Depending on the site, an assessment can be performed by professionals on the project development team (engineers, landscape architects or planners for example); however, to fully examine a site and its ecological conditions which will influence SCM design, more in-depth site analysis should be done by hydrologists, ecologists, biologists or others professionals with site assessment experience in order to test infiltration rates, asses soil type and quality, and be able to properly identify existing vegetation. In many cases, a geotechnical report may also be required to assess depth to groundwater among other factors. When done before the concept plan phase, the planned conservation areas, and identification of other sensitive features also outlined above, can then be used to guide the layout of a project. For more guidance on conducting a site assessment, visit the Sustainable Sites Initiative™ guidelines[7].

Conservation Planning Areas typically incorporate smaller lot sizes to reduce overall impervious cover while providing more undisturbed open space and protection of water resources. This approach concentrates structures and impervious surfaces in a compact area in one portion of the development site in exchange for providing open space and natural areas elsewhere on the site. Typically smaller lots and/or nontraditional lot designs are used to cluster development and create more conservation areas on the site. Conservation developments have many benefits compared with conventional commercial developments or residential subdivisions. They can reduce:

- Impervious cover;
- Stormwater pollution:
- Construction costs; and
- The need for grading and landscaping, while providing for the conservation of natural areas.

Along with reduced imperviousness, which carries multiple ancillary benefits as mentioned above, conservation designs provide a host of other environmental benefits lacking in most conventional designs. They can:

- Prevent encroachment on conservation and buffer areas;
- Create community-wide interconnected network of protected meadows, fields and woodlands, and provide opportunities for trail and recreation connection; and
- Help to provide habitat, and protect farmland and other natural resources while allowing for the maximum number of residences under current community zoning.

As less land is cleared during the construction process, alteration of the natural hydrology and the potential for soil erosion are also greatly diminished. Perhaps most importantly, open space design reserves 25 to 50% of the development site in conservation areas that would not otherwise be protected.

Conservation developments can also be significantly less expensive to build than conventional projects. Most of the cost savings are due to reduced infrastructure cost for roads and stormwater management controls and conveyances. Further, developers find that these properties often command higher prices than those in more conventional developments. Several studies estimate that residential properties in open space developments garner premiums that are higher than conventional subdivisions and moreover, sell or lease at increased rates.

Once established, common open space and natural conservation areas must be managed by a responsible party able to maintain the areas in a natural state in perpetuity. Typically, the conservation areas are protected by legally enforceable deed restrictions, conservation easements, and maintenance agreements.

Preservation of natural areas and conservation designs can help to preserve predevelopment hydrology of the site and aid in reducing stormwater runoff and pollutant load. Undisturbed vegetated areas also promote soil stabilization and provide for filtering and infiltration of runoff. Maintaining existing vegetation can be particularly beneficial to sites with floodplains, wetlands, stream banks, steep slopes, critical environmental features, or where erosion controls are difficult to establish, install, or maintain.

7 www.sustainablesites.org

B.2.1.4.4: Step Seven: Determine Appropriate SCMs.

Stormwater control measures employ a number of processes: settling/sedimentation, filtration, sorption, photolysis, biological processes, and chemical processes for pollutant removal. In addition to pollutant removal, LID SCMs provide hydrologic controls by reducing peak flows and volume through processes of infiltration, evaporation, and storage and reproducing predevelopment hydrologic functions. Traditional SCMs primarily provide hydrologic controls via detention and retention of water.

Evaluating which LID SCM (or SCMs) are required on a site involves determining what the end goals and requirements are. There are several SCMs (such as bioretention facilities) that qualify as volume type BMPs and can be utilized to meet water quality volume and treatment standards. Other BMPs (such as vegetated filter strips and vegetated swales) help to improve water quality, but do not meet the City of San Marcos' volume requirements. These SCMs are typically used in tandem with volume-based SCMs to meet the total treatment requirements.

Determine which SCMs should be used to ensure multiple benefits are achieved. During SCM selection, it is important to consider the unit processes of a SCM to ensure that the management practice will provide the necessary benefits and avoid potential complications or failures.

Hydrologic controls dictate how incoming stormwater is partitioned into the various components of the hydrologic budget. Stormwater volume can be detained, infiltrated, evapotranspired, drained, or bypassed depending on the design of hydrologic controls and features such as impermeable liners, underdrains, inlet and outlet structures, soil media permeability, and storage capacity.

Settling/sedimentation is the physical process of particle separation as a result of a difference in density between the solids and water. Most SCMs use settling to some degree, especially through detention or retention practices such as bioretention facilities and wet ponds. Settling is enhanced by slowing down or spreading out runoff to create low velocity flow conditions.

Filtration is the physical process of separating solids from a liquid media; particles are filtered from water by the smaller interstitial space the water flows through in the porous medium. Sedimentation and sorption can also occur as water passes through a filtering practice. *Sorption* refers to the processes of *absorption* (an incorporation of a pollutant into a substance of a different state) and *adsorption* (the adherence of a pollutant to the surface of another molecule). Sorption is also referred to under chemical treatment processes. Filtration is a common unit process in a number of SCMs such as bioretention facilities and planter boxes.

Floatation is a treatment unit process where the mechanism for pollutant removal is opposite to that in settling and sedimentation. In floatation, the density of pollutants, such as trash and petroleum, is less than that of water. Oil/water separators and trash guards are the primary SCM practices that use floatation.

Biological treatment processes (bioaccumulation, biotransformation, phytoremediation) are processes that occur in practices that incorporate soils and plants for pollutant removal via biological transformation or mineralization, pollutant uptake and storage, or microbial transformation. It can also include organisms that consume bacteria. SCMs that can be designed to use such unit processes are bioretention, bioswales, and planter boxes.

Chemical treatment processes include sorption, coagulation/flocculation, and disinfection. Chemical characteristics of stormwater such as pH, alkalinity, and reduction-oxidation (redox) potential, determine which chemical process is appropriate. Sorptive SCMs generally include engineered media for removing pollutants of concern. Precipitation and disinfection processes require actively adding chemicals to encourage coagulation/flocculation and precipitation or chemicals such as chlorine to mitigate pathogenic microbes in stormwater. Chemical treatment processes are usually employed as end of pipe solutions where no other SCM can effectively treat an existing storm drain system. In these cases, low flow may be more effectively treated by pumping into a sanitary sewer.

Using multiple treatment processes either in individual or multiple SCMs is called a *treatment train*. Meeting targeted treatment objectives can usually be achieved using a series of LID SCMs in a treatment train. Treatment trains can often be designed along rights-of-way, in parking lots, underground, or incorporated into landscaped areas. LID site planning should result in a treatment train of LID strategies and SCMs to meet treatment and water quality goals. For further details on treatment train SCM implementation, see Section B.7 of this appendix.

A number of factors should be considered for choosing appropriate SCMs for a site. For example, the presence of group C or D soils on a site might preclude the use of an infiltration SCM or require the use of an underdrain into the design of infiltration SCMs described in this Appendix. Additionally, the low level of precipitation and high

evapotranspiration rates usually present in San Marcos would likely exclude the use of a SCM requiring a permanent pool, such as a stormwater wetland, because precipitation is not great enough to maintain a continual or permanent pool of water. Native vegetation, which is adapted to the local climate and soils, should be used for vegetated SCMs, as much as possible, when soils allow. If native soils are replaced with imported soils to improve infiltration, non-native noninvasive but drought-tolerant plants might be a desired choice. For information regarding appropriate vegetation, see the *SARB Low Impact Development Manual* (link provided in Appendix A.). Other geotechnical, site-specific considerations include the level of the underlying water table and bedrock, any existing infrastructure in retrofit designs, and the presence of areas of concern that exhibit soil and ground water contamination.

Account for contaminated soils on brownfields sites. Preparing brownfields for redevelopment often requires capping of contaminated soils, creating even larger impervious surfaces. The challenge for managing stormwater on brownfield sites is allowing this capping while mitigating the impervious surface conditions that can negatively impact local waterways. Unlike many conventional developments, impervious footprints on brownfields cannot always be minimized through site designs that incorporate more porous surfaces to allow for infiltration. However, LID measures exist that can retain, treat and then release stormwater without it ever coming in contact with contaminated soils.

The first practice is to consider site location within the watershed. New and redeveloped sites near brownfields should use LID measures to prevent additional runoff from flowing onto potentially contaminated areas. By retaining stormwater on site, these sites are preventing runoff from coming in contact with nearby contaminated soils.

The second practice is consideration for the treatment and storage of stormwater, without an infiltration component. Direct infiltration on a brownfield site may introduce additional pollutant loads to groundwater and nearby surface waters. On brownfields that have caps, vegetated areas need to be located above caps and fitted with underdrain systems to direct overflow stormwater to surface drainage. Buildings and other impervious surfaces can be strategically located to act as caps over areas with known contamination. Areas with fill caps can include soils and vegetation above the cap in the form of swales or rain gardens. If fitted with a liner or an underdrain system to prevent infiltration and to release treated stormwater off site, these planted areas can safely allow filtration and evapotranspiration of stormwater. Additional features like impermeable liners or gravel filter blankets can be coupled with modified LID measures that safely filter stormwater without exposing the water to contaminated soils.

Ensure infiltration is done properly within Edwards Aquifer Zones. The karst limestone formations of the contributing and recharge zones are overlain by thin soils. In these areas currently, direct infiltration of stormwater is prohibited per the Texas Commission on Environmental Quality's (TCEQ) Edwards Rules. Thus, bioretention and porous pavement facilities must be designed with underdrains. However, the stormwater passing through these systems will still have water quality and peak flow reduction benefits, even though there will be little to no benefit to groundwater recharge. Controls such as vegetated buffer strips and swales will function effectively and no special design or maintenance efforts will be required in these areas. In areas of steep slopes, LID measures will be difficult to implement but options include terracing of bioretention systems, using rock berms, or other similar treatments. For instance, a series of level bioretention systems down a slope will slow water down and allow it to pond temporarily behind the rock berm before continuing down slope to the next treatment area. In these cases, consideration should be given to the volume and velocity of the flows to assess whether filter fabric or splash pads are necessary.

In the transition and artesian zones, infiltration is allowed and there are currently no regulations prohibiting infiltration. Infiltration rates will be lower, which will influence BMP sizing, and in some cases might call for an underdrain. Plant selection is essential in these areas as certain native grasses can help improve infiltration with their deeper root system. Some sites may require modification of the native soils to increase infiltration rates but the BMP will always be limited by the underlying substrate. In other sites, an underdrain may be necessary under bioretention and porous pavement facilities. Ultimately, BMP designs in these areas will depend on the performance goals for the site. Vegetated buffer strips and swales will function effectively and no special design or maintenance efforts will be required in the Artesian and Transition zones.

Integrating SCMs into downtown in a way allowed by SmartCode. Incorporating LID into San Marcos' SmartCode transects can help improve water quality, offset the negative impacts of development, and protect the existing natural features such as rivers and creeks present in *all* transect zones. The table on the following page provides a menu of LID SCMs appropriate for each transect. From this diagram it is clear, for example, that Vegetated Filter Strips will

Transects	LID Options & Considerations
T-1 Natural Zone T-1 Natural Zone consists of lands approximating or reverting to a wilderness condition, including lands unsuitable for settlement due to topography, hydrology or vegetation.	**BMPs:** All non-structural and structural BMPs. **Cost:** Low to none if non-structural; low if structural. **Maintenance:** Low to none **Notes:** T1 is an ideal place for many of the non-structural LID practices. Of course, in areas with no development or building, green roofs will not be appropriate.
T-2 Rural Zone T-2 Rural Zone consists of sparsely settled lands in open or cultivated states. These include woodland, agricultural land, grassland, and hill country. Typical buildings are farmhouses, agricultural buildings, cabins, and villas.	**BMPs:** All non-structural and structural BMPs. **Cost:** Low to none if non-structural; low if structural. T-2's rural character may not require added infrastructure such as curbs, and may allow for larger systems to be in place to account for lower infiltration rates in native soils. These two options help reduce overall cost. **Maintenance:** Low **Notes:**
T-3 Sub-Urban Zone T-3 Sub-Urban Zone consists of low density walkable residential areas, adjacent to higher zones that have some mixed use. Home occupations and outbuildings are allowed. Planting is naturalistic and setbacks are relatively deep. Blocks may be large and Thoroughfares irregular to accommodate natural conditions, but designed for slow movement.	**BMPs:** All non-structural and structural BMPs. **Cost:** Low **Maintenance:** Low, similar to general landscaping maintenance in suburban zones. **Notes:**
T-4 General Urban Zone T-4 General Urban Zone consists of a mixed use but primarily residential urban fabric. It may have a wide range of building types: single, side yard, and rowhouses. Setbacks and landscaping are variable. Streets with curbs and sidewalks define medium-sized blocks.	**BMPs:** Bioretention & Rain Gardens; Green Roofs; Porous Pavement; Rainwater Harvesting; Proprietary Systems. **Cost:** Low to medium, depending on design and site constraints. **Maintenance:** Low, similar to general landscaping maintenance in denser residential and mixed-use zones. **Notes:** Swales may be feasible in certain areas but ONLY if in **a treatment train.**
T-5 Urban Center Zone T-5 Urban Center Zone consists of higher density mixed use building that accommodate retail, offices, rowhouses and apartments. It has a tight network of streets, with wide sidewalks, regularly spaced street tree planting and buildings set close to the sidewalks.	**BMPs:** Bioretention & Rain Gardens; Green Roofs; Porous Pavement; Rainwater Harvesting; Proprietary Systems. **Cost:** BMPs in T-5 may cost more due added infrastructure needs (e.g., curbs, underdrains) or if project is a retrofit. **Maintenance:** Additional requirements in T5 may include more frequent inspection for debris accumulation in the BMP. Notes: BMPs such as Bioretention are flexible in their design so they can easily be adapted to an urban setting. Swales may be feasible in certain areas but ONLY if in a treatment train.

not be an appropriate SCM for T5, the downtown urban core as this practice requires a significant amount of space for implementation. However, other tools such as bioretention systems, are appropriate across all transects as the design and implementation of this practice can be modified to work across all land uses and types. This malleability is one of the great benefits to LID; that many of the practices are flexible enough that they can be modified to meet site constraints and performance goals across multiple transect.

The information gathered and organized during Steps 1–6 provide the foundation for selecting SCM types that are most appropriate to meet the stormwater management needs of the site. Appendix B provides substantial detail about SCM applicability and design requirements and can be referenced during the process.

At the completion of Phase II, the site planning for the project is complete. At that point in the site planning process, the development

area should be delineated and the approximate type and potential locations for appropriate SCMs should be identified. The preliminary plan should be documented in addition to the decisions that were made in developing the preliminary plan for future reference and to ensure that the LID planning concepts are carried through to project construction. After the preliminary design is completed, the final design is achieved through identifying the appropriate LID facility type and size for meeting stormwater management needs and requirements.

The example shown in Figure B.2.17 indicates the approximate type and locations of potential stormwater management practices. The type, size, or location could change according to site construction or other site design changes and requirements.

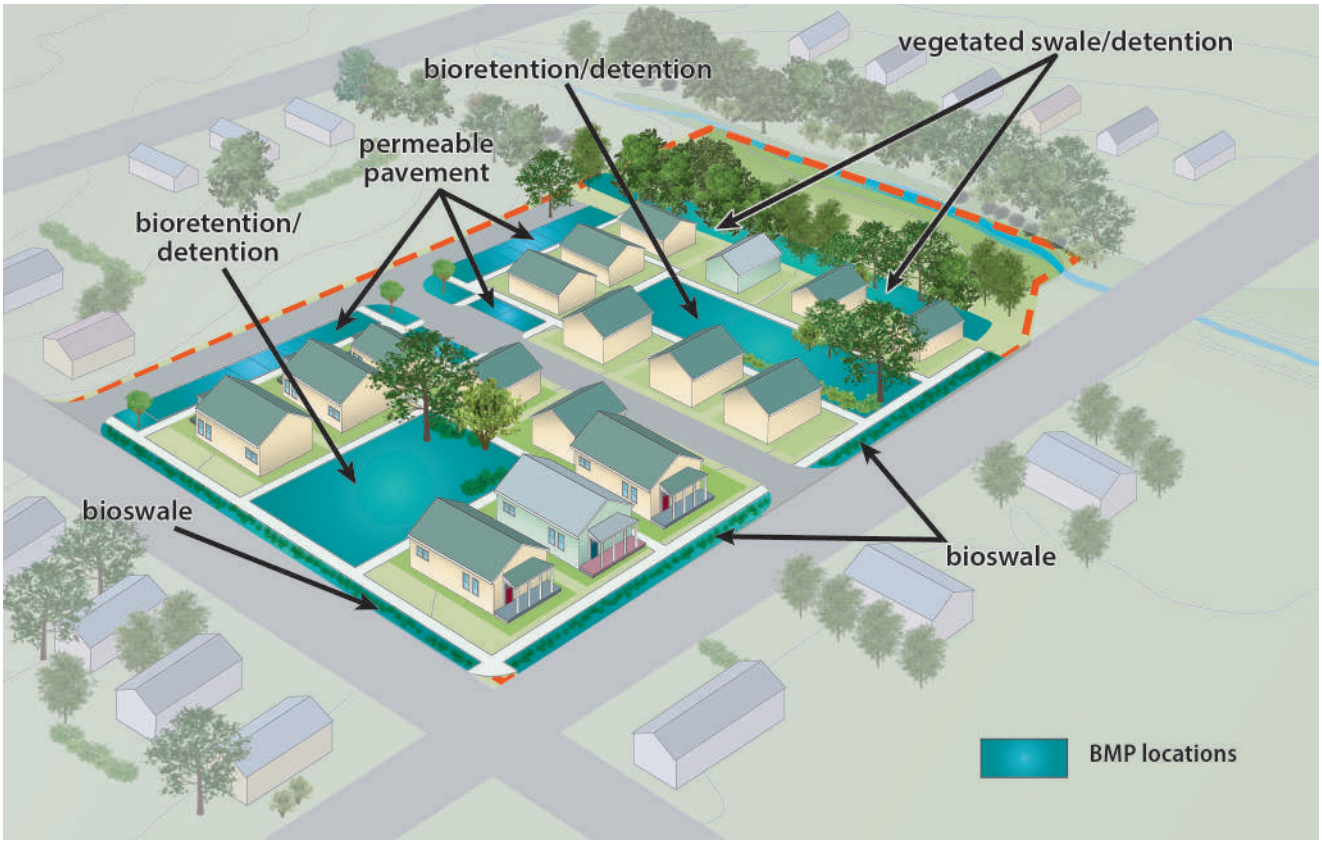


FIGURE B.2.17 SITE PLAN INDICATING ALL POSSIBLE SCM LOCATIONS (BLUE AREAS) AND TYPES (ANNOTATED).

B.2.1.3: Phase III – Final Design

B.2.1.1.1: Step Eight: Size SCMs

Determine the appropriate SCM size using the SCM sizing tools. The level of control that is required for a site to achieve stormwater management goals can be determined through a site-specific hydrologic evaluation. The hydrologic evaluation is performed using hydrologic modeling and analysis techniques. A stepwise process is followed to conduct a hydrologic evaluation:

1. Delineate the watershed and subwatershed areas.
2. Define the design storm (for traditional SCM sizing guidance, refer to Appendix C for LID SCMs refer to Section 3 of the main document).
3. Select the method of analysis to be used.
4. Collect data for predevelopment conditions.
5. Using hydrologic models, evaluate predevelopment (baseline) conditions.
6. Using hydrologic models, evaluate the hydrologic benefits from decreasing and disconnecting impervious areas, and compare the benefits to baseline conditions.
7. Using hydrologic models, evaluate the hydrologic control from implementation of one or more LID SCMs.

For detailed design steps, refer to Section III-B-2 of this manual

B.2.1.2.2: Step Nine: Develop Final Design

Following iterations of Steps 4–7 and SCM sizing in Step 8, additional conventional stormwater control techniques can be added to the site as necessary to meet site drainage and other requirements (Figure B.2.17). Review of the earlier documentation of decisions made during planning phases should also be conducted to ensure that the intent of the LID planning principles were carried through to the final design. The iterative review process can result in more or less area required for stormwater management. Notice that in Figure B.2.17, the iterative process resulted in the use of the common areas for detention.

The following items should be completed during Step Nine:

- Integrate traditional and LID stormwater management controls.
- Verify geotechnical and drainage requirements have been met.
- Complete SCM design.
- Complete site plans.

The key to finalizing the SCM design process is to consult the design instructions for the selected SCM types in Section 3 of this appendix. By following those instructions, the designer can develop final details, plan views, cross sections, profiles, and notes. The example site design in Section III-B-2 illustrates the final sizing and site layout for the properly sited and sized SCM locations.

B.3: RAIN GARDENS AND BIORETENTION FACILITIES



RAIN GARDEN, SAN ANTONIO, TEXAS.

B.3.1: DESCRIPTION

Bioretention areas are landscaped, shallow depressions that capture and temporarily store stormwater runoff. Bioretention areas are the most commonly implemented LID technique because they mimic predevelopment hydrologic conditions, enhance biodiversity and water quality, and can be easily incorporated into both new and existing development (Davis et al. 2009). Runoff intercepted by the practice is temporarily captured in shallow, vegetated depressions then filtered through the soil (often engineered soil) media. Pollutants are removed through a variety of physical, biological, and chemical treatment processes.

Bioretention areas usually consist of a pretreatment system, surface ponding area, mulch layer, and planting soil media. The depressed area is planted with small- to medium-sized vegetation including trees, shrubs, and groundcover that can withstand urban environments and tolerate periodic inundation and dry periods. Plantings also provide habitat for beneficial pollinators and aesthetic benefits for stakeholders and can be customized to attract butterflies or particular bird species. Ponding areas can be designed to increase flow retention and flood control capacity.

In general, there are two kinds of rain gardens. Filtration rain gardens cleanse and detain stormwater runoff. Because they are

specifically lined or have an underdrain to prevent infiltration in some areas (in areas where the influent is deemed too pollutant heavy or for other site constraints) they do not significantly reduce stormwater volumes. However, these systems still provide substantial pollutant removal and increase the time of concentration. Infiltration rain gardens cleanse, detain and reduce runoff volumes by allowing water to seep into the surrounding soils.

Rain gardens and bioretention systems are very similar SCMs in their design and function. Both systems can be used in any land use type or for any site. For the purposes of this manual, the main difference between the two systems is that a bioretention system uses engineered soils. Engineered soil refers to the bioretention planting media which has been modified from its original condition or is a specialized, pre-determined mix of materials. While rain gardens do not include engineered soils, they can include slightly modified soils. Both systems can be designed with or without underdrains or liners. Circumstances where underdrains or liners would be required are described in Section B.3.3. The basic components of the system are shown in Figure B.3.1.

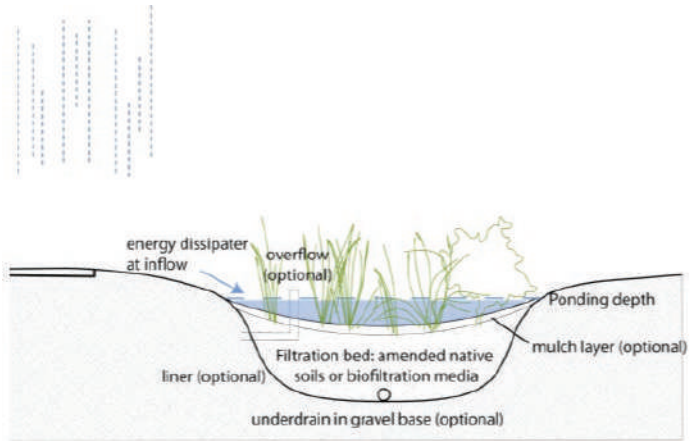


FIGURE B.3.1 A DIAGRAM OF THE BASIC RAIN GARDEN/ BIORETENTION SYSTEM COMPONENTS, INCLUDING OPTIONAL COMPONENTS.

Bioretention areas are well suited to the San Marcos region because they can be adapted to a variety of site constraints and take advantage of the semi-arid climate for evapotranspiration. Advantages and limitations of bioretention areas are outlined below in Table B.3.1.

TABLE B.3.1 ADVANTAGES AND LIMITATIONS OF BIORETENTION AREAS

ADVANTAGES	LIMITATIONS
• Efficient removal of suspended solids, heavy metals, adsorbed pollutants, nitrogen, phosphorus, and pathogens • Can effectively reduce peak runoff rates for relatively frequent storms, reduce runoff volumes, and recharge ground water if soil conditions allow • Flexible to adapt to urban retrofits • Applicable for use in recharge zones, karst, expansive clays, and hotspots when properly designed with impermeable liners • Well suited for use in small areas, and multiple, distributed units can provide treatment in large drainage areas • Can be integrated naturally into landscaping to enhance aesthetics and provide habitat • Standing water only present for 12-24 hours to minimize vector control concerns • Vegetation provides aesthetic benefits, as well as shade and wind breaks and can absorb noise	• Surface soil layer will require restoration if clogged over time • Frequent trash removal might be required, especially in high-traffic areas • Vigilance in protecting native soils from compaction during construction is essential • Single units can serve only small drainage areas • Requires maintenance of plant material and mulch layer • Site limitations include use in areas with slopes less than 5:1, and away from mature trees where the root systems can damage the bioretention.

B.3.2: SELECTION CRITERIA

See B.3.3, Step 2: SCM Siting for placement of bioretention and rain garden facilities.

The major costs associated with bioretention systems are the soil mixture and plants.

The costs are greater than those for landscaping alone; however, the water quality benefits are substantial. The use of underdrains and liners will add to the cost versus bioretention systems designed for infiltration. An additional cost-benefit is received when bioretention or rain garden features double as traffic calming devices, such as a street bump-out.

B.3.3: DESIGN SPECIFICATIONS

The design of a bioretention area can be broken down to a nine-step process. Table B.3.2 summarizes the steps, which are described in greater detail in this chapter.

Step 1. Determine the Volume of Water and Flow Rates to Treat

The bioretention area must be sized to fully capture the desired or required design storm volume and filter it through the soil media. Relevant regulatory requirements are summarized in Appendix E. Surface storage (in the ponding area) and soil pore space (in the plant rooting zone and the underlying media and gravel drainage layers) provide capacity for the design storm volume retention. Appendix E outlines methods for determining design runoff depths associated with a range of annual treatment efficiencies. Once the design runoff depth is determined (on the basis of the desired level of treatment), a runoff volume can be determined for the contributing watershed using this depth and the methods outlined in Appendix E.

Peak flow rates for the design storm should also be calculated, using the methods outlined in Appendix E so that the inlet and pretreatment can be accordingly sized and flow attenuation can be considered.

Step 2. SCM Siting

Bioretention basins are versatile stormwater SCMs that can effectively reduce pollutants and can be integrated into site plans with various configurations and components. Stormwater treatment should be considered as an integral component and incorporated in the site design and layout from conception. Many times, determining how the bioretention area will be included in the site design is a critical and required first step. How the water is routed to the bioretention area and the available space will be key components in determining how the bioretention area is configured. Site assessment, planning, and site design are discussed in detail in the Stormwater Integration Technical Manual. The following is a list of settings where bioretention can be incorporated to meet more than one project-level or watershed-scale objective:

- Landscaped parking lot islands
- Common landscaped areas
- In parks and along open space edges
- Within rights-of-way along roads

How the bioretention area is configured will determine the required components. Bioretention areas can serve the dual purpose of stormwater management and landscape design and can significantly enhance the aesthetics of a site. Figure B.3.2 Basic bioretention components shows an example of the components of a typical bioretention area. When siting bioretention, consideration must always be given to providing access for routine, intermittent, and rehabilitative maintenance activities. When siting bioretention facilities, a 50-foot setback from septic fields should be provided. Setback from a foundation or slab should be 5 feet or greater.

Bioretention areas can be combined with other SCMs to form a treatment train that can provide enhanced water quality treatment and reductions in runoff volume and rate. For example, runoff can be collected from a roadway in a vegetated swale that then flows to a bioretention area. Both facilities can be reduced in size on the basis of demonstrated performance for meeting the stormwater runoff requirements and addressing targeted pollutants of concern.

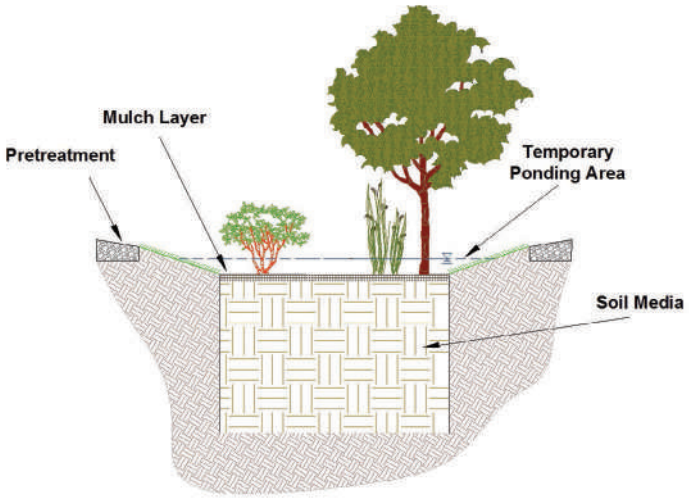


FIGURE B.3.2 BASIC BIORETENTION COMPONENTS.

TABLE B.3.2 ITERATIVE DESIGN STEP PROCESS

DESIGN STEP	DESIGN COMPONENT/ CONSIDERATION	GENERAL SPECIFICATION
1 Determine SCM Treatment Volume	Use Appendix E	
2 SCM Siting	Based on available space and maintenance access, incorporate into parking lot islands, medians, and perimeter; install along the roadway right-of-way; incorporate as landscaped areas throughout the property; or dedicate space for larger, centralized bioretention areas	
3 Determine SCM Function and Configuration	Impermeable liner	If non-infiltrating (per geotechnical investigation), use an impermeable clay layer, geomembrane liner, and concrete (as described in Common Design Elements)
	Lateral hydraulic restriction barriers	Use concrete or geomembrane to restrict lateral flows to adjacent subgrades, foundations, or utilities.
	Underdrain (required if subsoil infiltration rate is less than 0.5 in/hr)	Schedule 40 PVC pipe with perforations (slots or holes) every 6 inches. 4-inch diameter lateral pipes should join a 6-inch collector pipe, which conveys drainage to the downstream storm network. Provide cleanout ports/observation wells for each underdrain pipe at spacing consistent with local regulations. See Common Design Elements
	Internal water storage (IWS)	If using underdrain and infiltration, elevate the outlet to create a sump for additional moisture retention to promote plant survival and enhanced treatment. Top of IWS should be greater than 18 inches below surface.
	No underdrain	If design is fully infiltrating, ensure that subgrade compaction is minimized.
4 Size the System	Temporary ponding depth	6-18 inches (6-12 inches near schools or in residential areas); average ponding depth of 9 inches is recommended
	Soil media depth	2-4 feet (deeper for better pollutant removal, hydrologic benefits, and deeper rooting depths)
	Surface area	Find surface area required to store treatment volume within temporary ponding depth, soil media depth, and gravel drainage layer depth (media porosity $\approx$ 0.35 and gravel porosity $\approx$ 0.4)
5 Specify Soil Media	Composition and texture	85-88% sand, 8-12% fines, 2-5% plant-derived organic matter (animal wastes or by-products should not be applied)
	Permeability	1-6 in/hr infiltration rate (1-2 in/hr recommended)
	Chemical composition	Total phosphorus $<$ 15 ppm, pH 6-8, CEC $>$ 5 meq/100 g soil
	Drainage layer	Separate soil media from underdrain layer with 2 to 4 inches of washed sand, followed by 2 inches of choking stone (ASTM No. 8) over a 1.5-foot envelope of ASTM No. 57 stone.
6 Design Inlet and Pretreatment	Inlet	Provide stabilized inlets (see Common Design Elements)
	Pretreatment	Install rock armored forebay (concentrated flow), gravel fringe and vegetated filter strip (sheet flow), or vegetated swale
	Sediment Marker	Sediment markers are required for all areas with forebays and in areas where a high sediment load is expected (see Common Design Elements)
7 Select and Design Overflow/Bypass Method	Outlet configuration	**Online:** All runoff is routed through system—install an elevated overflow structure or weir at the elevation of maximum ponding **Offline:** Only treated volume is diverted to system—install a diversion structure or allow bypass of high flows (see Diversion Structures for details)
	Peak flow mitigation	Provide additional detention storage and size an appropriate non-clogging orifice or weir to dewater detention volume
	Sediment Marker	Sediment markers should be included at a facility's outfall (see Common Design Elements)
8 Select Mulch and Vegetation	Mulch	Dimensional chipped hardwood or triple shredded, well-aged hardwood mulch 3 inches deep.
	Vegetation	See Plant List
9 Design for Multi-Use Benefits	Include features to enhance habitat, aesthetics, public education, and shade.	

Step 3. Determine SCM Function and Configuration

Intended bioretention functions and configuration must be characterized early in the design process. Infiltration through native soils provides the greatest treatment potential and lowest cost. Where infiltration is limited, a high level of treatment can still be provided by filtering stormwater through an engineered soil media. The following subsections describe the necessary steps to determine if the bioretention area will safely function as an infiltration or filtration SCM.

Geotechnical Investigation

A licensed soil scientist or geotechnical engineer should conduct a geotechnical investigation before the SCM design. The investigator should determine the infiltration rate of the soils at the potential subgrade of the bioretention cell, the depth to the seasonally high groundwater table, presence of expansive clay minerals, and whether there is a risk of sinkhole formation. Site location with respect to aquifer recharge zones, steep slopes, water supply wells, and septic drain fields must also be assessed. For more details, see Common Design Elements.

Determine if Underdrains and Impermeable Liners are Required

Underdrains will be required if a bioretention cell is lined, adjacent to a steep slope, or if the subsoil infiltration rate (as determined during the geotechnical analysis) is less than 0.5 inch per hour (in/hr). Use Table B.3.3 to determine if a bioretention area requires an impermeable liner or underdrain.

Determine if Lateral Hydraulic Restriction Barriers are Required

When bioretention areas are near sensitive infrastructure such as pavement subgrades or buried utilities, hydraulic restriction barriers are often required to prevent lateral seepage. Hydraulic restriction barriers are often installed the full depth of excavation, but occasionally they are keyed in to greater depths to ensure vertical, deep infiltration; the geotechnical investigator should determine the required extent of hydraulic restriction barriers. Common Design Elements provides specific details concerning lateral hydraulic restriction barrier design.

Design Underdrain and Internal Water Storage

The underdrain configuration greatly affects the gradient for water movement through a bioretention cell, and the hydrologic and water quality performance. Conventionally drained cells feature an underdrain that freely drains and outlets at the elevation of the subgrade (Figure B.3.3). Infiltration and pollutant load reduction can be further enhanced by upturning the underdrain to create a sump (Brown and Hunt 2011a). This internal water storage (IWS) zone enhances exfiltration into underlying soils while maintaining aerobic soil conditions in the plant rooting zone. It is most convenient to upturn the underdrain in the outlet structure using a tee-connection; this allows easy access to the underdrain for inspection and maintenance (Figure B.3.4 and Figure B.3.5). IWS can be used in conjunction with an impermeable liner, but volume calculations must account for the possibility of prolonged saturation in the lower media. Inclusion of IWS is recommended in arid and semi-arid regions (such as San Marcos) to maintain soil moisture for plant health (Li et al. 2010; Barrett et al. 2012; Houdeshel et al. 2012). To provide an aerobic root zone and to reduce mobilization of previously captured pollutants, the IWS zone should be at least 18 inches below the surface (Hunt et al. 2012).

Table B.3.3 Decision Table for Determining Underdrain and Impermeable Liner Requirements

Impermeable liners must be used if…	Underdrains must be used if…
• Site is in Edwards Aquifer Recharge Zone, Contributing Zone, or Transition Zone (Barrett 2005) • Soil contamination is expected or present • Karst geology presents risk of sinkhole formation • Runoff could unintentionally be received from a stormwater hotspot • Site is within 100 feet of a water supply well or septic drain field • Site is within 10 feet of a structure/foundation • Infiltrated water could interfere with utilities	• An impermeable liner is needed • Infiltration rate of underlying soils is less than 0.5 in/hr • Site is within 50 feet of a steep, sensitive slope (as determined in the geotechnical analysis—see Common Design Elements)

The underdrain piping should consist of a main collector pipe and two or more lateral branch pipes, each with a minimum diameter of 4 inches. Underdrains should be perforated with ¼–½ inch openings, 6 inches center to center. The pipes should have a minimum slope of 1% (1/8 inch per foot) and the laterals should be spaced at intervals of no more than 10 feet. Each individual underdrain pipe should have a cleanout access location. Ideally the cleanout access will be located in the facility embankment to reduce the possibility of bypass if the cleanout is damaged. All piping is to be Schedule 40 PVC. For recommended underdrain specifications, see Table B.8.1 in Common Design Elements.

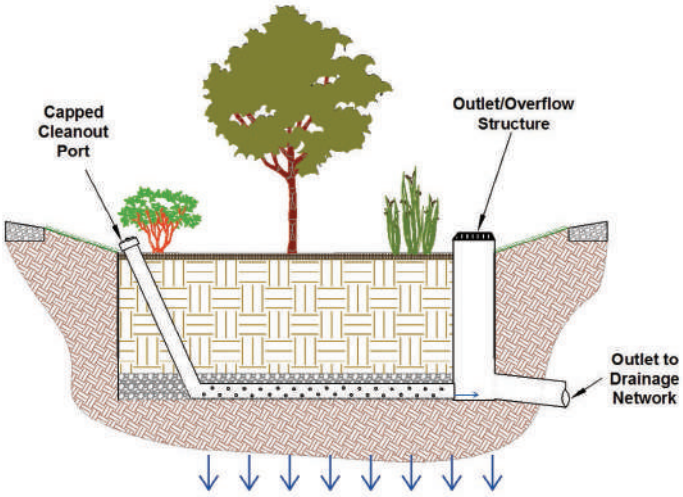


FIGURE B.3.3 CONVENTIONALLY DRAINED BIORETENTION CROSS SECTION SHOWING UNDERDRAIN.

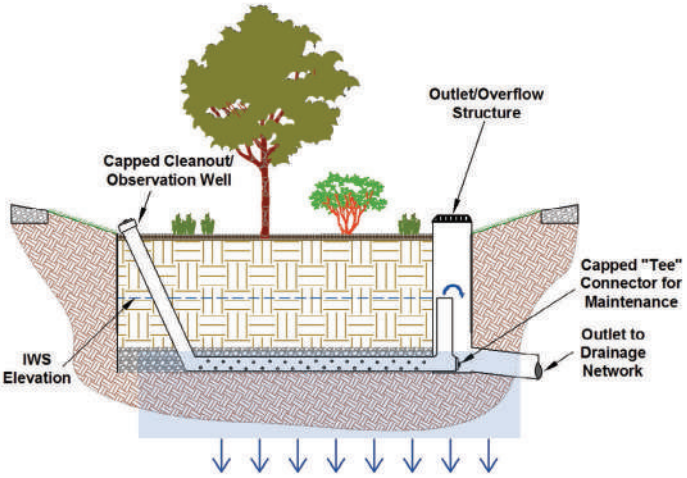


FIGURE B.3.4 BIORETENTION CELL PROFILE WITH IWS DRAINAGE CONFIGURATION.





FIGURE B.3.5 UPTURNED UNDERDRAINS INSIDE BIORETENTION OUTLET STRUCTURES CREATE IWS IN SOIL MEDIA TO IMPROVE INFILTRATION, WATER QUALITY, AND PLANT HEALTH.

Raleigh, North Carolina. Source: Tetra Tech

Rocky Mount, North Carolina. Source: NCSU BAE

Step 4. Size the System

The required water quality treatment volume is determined in Appendix E. Vertical dimensions should be selected on the basis of pollutants of concern and site constraints before calculating the SCM footprint. The following subsections provide guidance on sizing the surface ponding depth, media depth, and footprint of bioretention areas. A schematic of a bioretention system is shown below in Figure B.3.6.

Surface Ponding Depth

Bioretention areas should have a maximum ponding depth of 12 inches but can temporarily detain runoff to a depth of 18 inches if designed for peak flow mitigation (Heasom et al. 2006; more detail concerning peak flow mitigation is provided in Step 7). Although research has demonstrated excellent performance from bioretention areas with deeper ponding depths (more than 12 inches), greater care must be taken to select vegetation that can withstand both inundation and drought, and public safety must be considered (Hunt et al. 2012). Maximum ponding depth might also be limited by vertical constraints of the site, including the elevation of existing downstream storm drain networks. For these reasons, a 9-inch average ponding depth is typically preferred. Local freeboard requirements (typically 1.0 foot for online systems and 0.5 foot for offline systems) should also be considered when selecting a ponding depth (Barrett 2005).

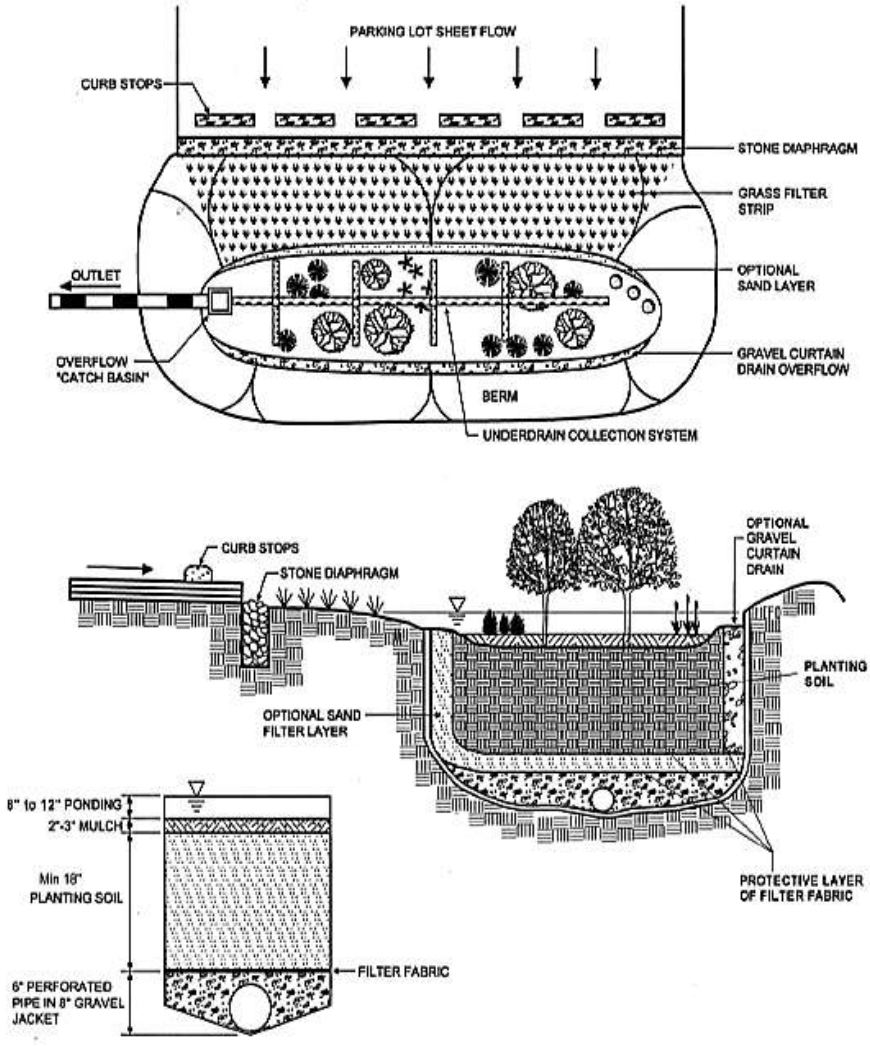


FIGURE B.3.6 SCHEMATIC OF A BIORETENTION SYSTEM.

Soil Media Depth

Soil media depth should be optimized to meet hydrologic and water quality goals but should have a minimum depth of 2 feet (3 feet is recommended for systems with IWS; Hunt et al. 2012). The soil media provides a beneficial root zone for the chosen plant palette and adequate water storage for the water quality volume. A deeper soil media depth will provide a smaller surface area footprint by allowing more storage in the pore spaces and subsequently more evapotranspiration of stormwater by plants.

Table B.3.4 summarizes the minimum recommended media depths for targeted removal of various pollutants. Considering the target pollutant, the depth of the media in a bioretention cell should be between 2 and 4 feet. That range reflects the fact that most of the pollutant removal occurs within the first 2 feet of soil, and excavations deeper than 4 feet become more expensive. The depth should accommodate the desired vegetation (shrubs or trees). If the minimum depth of 2 feet is used over restrictive underlying soils or an impermeable liner, only shallow-rooted vegetation should be planted; grassed bioretention cells can be as shallow as 2 feet. Bioretention facilities where shrubs or trees are planted could be as shallow as 3 feet unless a soil test indicates that shallower depths will support plant health. Media depths greater than 3 feet might be desired for additional pollutant removal, thermal load reduction, and hydrologic benefits, but 3 feet is typically sufficient. If large trees are to be planted in deep fill media, care should be taken to prevent overturning in high winds. Stakes and guy lines might be required to stabilize the trees during establishment.

TABLE B.3.4 MINIMUM BIORETENTION MEDIA DEPTH TO TREAT POLLUTANTS OF CONCERN (HUNT ET AL. 2012)

POLLUTANT OF CONCERN	REMOVAL ZONE	RECOMMENDED DEPTH
Sediment	Surface, top 2-8 inches	2 feet
Total nitrogen	At depth in IWS layer (>2 feet)	3 feet
Total phosphorus	Top 1-2 feet	2 feet
Pathogens	Top 1-2 feet	2 feet
Metals	Top 1-2 feet	2 feet
Oil and grease	Surface	2 feet
Temperature	At depth	4 feet

Size Surface Area

The footprint of the bioretention area should be calculated after the desired ponding depth and soil media depth have been selected. Bioretention areas should be sized to fully capture the treatment volume (from Appendix E) within the surface ponding zone and subsurface pore space. Available storage in the subsurface soil media and gravel drainage layer should be determined on the basis of the laboratory-measured porosity of materials that will be installed on-site; this information is typically available from suppliers or quarries. The porosity, *n*, of bioretention media can be estimated as 0.35, and the porosity of ASTM No. 57 gravel can be estimated as 0.40 for preliminary calculations (Brown et al. in press).

$$n = \frac{V_v}{V_T} \qquad \text{[Equation B.3.1]}$$

where

n = porosity (volume/volume)

V_v = volume of void space

V_T = total volume

The equivalent storage depth for a unit bioretention cross section can be calculated as follows:

$$D_{eq} = (D_{surface}) + (n_{media} \times D_{media}) + (n_{gravel} \times D_{gravel}) \qquad \text{[Equation B.3.2]}$$

where

D_{eq} = equivalent depth of water stored in representative cross sectional of bioretention

$D_{surface}$ = average depth of temporary surface ponding (maximum 12 inches)

n_{media} = porosity of soil media

D_{media} = depth of soil media

n_{gravel} = porosity of gravel drainage layer

D_{gravel} = depth of gravel drainage layer

If the bioretention area is being used for peak flow mitigation, the detention storage depth (volume that will bypass the soil media) cannot be included in $D_{surface}$. More information is provided in Step 7.

The treatment volume (V_{wq}) is divided by the equivalent depth (D_{eq}) to calculate the required bioretention footprint:

$$A = \frac{V_{wq}}{D_{eq}} \qquad \text{[Equation B.3.3]}$$

where

A = required bioretention footprint (area)

V_{wq} = water quality treatment volume (determined in Appendix E)

D_{eq} = equivalent depth

Step 5. Specify Soil Media

Bioretention areas are intended to drain to below the surface in less than 24 hours but should be designed to drain in 12 hours or less as a safety factor. Typically the soil media is dewatered in less than 48 hours for plant health. If a gravel drainage layer is included beneath the bioretention area soil media, stored runoff in the drainage layer should drain in less than 72 hours. The soils must be allowed to dry out periodically to restore hydraulic capacity to receive flows from subsequent storms, maintain infiltration rates, maintain adequate soil oxygen levels for healthy soil biota and vegetation, and to provide proper soil conditions for biodegradation and retention of pollutants.

Organic matter is considered an additive to help vegetation initially establish and contributes to sorption of pollutants; however, organic materials will oxidize over time causing an increase in ponding that could adversely affect the performance of the bioretention area. Additionally, studies in Texas have demonstrated pollutant leaching when bioretention soils were amended with excessive compost (Li et al. 2010). Organic material should therefore be minimized (less than 5 percent of media volume) and consist of minimal plant-based materials. Organic amendments should *not* include any animal manure or by-products, which can export nutrients and pathogens.

High levels of phosphorus in the media have been identified as the main cause of bioretention areas exporting nutrients (Hunt and Lord 2006). All bioretention media should be analyzed for background levels of nutrients. All soil properties should be measured by a qualified soils laboratory with AASHTO, USACE, or State accreditation.

Soil media should meet the specifications listed in Table B.3.5. If the existing soils meet the criteria, it can be used as the soil media. If the existing soils do not meet the criteria, soils should be amended with the appropriate components or a substitute media must be used.

Table B.3.5 Bioretention Soil Media Specifications (Hunt et al. 2012)

Parameter	Specification
Texture and Composition (by volume)	Soil media should consist of a loamy sand conforming to the following specifications: • 85 to 88% washed course sand (concrete sand passing a one-quarter-inch sieve or thoroughly washed mortar sand passing a one-eighth-inch sieve; either ASTM C-33 (concrete sand) or ASTM C-144 (masonry sand)) • 8 to 12% fines passing a #270 sieve (8% fines typically yields an infiltration rate near 2 in/hr, whereas 12% fines yields an infiltration rate near 1 in/hr) • 2 to 5% organic matter • Less than 5% clay content
Organic Matter Material	Aged bark fines, hardwood chips, leaf litter, or similar plant-derived organic material. Studies have also shown newspaper mulch to be an acceptable additive (Kim et al. 2003; Davis 2007). Organic matter should not include animal manure or by-products.
Infiltration Rates	0.5 to 6 in/hr (1-2 in/hr recommended for comprehensive pollutant treatment and hydrologic benefit; Hunt et al. 2012)
pH	6 to 8
Cation Exchange Capacity (CEC)	Greater than 5 milliequivalents (meq)/100 g soil
Phosphorus	Total phosphorus should not exceed 15 ppm

Step 6. Design Inlet and Pretreatment

Inlets must be designed to convey the design storm volume into the bioretention area while limiting ponding or flooding at the entrance to the bioretention area and protecting the interior of the bioretention area from damage. Take care during grading to ensure that the drainage area is properly sloped toward the bioretention area and that the inlet elevation is at least as high as the intended maximum ponding depth. In addition to inlet design, pretreatment is critical to remove coarse sediment and debris to prolong the functional life of the soil media. Several options are available depending on the configuration of the bioretention area and the drainage area characteristics.

Sediment markers are required at all bioretention inlets, especially in areas where a high sediment load is expected. The sediment marker is a tool in order to make a visual assessment of the amount of sediment that is entering a SCM. Sediment removal should be included in the routine maintenance of a bioretention facility, and the sediment should be disposed of properly.

Inlets

The way in which runoff is routed to the bioretention area will dictate the type of inlet. If sheet flow constitutes the source of runoff, curb cuts are typically used; design guidance for curb cuts is provided in the Common Design Elements section. If flows are concentrated, channels or conduit can be used to convey runoff to the bioretention area.

Energy Dissipation and Pretreatment

Design of pretreatment measures will vary depending on the site layout. If sheet flow (such as parking lot runoff) is conveyed to the treatment area, the site must be graded in such a way that minimizes erosive conditions. Gravel fringes between pavement and grassed surfaces can help distribute flow and provide initial pretreatment. Gravel should consist of a 2-inch layer of ASTM No. 57 stone (underlain by filter fabric) extending 2 to 3 feet from the pavement edge, where space allows (Figure B.3.7). Filter strips should ideally be sodded and graded at 3:1 (horizontal: vertical) slopes or flatter. Any slopes that convey flow should be routinely



FIGURE B.3.7 CURB CUT OPENINGS WITH FLOW TO VEGETATED AREA
San Antonio, Texas: Mission Branch Library. Source: Tetra Tech

inspected for rill erosion, which can contribute excessive sediment to the bioretention area and often represents the most common maintenance issue (Wardynski and Hunt 2012). Take care to prevent flow from concentrating between parking lot curb stops/blocks.

Runoff can be routed to a bioretention area through a vegetated swale to pretreat incoming flows from impervious surfaces. Whenever concentrated flow is conveyed to the bioretention area (via channels or conduit) a rock-armored forebay should be used to dissipate energy and provide pretreatment of gross solids and sediment. Forebays should compose approximately 10 percent of the total bioretention area and should be designed to dewater between storm events to prevent vector hazards (Hunt and Lord 2006, Hunt et al 2007). Armored inlets can be used where space is limited (as shown in Figure B.3.8 and Figure B.3.9).

Bioretention areas that treat runoff from residential roofs or other *cleaner* (low sediment and debris yield) surfaces might not require pretreatment for trash or sediment but should include energy dissipation to the extent practicable. Energy dissipation can be provided by upturning inflow pipes so that they bubble up diffusely onto a rock apron (Figure B.3.10); otherwise, baffles, blocks, or cobbles can be used to still high velocities. Flow velocities should not exceed 3 feet per second (ft/sec) for grassed surfaces and 1 ft/sec for mulched surfaces.



FIGURE B.3.8 INLET AND PRETREATMENT PROVIDED BY MORTARED COBBLE FOREBAY AND ENERGY DISSIPATER.
Los Angeles, California. Source: Tetra Tech



FIGURE B.3.9 INLETS STABILIZED WITH MORTARED COBBLE.
Los Angeles, California. Source: Tetra Tech



FIGURE B.3.10 UPTURNED INLET FROM ROOFTOP BUBBLES UP DIFFUSELY ONTO GRAVEL PAD.
Chocowinity, North Carolina. Source: Tetra Tech

Step 7. Select the Appropriate Outlet or Bypass Method

Two design configurations (offline or online) can be used for treating storms that are larger than the bioretention area is designed to store. If peak flow cannot be fully mitigated by the flow rate through the soil media, the outlet can be adapted to meter the rate of outflow.

Regardless of the layout of the bioretention facility, a sediment marker should be included at the outlet. This marker will help to determine how much sediment is exiting the system, and can help the designer evaluate any problems that might be occurring within the SCM.

Offline

An offline bioretention area (Figure B.3.11) can be designed such that stormwater bypasses the bioretention area once the capacity has been exceeded. A structure can also be designed that diverts into the bioretention area only the volume of stormwater for which the bioretention area is designed. For more information on diversion structures, see Common Design Elements.

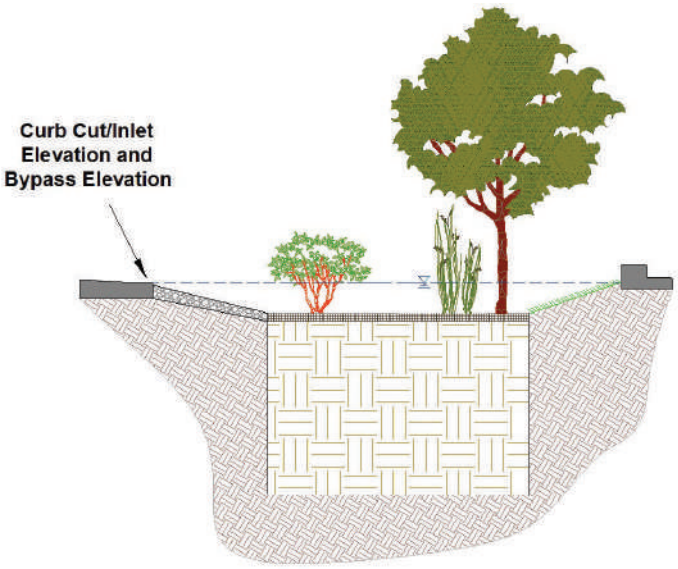


FIGURE B.3.11 OFFLINE BIORETENTION AREA WHERE SYSTEM FILLS TO CAPACITY AND EXCESS FLOW BYPASSES ALONG CURBLINE AT INLET.

Online

For online systems, all flow is routed through the bioretention area and excess runoff overflows an outlet structure. Outlet systems for online bioretention areas can be designed to provide some peak flow mitigation in addition to storing the design volume (see Designing for Peak Flow Mitigation). Appropriate energy dissipation should be incorporated in online systems such that media is not scoured during higher flow events. Two basic options can be used for outlets or overflow for online bioretention systems.

Option 1: Vertical riser

1. An elevated outlet structure (typically an above-grade concrete drop inlet for larger bioretention areas or a PVC pipe for smaller bioretention areas) that is connected to the underdrain or directly to the drainage system.
2. The vertical riser should be sized to safely convey flow greater than the water quality volume. The vertical pipe will provide access for cleaning the underdrains.
3. The inlet to the riser should be set at the specified ponding depth and capped with an appropriate non-clogging grate. Figure B.3.12 shows an example of an online bioretention area with a vertical riser overflow design.

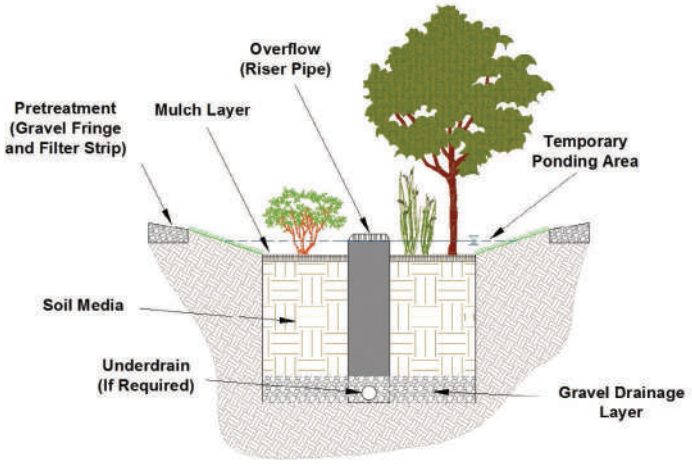


FIGURE B.3.12 ONLINE BIORETENTION AREA WITH A VERTICAL RISER OVERFLOW WITH A VARIABLE FLOW OUTLET STRUCTURE.

Option 2: Level Spreader

1. A level spreader can be used to diffuse overflows from the bioretention area and should be installed along the exit edge or outflow section of the bioretention area. The level spreader should be concrete.
2. The top surface of the level spreader should be installed at a height equal to the ponding depth, or slightly greater if in conjunction with a vertical riser, to allow runoff exceeding the capacity of the bioretention area to safely pass.
3. The level spreader can be designed as a weir to allow for varied outlet flows providing some peak flow mitigation.
4. See Common Design Elements for details on level spreader design.

Typically, bioretention areas constructed in the right-of-way should be designed as offline stormwater treatment facilities. Once a bioretention area constructed in the right-of-way has reached capacity, stormwater flows should bypass the system and continue flow in the existing stormwater drainage system or continue to the next SCM. If a bioretention area constructed in the right-of-way requires underdrains, a vertical riser overflow system can be incorporated as the primary overflow method in addition to the bypass.

Designing for Peak Flow Mitigation

Bioretention areas can be designed for peak flow mitigation by providing additional storage and, if necessary, modifying the outlet structure to discharge water at a controlled rate. Some additional water can be retained in the system above the water quality treatment volume for a short period without affecting the vegetation. If additional ponding depth is provided to store the flood control volume, maximum ponding depth must not exceed 18 inches. The riser should be designed to mitigate for the required peak flow without exceeding the maximum ponding time of 24 hours. This requirement can be achieved by incorporating an orifice or a weir with its invert at the elevation of the water quality treatment volume ponding depth (Figure B.3.13). Orifices that could be clogged by floating mulch or debris should be protected with a trash rack, a hood, or by installing a downturned pipe. The volume of water detained above the elevation of the drawdown orifice or weir cannot be credited toward the water quality treatment volume because this excess water will drain untreated to the storm sewer network without filtering through the soil media. Alternatively, underdrain outflow can be regulated using a restrictor plate, and all runoff can be routed through the soil media.



FIGURE B.3.13 BIORETENTION OUTLET STRUCTURES DESIGNED FOR PEAK FLOW MITIGATION.
Outlet structures designed for peak flow mitigation in Camp Pendleton, CA (left) where a graduated riser pipe regulates drawdown of the detention volume, and (right) Southwest Middle School, Gastonia, NC, where orifices allow controlled dewatering of the detention volume—the water quality treatment volume is retained below the orifice elevation.
(Left) Fort Pendleton, California. Source: Tetra Tech (Right) Gastonia, North Carolina. Source: Tetra Tech, Inc.

Discharge of the detention volume through orifices and weirs can be calculated using the following equations. For further guidance on hydraulic design, refer to USDA-SCS (1956) or Chow (1959).

$$\text{Orifice: } Q = C_d A\sqrt{2gH} \qquad \text{[Equation B.3.4]}$$

$$\text{Weir: } Q = CLH^{3/2} \qquad \text{[Equation B.3.5]}$$

where

Q = discharge (cubic feet per second)

C_d = coefficient of discharge (0.6 for sharp openings, 0.8 for pipe openings)

A = cross sectional area of orifice (square feet)

g = acceleration due to gravity (32.2 ft/s^2)

H = head of water acting on the structure (height of water over the centerline of the orifice or height of water over the crest of the weir; feet

C = discharge coefficient (3.33 for broad-crested weir, 3.0 for sharp crested weir)

L = total length of weir (perpendicular to flow; feet)

Step 8. Select Mulch and Vegetation

Both mulch and vegetation are critical design components of bioretention areas from hydrologic, water quality, and aesthetic perspectives. Much of the biological activity in bioretention areas occurs in the mulch and root zone. The following subsections provide specifications for mulch and vegetation.

Mulch

Mulch is a critical component of the bioretention area because it provides a food source and habitat for many of the biological organisms critical to the function of the bioretention area. Much of the hydrocarbon, metals, and total suspended solids removal is believed to occur near the surface in the mulch layer (Hong et al. 2006; Hatt et al. 2008; Li and Davis 2008; Stander and Borst 2010). The bioretention area should be covered with mulch when constructed and annually replaced to maintain adequate mulch depth. Mulch is also important to sustain nutrient levels, suppress weeds, retain moisture for the vegetation, and maintain infiltrative capacity. Mulch should meet the following criteria:

- Dimensional chipped hardwood material is preferred for its permeability of both water and air. Well-aged, triple-shredded hardwood material (Figure 3.3.14) can also be used if dimensional chipped hardwood material is unavailable (well-aged mulch is defined as mulch that has been stockpiled or stored for at least 12 months).
- Free of weed seeds, soil, roots, and other material that is not hardwood material.
- Mulch depth will be 2 to 4 inches thick, with 3 inches preferred (thicker applications can inhibit proper oxygen and carbon dioxide cycling between the soil and atmosphere).
- Grass clippings, pine nuggets, or pure bark should not be used as mulch.



FIGURE B.3.14 TRIPLE-SHREDDED HARDWOOD MULCH.
Source: Tetra Tech, Inc

Vegetation

One advantage of bioretention areas is that they can be used for the dual purpose of stormwater treatment and landscaping or be integrated into the existing landscape. For bioretention areas to function properly as stormwater treatment and blend into the landscape, vegetation selection is crucial. Appropriate vegetation will have the following characteristics:

1. Plant materials must be tolerant of summer drought and extreme heat, ponding fluctuations, and saturated soil conditions for 12 to 48 hours.
2. It is recommended that a minimum of three tree, three shrub, and three herbaceous groundcover species be incorporated to protect against facility failure from disease and insect infestations of a single species.
3. Vegetation with deep and extensive root systems are more tolerant of extreme hydroperiods and can effectively transpire large volumes of soil water. Planting deep-rooting vegetation directly above buried underdrains should be avoided (although interference of plant roots with underdrains is not a common maintenance issue).
4. Native plant species or hardy cultivars that are not invasive and do not require chemical inputs are recommended to be used to the maximum extent practicable. Only native non-invasive species will be selected in areas designated as natural open space.
5. Vegetated cover with herbaceous material should be at least 70% coverage within the rain garden or bioretention area once established.
6. Shade trees should be free of branches for the bottom 1/3 of their total height and lines of sight must be maintained when planting along roadways. Consideration should be given to their placement in the right-of-way or other areas where existing utilities may exist, both underground and overhead.
7. Tree height and placement should consider overhead utilities.
8. If turfgrass is preferred, sod should be specified that was not grown in clay soils (or washed *bare root* sod should be specified).
9. If establishing the bioretention or rain garden feature by seed, the two ideal seeding periods for *grasses and forbs* are in the fall and spring: mid-September to the end of October, and mid-February to early May. If establishing by plugs, the ideal planting windows are September 1 to November 1 and March 1 to May 1. In general, wildflower seeding is most successful if seeded in the fall. Perennial wildflowers can be seeded in the spring but probably will have limited blooming activity. Annuals perform much better if seeded in the fall. Grasses can be seeded in the spring however some species benefit from cold stratification in the winter. Cool season grasses need to be seeded in the fall.

Buffalo grass (*bouteloua dactyloides*) and big muhly (*muhlenbergia lindheimeri*) have both been shown to provide enhanced nutrient removal. Many options exist for vegetation arrangement and will most likely depend on the landscaping of the area around the bioretention. Size-limited landscaping could be required for bioretention areas in the right-of-way to maintain the required sight distances. Consideration should be given to water depth, bioretention configuration, desired aesthetic appearance, and potential multi-use benefits. An example planting plan is shown in Figure B.3.15 and a non-exclusive list of regionally-appropriate species is shown in Table B.3.6.

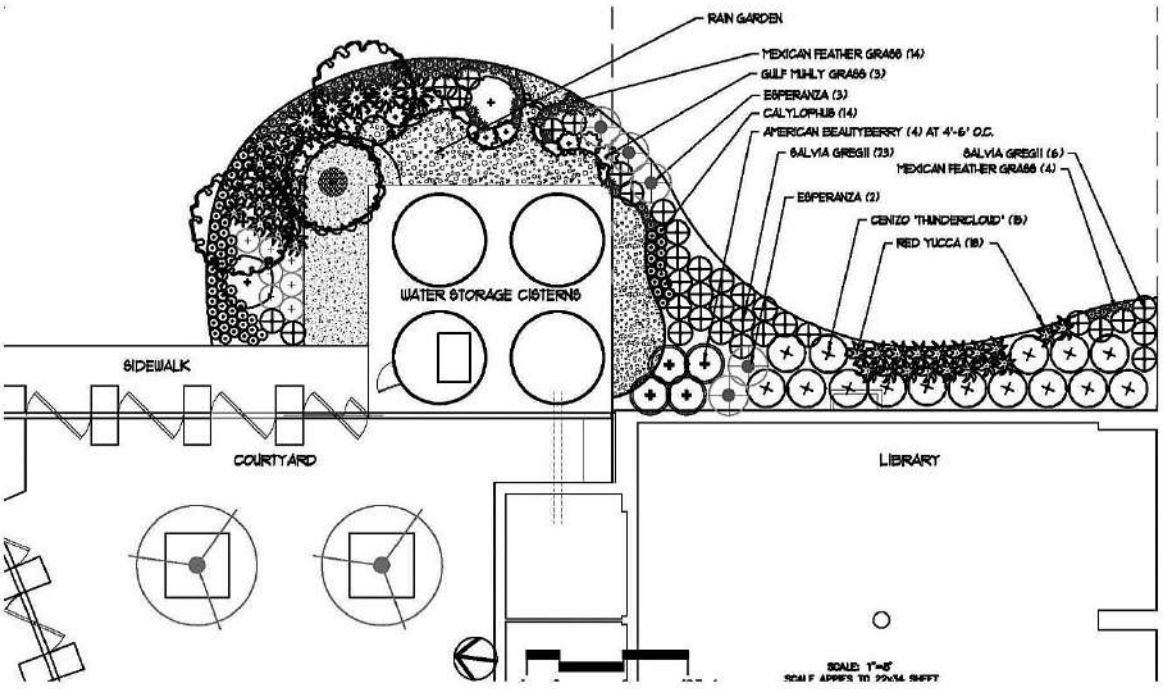


FIGURE B.3.15 EXAMPLE BIORETENTION PLANTING PLAN.

TABLE B.3.6 LIST OF REGIONALLY APPROPRIATE SPECIES

GRASSES	FORBS	SHRUBS AND TREES
Eastern gamagrass *Tripsacum dactyloides*	Frogfruit *Phyla Nodiora*	Sycamore *Platanus occidentalis*
Seep muhly *Muhlenbergia reverchonii*	Blue mistflower *Conoclinium coelestinum*	Bald cypress *Taxodium distichum*
Bushy bluestem *Andropogon glomeratus*	Baldwin's ironweed *Vernonia baldwinii*	Cedar elm *Ulmus crassifolia*
Upland switchgrass *Panicum virgatum*	Canada germander *Teucrium canadense*	Possum-haw *Ilex decidua*
Inland sea oats *Chasmanthium latifolium*	Horseherb *Calyptocarpus*	Red mulberry *Morus rubra*
Indiangrass *Sorghastrum nutans*		
Cherokee sedge *Carex cherokeensis*		

Step 9. Design for Multi-Use Benefits

Bioretention can provide excellent ecosystem services and aesthetic value to stakeholders. In addition to enhancing biodiversity and beautifying the urban environment with native vegetation, the following components can be incorporated into bioretention to promote multi-use benefits:

- Simple signage or information kiosks can educate the public on the benefits of watershed protection measures or provide a guide for native plant and wildlife identification (Figure B.3.16).
- Bird and butterfly feeders can be used to attract wildlife to the bioretention area.
- Sculptures and other art can be installed in the bioretention area, and outlet structures can be painted lively colors.
- Bioretention along the roadway (pop outs and curb extensions) can serve as a traffic calming features
- Ornamental plants can be cultivated along the perimeter and in the bed of bioretention areas (invasive plants should be avoided).
- Larger bioretention areas can be equipped with pedestrian cross-paths or benches for wildlife viewing (Figure B.3.17).
- Bioretention areas can function as irrigation beds for stormwater captured by other SCMs, such as rainwater harvesting or the reservoir layer of permeable pavement (Figure B.3.17).
- Vegetation with canopy cover can provide shade, localized cooling, and noise dissipation.
- Volunteer groups can be organized to perform basic maintenance as an opportunity to raise public awareness.



FIGURE B.3.16 SIGNAGE POSTED IN FRONT OF ULTRA-URBAN BIORETENTION AREA RAISES PUBLIC AWARENESS.
San Antonio, Texas. Source: Tetra Tech



FIGURE B.3.17 COBBLED BIORETENTION AREA THAT INVITES PEDESTRIAN INTERACTION.
Offices of Bender Wells Clark Design, San Antonio. Source: Bender Wells Clark Design



FIGURE B.3.18 OVERFLOW FROM A RAINWATER CISTERN IS DISCHARGED TO A COBBLED BIORETENTION AREA.
Mission Library, San Antonio. Source: Bender Wells Clark Design

CONSTRUCTION CONSIDERATIONS

Construction technique and sequencing are critical to bioretention cell performance. Failure of improperly constructed systems can be easily avoided by effective communication with the contractor and by inspection during key steps. In addition to the general construction considerations provided as a link in Appendix A, emphasizing the following points will help ensure successful installation of bioretention cells.

B.3.3.1: Install Erosion and Sediment Control Devices

All sediment control devices need to be in place before starting main construction as fine sediment from construction can significantly compromise the rain garden. If possible, rain gardens can be constructed last to avoid sediment entering the rain garden. If this is not possible, the rain gardens should be protected from all construction activities, especially from receiving any construction runoff, and only begin service when the contributing drainage area has been stabilized. Alternatively, the areas identified to become rain gardens can be used as temporary construction ponds so long as the areas are excavated with all fine sediment removed before the rain gardens are filled with appropriate soil or filtration media.

B.3.3.2: Minimize and Mitigate Compaction by Scarifying Subsoil Surface

Compaction of underlying soils (an inherent consequence of construction) can decrease infiltration rates and result in poor drainage, extended periods of standing water, plant die-off, and, in some cases, leaching of previously captured pollutants. Construction effects on underlying soils can be mitigated by excavating the last 12 inches of cut with a toothed excavator bucket. This method breaks up compacted layers and prevents *smearing,* which can seal the subsoil surface (Brown and Hunt 2010). Infiltration can also be significantly enhanced by ripping or trenching the subsoils to a depth of 9–12 inches (Tyner et al. 2009). Ripped furrows (on 3-foot centers) should be filled with a clean sand to maintain free-flowing conditions. Trenches are typically constructed 12 inches wide on 6-foot centers and are filled with pea gravel.

B.3.3.3: Inspect Soil Media before Placement

It is important to ensure that the soil media is consistent with specifications before installation—media that is too sandy will not provide adequate treatment, whereas media that is too fine might not drain in adequate time (Carpenter and Hallam 2010). To field-verify the texture of soil media, moisten the soil and form into a 1-inch ball. Drop the ball from 1 foot onto the open palm of the hand. The ball should break apart on impact, indicating that it is a sandy soil. When rubbed between the fingers, the moist soil should also leave a thin layer of mud residue on the skin, indicating that fines are present in the mix. Soil media should also contain a small amount of plant-based organic matter evenly distributed throughout the mix—the organic matter should not smell like manure. Note: These inspection techniques are intended for field verification and do *not* substitute for laboratory soil test results.

After placing the planting media, lightly water to allow for natural compaction and settlement. The media is expected to naturally compact by about 20%. Rake the soil as needed to level the soil

out. Due to settlement, the invert area should be overfilled. When leveling the media, it is easier to remove overfilled media than to add new media.

B.3.3.4: Verify that Average Ponding Depth is Provided

It is important to verify that the intended design volume of runoff can be captured by the bioretention cell. Contractors who are unfamiliar with construction of bioretention may try to minimize surface ponding by installing the outlet elevation too low. Cells that do not provide their intended capacity of surface storage will overflow more often than intended and discharge untreated runoff to waterways (Brown and Hunt 2011b; Luell et al. 2011). Therefore, it is critical to check that the average surface ponding depth has been provided and that the bed of the cell has been uniformly graded—this can be performed by simply verifying the overflow/bypass elevation of the system relative to the average elevation of the mulch bed surface. An average depth must be measured because the height of the outlet structure relative to adjacent ground surface is not a reliable indicator of average ponding depth (Wardynski and Hunt 2012). Excessive mulch (deeper than 4 inches) can also displace surface storage volume and should be avoided.

B.3.4: INSPECTION CRITERIA

To maintain the effectiveness of structural SCMs, regular inspection is essential. Generally, SCM inspection and maintenance can be categorized as routine and as needed. Routine activities, performed regularly (e.g., monthly) ensure that the SCM is in good working order and continues to be aesthetically pleasing. Routine inspection is an efficient way to prevent potential nuisance situations from developing and reduce the need for repair or maintenance. Routine inspection also reduces the chance of degrading the quality of the effluent by identifying and correcting potential problems regularly. Property maintenance personnel should be instructed to inspect SCMs during their normal routines.

In addition to routine inspections, as-needed inspection/maintenance of all SCMs should be performed after any event or activity that could damage the SCM, particularly after every large storm event. Post-storm inspections should occur after the expected drawdown period for the SCM, when the inspector can determine if the SCM is draining correctly.

In general, individual SCMs can be described with minimum performance expectations, design criteria, structural specifications, date of implementation, and expected life span. Recording such information will help the inspector determine whether a SCM's maintenance schedule is adequate or requires revision and will allow comparison between the intended design and the as-built conditions. Checklists also provide a useful way for recording and reporting whether major or minor renovation or routine repair is needed. The effectiveness of a SCM might be a function of the SCM's location, design specifications, maintenance procedures, and performance expectations. Inspectors should be familiar with the characteristics and intended function of the SCM so they can recognize problems and know how they should be resolved.

Routine and as-needed SCM inspections consist of technical and non-technical activities as summarized below:

- Inspect the general conditions of the SCM and areas directly adjacent.
- Maintain access to the site including the inlets, side slopes (if applicable), forebay (if one exists), SCM area, outlets, emergency spillway, etc.
- Examine the overall condition of vegetation.
- Eliminate any possibility of public hazards (vector control, unstable public access areas).
- Check the conditions of inflow points, pretreatment areas (if they exist), and outlet structures.
- Inspect and maintain the inlet and outlet regularly and after large storms.
- Ensure that the pretreatment areas meet the original design criteria.
- Check the encroachment of undesirable plants in vegetated areas. This could require more frequent inspections in the growing season.
- Inspect water quality improvement components. Specifically, check the stormwater inflow, conveyance, and outlet conditions.
- Inspect hydrologic functions such as maintaining sheet flow where designed, ensuring functional pretreatment, maintaining adequate design storage capacity, and verifying proper operation of outlet structures.
- Check conditions downstream of the SCM to ensure that flow is properly mitigated below the facility (e.g., excessive erosion, sedimentation).

In every inspection, whether routine or as needed, the inspector should document whether the SCM is performing correctly and whether any damage has occurred to the SCM since the last inspection. Ideally, the inspector will also identify what should be done to repair the SCM if damage has occurred. Documentation is very important in maintaining an efficient inspection and maintenance schedule, providing evidence of ongoing inspection and maintenance, and detecting and reporting any necessary changes in overall management strategies.

Inspection frequency for bioretention facilities is included in Table B.3.7 in Section B.3.6.

B.3.5: RECOMMENDED MAINTENANCE

Bioretention areas require regular plant, soil, and mulch layer maintenance to ensure optimum infiltration, storage, and pollutant-removal capabilities. Table B.3.7 provides a detailed list of maintenance activities and general maintenance considerations are provided in Section 4.3. In general, bioretention maintenance requirements are typical landscape care procedures and consist of the following:

1. **Erosion control:** Inspect flow entrances, ponding area, and surface overflow areas periodically during the rainy season, and replace soil, plant material, or mulch layer in areas if erosion has occurred (for a bioretention inspection and maintenance checklist, see the link to the City of San Marcos Specifications in Appendix A). Properly designed facilities with appropriate flow velocities should not have erosion problems except perhaps in extreme events. If erosion problems occur, the following must be reassessed: (1) flow velocities and gradients within the cell, and (2) flow dissipation and erosion protection strategies in the pretreatment area and flow entrance. If sediment is deposited in the bioretention area, immediately determine the source within the contributing area, stabilize, and remove excess surface deposits. Any exposed soil in the catchment should be permanently stabilized with grass, rock, or other erosion-resistant materials (per TxDOT 2011).

2. **Inlet:** The inlet of the bioretention area should be inspected after the first storm of the season, then monthly during the rainy season to check for sediment accumulation and erosion. Sediment can accumulate especially at inlets where curb cuts or bypass structures are used and should be inspected regularly. A sediment marker shall be installed in order to determine the amount of sediment entering the facility. Any accumulated sediment that impedes flow into the bioretention area should be removed and properly disposed of.

3. **Overflow and underdrains:** Sediment accumulation in the overflow device or underdrain system can cause prolonged ponding and potential flooding. Excess ponding can have adverse effects on vegetation and vector control. Overflow and underdrain systems should be inspected after the first storm of the season, then monthly during the rainy season to remove sediment and prevent mulch accumulation around the overflow. The underdrain system should be designed so that it can be flushed and cleaned as needed. If water is ponded in the bioretention area for more than 72 hours, the underdrain system should be flushed with clean water until proper infiltration is restored.

4. **Plant material:** Depending on aesthetic requirements, occasional pruning and removing dead plant material might be necessary. Replace all dead plants, and if specific plants have a high mortality rate, assess the cause and, if necessary, replace with more appropriate species. Periodic weeding is necessary until plants are established. The weeding schedule can become less frequent if the appropriate plant species and planting density have been used and, as a result, undesirable plants are excluded.

5. **Nutrient and pesticides:** The soil mix and plants are selected for optimum fertility, plant establishment, and growth. Nutrient and pesticide inputs should not be required and can degrade the pollutant processing capability of the bioretention area and contribute pollutant loads to receiving waters. By design, bioretention areas are located in areas where phosphorous and nitrogen levels are often elevated, and they should not be limiting nutrients. If in question, have the soil analyzed for fertility.

6. **Mulch:** Replace mulch annually in bioretention areas where heavy metal deposition is likely (e.g., contributing areas that include industrial and auto dealer/repair parking lots and roads). In areas where metal deposition is not a

concern, add mulch as needed to maintain a 2- to 3-inch depth. Mulch should be replaced every 2 to 5 years.

7. **Soil:** Soil mixes for bioretention areas are designed to maintain long-term fertility and pollutant processing capability. Estimates from metal attenuation research suggest that metal accumulation should not present an environmental concern for at least 20 years in bioretention systems. Replacing mulch in bioretention areas where heavy metal deposition is likely provides an additional level of protection for prolonged performance. If in question, have the soil analyzed for fertility and pollutant levels and consult local regulations for disposal protocol.

8. **Watering:** Plants must be selected to be drought tolerant and not require watering after establishment (2 to 3 years). Watering could be required during prolonged dry periods after plants are established. Irrigation recommendations are as follows:

    a. For grasses and forbs, on the day of installation, the area should be irrigated with at least 1" of water, in more than one event over the course of the next several days. For the following ten days, the area should receive 1" of irrigation three times per week. For the next thirty days, the area should receive 1" of irrigation twice per week. Over the next several months, the area should receive 1" of irrigation one time every other week or as needed depending on local weather conditions.

    b. Drench all trees and shrubs with water twice, during the first 24 hours after installation. This will ensure the root zone is well saturated. Maintenance of soil moisture at or greater than 6" below grade during early (3-6) months is critical for tree establishment. Young saplings should be watered twice a week (saturating the critical root zone) for 2-3 months. At each watering, thoroughly saturate the soil around each tree and ensure proper soil moisture at least 6" below grade. Over the next four months, the root depth should not be allowed to dry out, watering every other week or as necessary depending on local weather conditions. After this initial establishment period, stormwater runoff should provide sufficient irrigation needs. However if there is a long drought period or no significant precipitation for any 4-6 week period over the first two growing seasons, the trees will need supplemental irrigation. Trees should be maintained for two years and inspected at least once a month during this two-year establishment period.

    c. Reduce irrigation frequency if during period of seasonal rains and stop irrigation if there is any puddling, washing away of seed or plants, or erosion. Significant rainfall (on-site rainfall of 1 inch or greater) shall allow the postponement of watering until the next scheduled irrigation.

    d. Reseed areas which did not establish if coverage is less than 70% of the system area.

TABLE B.3.7 INSPECTION AND MAINTENANCE TASKS

TASK	FREQUENCY	INDICATOR MAINTENANCE IS NEEDED	MAINTENANCE NOTES
Catchment inspection	Weekly or biweekly with routine property maintenance	Excessive sediment, trash, or debris accumulation on the surface of bioretention.	Permanently stabilize any exposed soil and remove any accumulated sediment. Adjacent pervious areas might need to be regraded.
Inlet inspection	Weekly or biweekly with routine property maintenance	Internal erosion or excessive sediment, trash, and/or debris accumulation	Check for sediment accumulation to ensure that flow into the bioretention is as designed. Remove any accumulated sediment.
Litter and leaf litter removal	Weekly or biweekly with routine property maintenance	Accumulation of litter and leafy debris within bioretention area	Litter and leaves should be removed to reduce the risk of outlet clogging, reduce nutrient inputs to the bioretention area, and to improve facility aesthetics.
Pruning	1–2 times/year	Overgrown vegetation that interferes with access, lines of sight, or safety	Nutrients in runoff often cause bioretention vegetation to flourish.
Mowing	2–12 times/year	Overgrown vegetation that interferes with access, lines of sight, or safety	Frequency depends on location and desired aesthetic appeal.
Mulch removal and replacement	1 time/2–3 years	Less than 3 inches of mulch remains on surface	Mulch accumulation reduces available surface water storage volume. Removal of decomposed mulch also increases surface infiltration rate of fill soil. Remove decomposed fraction and top off with fresh mulch to a total depth of 3 inches
Temporary Watering	1 time/2–3 days for first 1–2 months, sporadically after established	Until established and during severe droughts	Watering after the initial year might be required.
Fertilization	1 time initially	Upon planting	One-time spot fertilization for first year vegetation.
Remove and replace dead plants	1 time/year	Dead plants	Plant die-off tends to be highest during the first year (commonly 10% or greater). Survival rates increase with time.
Outlet inspection	Once after first rain of the season, then monthly during the rainy season	Erosion at outlet	Remove any accumulated mulch or sediment.
Miscellaneous upkeep	12 times/year	Tasks include trash collection, plant health, spot weeding, removing invasive species, and removing mulch from the overflow device.	

B.3.6: REFERENCES

Barrett, M.E., M. Limouzin, and D.F. Lawler. 2012. Effects of media and plant selection on biofiltration performance *Journal of Environmental Engineering* 139(4):462-470.

Barrett, M.E. 2005. *Complying with the Edwards Aquifer Rules. Technical Guidance on Best Management Practices*. RG-348. Prepared for Texas Commission on Environmental Quality, Field Operations Division, Austin, TX.

Brown, R.A., and W. F. Hunt. 2010. Impacts of construction activity on bioretention performance. *Journal of Hydrologic Engineering* 15(6):386–394.

Brown, R.A., Skaggs, R.W., and Hunt, W.F. In press. Calibration and validation of DRAINMOD to model bioretention hydrology. *Journal of Hydrology.*

Brown, R.A., and W.F. Hunt. 2011a. Underdrain configuration to enhance bioretention exfiltration to reduce pollutant loads. *Journal of Environmental Engineering* 137(11):1082–1091.

Brown, R.A., and W. F. Hunt. 2011b. Impacts of media depth on effluent water quality and hydrologic performance of under-sized bioretention cells. *Journal of Irrigation and Drainage Engineering* 137(3):132–143.

Brown, R. A. and W. F. Hunt. 2010. Impacts of construction activity on bioretention performance. *Journal of Hydrologic Engineering* 15(6): 386-394.

Carpenter, D.D., and L. Hallam. 2010. Influence of planting soil mix characteristics on bioretention cell design and performance. *Journal of Hydrologic Engineering* 15(6):404–416.

Chow, V.T. 1959. Open-Channel Hydraulics. McGraw-Hill, New York, NY.

Davis, A.P. 2007. Field performance of bioretention: Water quality. *Environmental Engineering Science* 24(8):1048–1063.

Hatt, B.E., T.D. Fletcher, and A. Deletic. 2008. Hydraulic and pollutant removal performance of fine media stormwater filtration systems *Environmental Science & Technology* 42(7):2535–2541.

Heasom, W., R. Traver, and A. Welker. 2006. Hydrologic modeling of a bioinfiltration best management practice. *Journal of the American Water Resources Association* 42(5):1329–1347.

Houdeshel, C.D., C.A. Pomeroy, and K.R. Hultine. 2012. Bioretention design for xeric climates based on ecological principles. *Journal of the American Water Resources Association* 48(6):1178–1190.

Hong, E., M. Seagren, and A.P. Davis. 2006. Sustainable oil and grease removal from synthetic stormwater runoff using bench-scale bioretention studies. *Water Environment Research*. 78(2):141-155.

Hunt, W.F., and W.G. Lord. 2006. *Bioretention performance, design, construction, and maintenance*, North Carolina Cooperative Extension, Raleigh, NC.

Hunt, W.F., A.P. Davis, and R.G. Traver. 2012. Meeting hydrologic and water quality goals through targeted bioretention design. *Journal of Environmental Engineering* 138(6):698–707.

Kim, H., E.A. Seagren, and A.P. Davis. 2003. Engineered bioretention for removal of nitrate from stormwater runoff. *Water Environment Research* 75(4):355–367.

Li, H., and A.P. Davis. 2008. Urban particle capture in bioretention media. I: Laboratory and field studies. *Journal of Environmental Engineering* 143(6):409–418.

Li, M.-H., C.Y. Sung, M.H. Kim, and K.-H. Chu. 2010. *Bioretention for Stormwater Quality Improvements in Texas: Pilot Experiments.* Texas A&M University in cooperation with Texas Department of Transportation and the Federal Highway Administration.

Luell, S.K., W.F. Hunt, and, R.J. Winston. 2011. Evaluation of undersized bioretention stormwater control measures (SCMs) for treatment of highway bridge deck runoff. *Water Science and Technology* 64(4):974–979.

Stander, E.K., and M. Borst. 2010. Hydraulic test of a bioretention media carbon amendment. *Journal of Hydrologic Engineering* 15(6):531–536.

Tyner, J.S., W.C. Wright, and P.A. Dobbs. 2009. Increasing exfiltration from pervious concrete and temperature monitoring. *Journal of Environmental Management* 90:2636–2641.

USDA-SCS (United States Department of Agriculture Soil Conservation Service). 1956. Section 5 Hydraulics. National Engineering Handbook. 210-VI-NEH-5. ftp://ftp.wcc.nrcs.usda.gov/wntsc/H&H/NEHhydraulics/neh5.pdf

Wardynski, B.J., and W.F. Hunt. 2012. Are bioretention cells being installed per design standards in North Carolina: A field assessment. *Journal of Environmental Engineering* 138(12):1210–1217.

B.4: POROUS PAVEMENT



RIVER HOUSE PARKING LOT, SAN MARCOS, TEXAS SOURCE: TETRA TECH

B.4.1: DESCRIPTION

Permeable pavement is a highly versatile stormwater SCM because it can effectively reduce pollutants and can be integrated into site plans with various configurations and components. Permeable pavement allows streets, parking lots, sidewalks, and other impervious covers to retain the infiltration capacity of underlying soils while maintaining the structural and functional features of the materials they replace. Permeable pavement has small voids or aggregate-filled joints that allow water to drain through to an aggregate reservoir. Stormwater stored in the reservoir layer can then infiltrate underlying soils or drain at a controlled rate via underdrains to other downstream stormwater control systems. Permeable pavement systems can be designed to operate as underground detention if the native soils do not have sufficient infiltration capacity, or if infiltration is precluded by aquifer protection, hotspots, or adjacent structures. They can be used on both permeable and impermeable soils and in the latter case are designed with an underdrain system. Where soils are sufficiently permeable all the runoff will infiltrate and no discharge of stormwater or associated pollutants will occur. Systems designed with an underdrain provide substantial pollutant removal and increase the time of concentration, which are substantial benefits even when the volume of runoff is not substantially reduced.

Permeable pavement can be developed using modular paving systems (e.g., permeable interlocking concrete pavers, concrete

grid pavers, or plastic grid systems) or poured in place solutions (e.g., pervious concrete or porous asphalt). Some pervious concrete systems can also be precast. In many cases, especially where space is limited, permeable pavement is a cost-effective solution relative to other practices because it doubles as both transportation infrastructure and a SCM. The porous pavement surface is typically placed over a highly permeable layer of open graded gravel and crushed stone. The void spaces in the aggregate layers act as a storage reservoir for runoff. A filter fabric is placed beneath the gravel and stone layers to screen out fine soil particles.

There are several types of porous pavement, including porous asphalt, pervious concrete, pavers, and grid type systems. Porous asphalt pavement consists of an open graded coarse aggregate, bonded together by asphalt cement, with sufficient interconnected voids to make it highly permeable to water. Pervious concrete consists of specially formulated mixtures of Portland cement, uniform, open-graded coarse aggregate, and water. Pervious concrete has enough void space to allow rapid percolation of liquids through the pavement. Pavers themselves are typically impermeable; however, infiltration occurs either in the gaps between the pavers or within openings cast as part of the geometry of the paver.

Advantages and limitations of permeable pavement are outlined in Table B.4.1.

B.4.2: SELECTION CRITERIA

See Section B.4.3, Step 2: SCM Siting.

B.4.3: DESIGN

The design of a permeable pavement system follows a nine-step process, as described in Table B.4.2.

Step 1. Determine Required Storage Volume

Permeable pavement must be sized to fully treat the desired or required design storm volume. Aggregate pore space provides capacity for the design storm volume retention. Appendix E outlines methods for determining design runoff depths associated with a range of annual treatment efficiencies. Once the design runoff depth is determined (on the basis of the desired level of treatment), a runoff volume can be determined for the contributing watershed using this depth and the methods outlined in Appendix E.

Peak flow rates for the design storm should also be calculated, using the methods outlined in Appendix E so that the inlet and pretreatment can be accordingly sized and flow attenuation can be considered.

TABLE B.4.1 ADVANTAGES AND LIMITATIONS OF PERMEABLE PAVEMENT

ADVANTAGES	LIMITATIONS
• Replaces completely impervious surfaces with partially impervious surfaces • Reduces stormwater runoff rate and volume by increasing the time of concentration • Reduces loads of some pollutants in surface runoff by reducing the volume of stormwater leaving a site • Reduces stormwater infrastructure footprint and promotes multi-benefit uses by using treatment area for parking/driving with possible cost reductions • Increases ground water recharge • Adaptable to urban retrofits • Many options available depending on specific site needs and aesthetics • Applicable for use in recharge zones, karst, expansive clays, and hotspots when properly designed	• Potential for clogging of porous media by sediment, which could lead to reduced effectiveness without proper maintenance • Should not receive runon from adjacent pervious surfaces with high sediment/debris yield • Typically not cost effective for high-traffic areas or for use by heavy vehicles (requires increased structural design and maintenance frequency) • Permeable pavement should be installed only by contractors qualified and certified for permeable pavement installation • Typically recommended for grades of 5% or less

TABLE B.4.2 ITERATIVE DESIGN STEP PROCESS

DESIGN STEP		DESIGN COMPONENT/ CONSIDERATION	GENERAL SPECIFICATION
1	**Determine SCM Size**	Use Appendix E	
2	**SCM Siting**	Based on available space, incorporate into parking lots (solely parking stalls or parking stalls and driving lanes), parking lanes along roadways, pedestrian sidewalks and plazas, and fire access roads. If outside the Edwards Aquifer Contributing Zone or if the system will be lined with an impermeable liner, runoff from adjacent impervious surfaces less than or equal to the permeable pavement area is allowed.	
3	**Select Permeable Pavement Surface Course**	Surface course type	Pervious concrete, porous asphalt, and permeable interlocking concrete pavers (PICP) are the preferred types of permeable pavement because detailed industry standards and certified installers are available. Concrete grid pavers and plastic grid systems also have useful applications.
4	**Determine SCM Function and Configuration**	Impermeable liner	If non-infiltrating (per geotechnical investigation), use impermeable clay liner, geomembrane, or concrete (as described in Common Design Elements)
		Lateral hydraulic restriction layers	Use concrete or geomembrane to restrict lateral flows to adjacent subgrades, foundations, or utilities.
		Underdrain (required if subsoil infiltration rate $<$ 0.5 in/hr)	Schedule 40 PVC pipe with perforations (slots or holes) every 6 inches. 4-inch diameter lateral pipes should join a 6-inch collector pipe, which conveys drainage to the downstream storm network. Provide orifice at underdrain outlet sized to release water quality volume over 2-5 days. See Common Design Elements
		Internal water storage (IWS)	If using underdrain and infiltration is feasible, elevate the outlet to create a sump to enhance infiltration and treatment.
		No underdrain	If design is fully infiltrating, ensure that subgrade compaction is minimized.
		Observation wells	Provided capped observation wells to monitor drawdown.
		Subgrade slope and Geotextile	Subgrade slope should be 0.5% or flatter. Baffles should be used to ensure water quality volume is retained. Geotextile should be used along perimeter of cut to prevent soil from laterally entering the aggregate voids.
5	**Design the Profile**	Temporary surface ponding depth (Edwards Aquifer Zones)	Surface ponding should be provided (by curb and gutter) to capture the design storm in the event that the permeable pavement surface clogs
		Specify sand/soil filter layer	With underdrains: min. 4-inch layer of ASTM C-33 washed sand above gravel of underdrain drainage layer No underdrains: min. 12-inch subsoil (see Common Design Elements)
		Calculate surface area and reservoir depth	Water quality volume should be fully stored within the aggregate base layers below the surface course. Base layer should be washed ASTM No. 57 stone (washed ASTM No. 2 may be used as a subbase layer for additional storage).
		Structural design	A pavement structural analysis should be completed by a qualified and licensed professional
6	**Design for Safe Bypass/Conveyance of Larger Storms**	Large storm routing	Poured in place systems (pervious concrete or porous asphalt): system can overflow internally or on the surface Modular/Paver-type systems (PICP): internal overflow is required to prevent upflow and transport of bedding course
7	**Edge Restraints and Transitions**	Transition strip	Provide a concrete transition strip between any permeable and impermeable surfaces and around the perimeter of PICP installations
8	**Design Signage**	Signage should prohibit activities that cause premature clogging and indicate to pedestrians and maintenance staff that the surface is intended to be permeable	
9	**Design for Multi-Use Benefits**	Provide educational signage, enhanced pavement colors, or stormwater reuse systems.	

Step 2. SCM Siting

Permeable pavement is a highly versatile stormwater SCM because it can effectively reduce pollutants and can be integrated into site plans with various configurations and components. Stormwater treatment should be considered as an integral component and incorporated in the site design and layout from conception. Many times, determining how permeable pavement will be included in the site design is a critical and required first step. How the water is routed to the permeable pavement and the available space will be key components in determining how the permeable pavement is configured.

Permeable pavement is typically designed to treat stormwater that falls on the actual pavement surface area and has been used at commercial, institutional, and residential sites in spaces that are traditionally impervious. Outside the Edwards Aquifer Contributing Zone, Transition Zone, and Recharge Zone, runoff from adjacent surfaces is allowed but must be limited to runoff from stabilized areas with very little sediment yield. A maximum drainage area to permeable pavement area ratio of 1:1 is recommended. The maximum drainage area is 15 acres. In addition, there should be a minimum 4-foot clearance from the bottom of the system to bedrock or to the water table.

Runoff from pervious surfaces or high-sediment areas should be prevented, and permeable pavement should not be installed in areas prone to flooding with sediment-laden water (e.g., floodplains) because excessive sediment can prematurely clog the pores. In addition, porous pavement is not recommended in areas where wind erosion supplies significant amount of windblown sediment. Overhanging trees should also be avoided to reduce the deposition of detritus on the pavement surface, which can be ground into joints and pores if not routinely removed.

Several minimum setback requirements must be met for porous pavement:

- Minimum setback from water supply wells: 100 feet.
- Minimum setback from building foundations (down gradient): 10 feet.
- Minimum setback from building foundations (up gradient): 100 feet.

Because permeable pavements are intended for use in fully stabilized catchments, pretreatment measures are generally not required. An exception is presented if runoff is contributed to the permeable pavement from adjacent rooftops; leaves and debris should be screened before discharge onto the pavement surface.

Following is a list of settings in which permeable pavement can be incorporated to meet more than one project-level or watershed-scale objective:

- Parking lots
- Parking lanes in rights-of-way along roads
- Sidewalks and pedestrian plazas
- Access roads and shoulders

Estimated costs for an average annual maintenance program of a porous pavement parking lot are approximately $200 per acre per year. This cost assumes four inspections each year with appropriate jet hosing and vacuum sweeping treatments.

In addition, permeable pavement areas can be combined with other SCMs to form a treatment train that provides enhanced water quality treatment and reductions in runoff volume and rate. For example, runoff can flow from a roadway to the permeable pavement section and overflow to a bioretention area as shown in Figure B.4.1. Both facilities can be reduced in size according to demonstrated performance for meeting the stormwater runoff requirements and addressing targeted pollutants of concern.



FIGURE B.4.1 PERMEABLE PAVEMENT AND BIORETENTION TREATMENT TRAIN.
Source: Tetra Tech

Step 3. Select Permeable Pavement Surface Course and Aggregate Materials

Several types of permeable pavement are available: pervious concrete, porous asphalt, permeable interlocking concrete pavers, concrete grid pavers, and plastic grid systems, among others. Each type of pavement has advantages and disadvantages, so factors such as cost, pavement use (parking area, driveway, sidewalk, fire lane, and such) and maintenance requirements should be considered on a site-by-site basis. When applicable, follow manufacturers' instructions to ensure a successful implementation.

Pervious concrete and porous asphalt are typically better suited for large parking areas. The advantage to those systems is that the same mixing and application equipment is used as for traditional asphalt and concrete. PICPs, grid pavers, and plastic grid systems are generally better suited to smaller areas because of the labor involved with installation; however, many contractors now employ mechanical placement technologies to expedite the installation of pavers making larger parking areas more feasible. PICP and block pavers can be used for driveways, entryways, walkways, or terraces to achieve a more traditional, formal appearance.

More detailed information for the various types of permeable pavement follow.

Porous Asphalt

Porous asphalt pavement consists of fine- and course-aggregate stone bound by a bituminous-based binder. The amount of fine aggregate is reduced to allow for a larger void space of typically 15 to 20 percent. Because porous asphalt is a hot-mixed pavement, binder temperature performance grade (PG) should be specified on the basis of the anticipated climate to prevent premature failure (melting and sealing) under extreme heat conditions. PG 76-22 liquid asphalt binder is recommended (CAPA n.d.; TxDOT 2004). Thickness of the asphalt depends on the traffic load but usually ranges from 3 to 7 inches. A required underlying base course, typically a washed No. 57 stone, increases storage, and adds strength because porous asphalt is design to be a flexible pavement (Ferguson 2005). A 1- to 2-inch layer of choker course (single-sized crushed aggregate, one-half inch) is typically required to stabilize the surface. Porous asphalt with an aggregate reservoir layer is currently not approved by TCEQ to meet the TSS reduction criteria in the Edwards Aquifer rules (TCEQ 2017).

Porous asphalt can also be installed directly over existing concrete to form a permeable friction course (PFC) overlay. PFCs do not provide the volume storage capacity of porous asphalt systems with reservoir layers, but they can provide excellent water quality improvements in addition to enhanced driver safety (reduced hydroplaning, improved stopping distance, reduced spray, and improved visibility), noise reduction, and improved ride quality (Rand 2006; NCHRP 2009; Eck et al. 2012). PFC overlays have been used with great success on San Marcos and other Texas highways and have been approved to meet the TSS removal rules in the Edwards Aquifer (Rand 2006; TCEQ 2017). A typical porous asphalt profile is shown in Figure B.4.2.

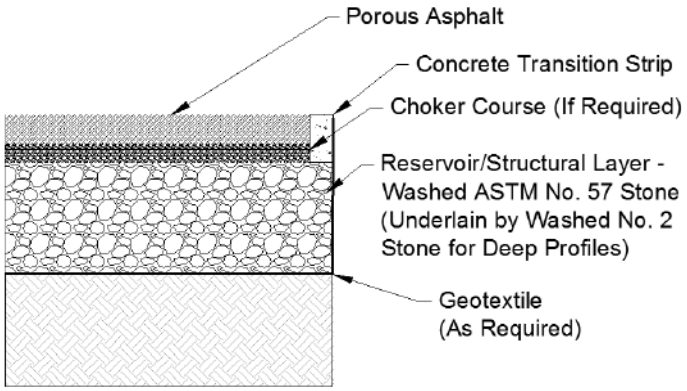


FIGURE B.4.2 TYPICAL POROUS ASPHALT CROSS SECTION.

The properties of porous asphalt depend on the materials used and the compaction procedures. Specified mix design should be in accordance with the National Asphalt Pavement Association (NAPA) *Porous Asphalt Pavements for Stormwater Management* (NAPA 2008). General guidelines are provided below.

Permeability. Typical flow rates for water through porous asphalt range from 150 to 300 in/hr (Roseen and Ballestero 2008). Those values exceed the typical permeability of subsurface soils, so the soils would be the limiting factor.

Aggregates. A typical aggregate size distribution for porous asphalt is below.

Aggregate Gradation Size	Percent Passing
0.75 inch	100%
0.50 inch	85%–100%
0.375 inch	55%–75%
No. 4	10%–25%
No. 8	5%–10%
No. 200	2%–4%

Durability. As with all SCMs, the longevity of porous asphalt (Figure B.4.3) is highly dependent on proper maintenance. Many porous asphalt parking lots have been in service for more than 20 years.



FIGURE B.4.3 EXAMPLE OF POROUS ASPHALT.
San Diego, California Source: Tetra Tech

Pervious Concrete

Pervious concrete is a mixture of Portland cement, fly ash, washed gravel, and water. The water-to-cementitious material ratio is typically 0.35–0.45 to 1 such that the mixture displays a wet metallic sheen without the paste flowing from the aggregate (NRMCA 2004; Barrett 2005). Unlike traditional installations of concrete, permeable concrete usually contains a void content of 15 to 25 percent, which allows water to infiltrate directly through the pavement surface to the subsurface. A fine, washed gravel, less than 13 mm in size (No. 8 or 89 stone), is added to the concrete mixture to increase the void space (GCPA 2006). An admixture improves the bonding and strength of the pavements. The pavements are typically laid with a 4- to 8-inch (10 to 20 cm) thickness over a gravel reservoir (depth varies according to water volume capture requirements), typically

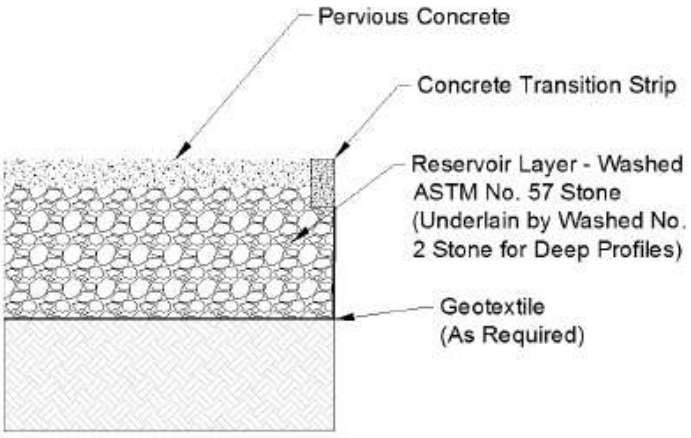


FIGURE B.4.4 TYPICAL PERVIOUS CONCRETE CROSS SECTION.

The properties of pervious concrete (Figure B.4.5) vary with design and depend on the materials used and the compaction procedures. Design mix should conform to the latest version of the American Concrete Institute's ACI 522.1-08 *Specification for Pervious Concrete Pavements.* General guidelines for specifications are provided below.

Permeability. Typical flow rates through well-maintained pervious concrete average greater than 1,500 in/hr (Bean et al. 2007).

Compressive Strength. Pervious concretes can develop compressive strengths in the range of 500 to 4,000 pounds per square inch (psi)—suitable for a wide range of applications.

Flexural Strength. Flexural strength of pervious concrete ranges between 150 and 550 psi. Pervious concrete does not typically incorporate rebar.

Shrinkage. Drying shrinkage of pervious concrete is faster but much less than that experienced with conventional concrete. Pervious concretes should be constructed with control joints to regulate cracking. In general, joints should be cut one-quarter of the pervious concrete thickness, be placed a maximum of 20 feet on centers (15 feet is typical) perpendicular to the curb, and should form square panels.

Abrasion resistance. Because of the rougher surface texture and open structure of pervious concrete, abrasion and raveling of aggregate particles can be a problem. Surface raveling in new pervious concrete can occur when rocks loosely bound to the surface break free under traffic loads. Such raveling is considerably reduced after the first few weeks. Raveling can be reduced by carefully covering the pervious concrete during curing to prevent the surface from drying prematurely. Polypropylene fibers and/or latex can also be added to reduce abrasion resistance (Dong et al. 2010).



FIGURE B.4.5 EXAMPLE OF PERVIOUS CONCRETE.
Kinston, North Carolina Source: NCSU BAE

Permeable Interlocking Concrete Pavement

PICPs are available in many different shapes and sizes. When lain, the blocks form patterns that create openings through which rainfall can infiltrate. Orientation of rectangular pavers is important for structural purposes—*herringbone* patterns tend to provide the most efficient structural design, especially where vehicle stopping and turning are expected. ASTM C936-13 specifications state that the pavers be at least 2.36 inches (60 mm) thick with a compressive strength of 55 MPa (8,000 psi) or greater. Typical installations consist of the pavers and crushed aggregate fill, a 1.5- to 3.0-inch (38 to 76 mm No. 8) fine-aggregate bedding layer, and an aggregate base-course, typically a washed No. 57 stone, storage layer (Smith 2011). If greater storage is required, a reservoir subbase layer of No. 2 stone can be included. More details on PICP can be found in Smith (2011). An example PICP profile is shown in Figure B.4.6.

Unlike permeable concrete and porous asphalt, PICP (Figure B.4.7) is not subject to time and temperature limitations in installation. Interlocking Concrete Pavement Institute (ICPI) standards should be followed during design and construction. Below are listed general specification guidelines.

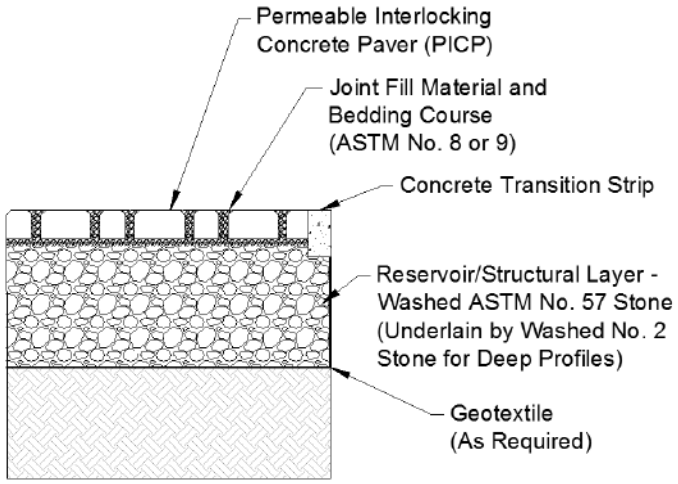


FIGURE B.4.6 TYPICAL PICP CROSS SECTION.



FIGURE B.4.7 EXAMPLE OF PICPS IN A HERRINGBONE PATTERN.
San Marcos, Texas. Source: Tetra Tech.

Permeability. Lifetime infiltration rates on maintained PICP surfaces range from 14 to 4,000 in/hr depending on the joint filling material (Borgwardt 2006; Bean et al. 2007).

Compressive Strength. PICP has a minimum compressive strength of 8,000 psi (55 MPa).

Durability. Regularly maintained permeable pavement systems can last more than 20 years and provide an initial high level of surface infiltration even as the surface takes in moderate amounts of sediment.

Concrete Grid Pavers

Concrete grid pavers (CGPs) conform to ASTM C 1319, *Standard Specification for Concrete Grid Paving Units* which describes paver properties and specifications. CGP units have a minimum thickness of 3.125 inches (80 mm) thick with a maximum 24 × 24 inch (60 × 60 cm) dimension. The percentage of open area ranges from 20 to 50 percent and can contain topsoil and grass, sand, or aggregate in the void space (Figure B.4.8). The minimum average compressive strength of CGP units can be no less than 35 MPa (5,000 psi). A typical installation consists of grid pavers with fill media, 1–1.5 inches (25 to 38 mm) of bedding sand or No. 8 gravel, gravel base course typically consisting of washed No. 57 stone, and a loosely compacted soil subgrade (ICPI 2004). If sand is used, a geotextile should be used between the sand course and the reservoir media to prevent the sand from migrating into the stone media.



FIGURE B.4.8 EXAMPLE OF CGPS WITH VOLCANIC ROCK BEDDING COURSE.
San Diego, California. Source: Tetra Tech.

The ICPI provides design standards for CGP design and installation, but application of CGPs is typically limited to very low traffic areas (such as emergency vehicle access roads or event overflow parking). This limitation is because of differential settling and subsequent *rocking* of pavers that can occur because CGPs (unlike PICP) do not interlock.

Plastic Grid Systems

Plastic grid systems, also called geocells, turf pavers, or turf reinforcing grids, consist of flexible-plastic, interlocking units that allow for infiltration through large gaps filled with gravel or topsoil planted with turf grass. Similar to PICP, a 1–2 inch sand bedding layer and gravel base course are often added to increase infiltration and storage. The empty grids are typically 90 to 98 percent open space, so void space depends on the fill media (Ferguson 2005). To date, no uniform standards exist; however, one product specification defines the typical load-bearing capacity of empty grids at approximately 13.8 MPa (2,000 psi) (Invisible Structures 2001). That value increases up to 38 MPa (5,500 psi) when filled with various materials (Invisible Structures 2001). If sand is used, a geotextile should be used between the sand course and the reservoir media to prevent the sand from migrating into the stone media. Plastic grid systems are currently not approved by TCEQ to meet the TSS reduction criteria in the Edwards Aquifer rules. A typical plastic grid system profile is shown in Figure B.4.9.

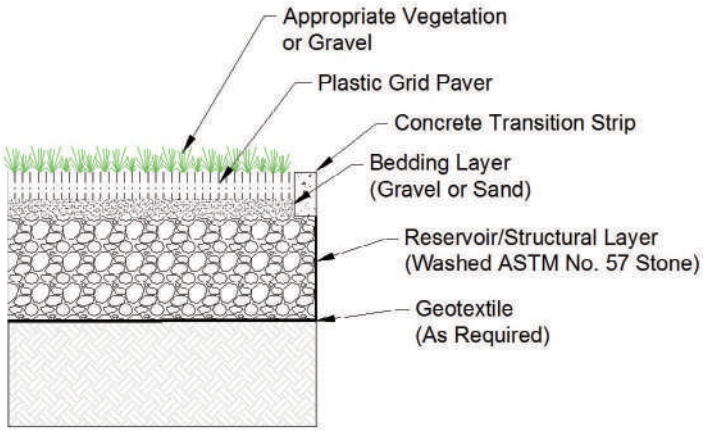


FIGURE B.4.9 TYPICAL PLASTIC GRID SYSTEM CROSS SECTION.

Plastic grids (Figure B.4.10) provide structural support but are generally limited to very low traffic areas such as emergency vehicle access lanes and event overflow parking. They are usually planted with grass. Several companies manufacture plastic grid systems.

Load bearing capacity. Plastic grid systems have a load-bearing capacity up to 6,700 psi when filled (CONTECH 2011).

Durability. Because plastic grid systems are typically manufactured from high-density polyethylene (HDPE), long service lives, up to 50 years, can be expected with proper maintenance.



FIGURE B.4.10 EXAMPLE OF PLASTIC GRID SYSTEMS.
(Left) Carolina Beach, North Carolina. Source: Tetra Tech
(Right) Seguin, Texas. Source: Tetra Tech

Step 4. Determine SCM Function and Configuration

The hydrologic and water quality performance of permeable pavement is largely determined by the drainage configuration. Furthermore, some areas might *not* warrant infiltration. The following design steps can be used to determine the drainage configuration design.

Perform Geotechnical Investigation

Once the appropriate surface course has been selected and discussed with the property owner, the in situ soils must be tested before the system can be sized. Performing soil tests during the conceptual and preliminary design phases will ensure that the proposed permeable pavement system is optimized to actual site conditions and to prevent costly change orders resulting from poorly estimated soil parameters.

A geotechnical investigation should be performed by a licensed soil scientist or geotechnical engineer. All soil testing should be performed at the depth of the initially proposed subgrade because this is the soil strata where infiltration might occur. If a detention (non-infiltrating system) is proposed, soil tests must still be performed to determine structural requirements and to identify the elevation of the seasonal high water table. As part of the site evaluation take soil boring to a depth of at least 4 feet below the bottom of the stone reservoir to check for soil permeability, porosity, depth of seasonally high water table, and depth to bedrock. For details on geotechnical analyses, see Common Design Elements.

Determine if Underdrains and Impermeable Liners are Required

On the basis of the infiltration rate measured in the previous step, the drawdown time of the system at full capacity should be calculated. If the infiltration rate of the soils on which the permeable pavement area will be installed is less than 0.5 in/hr, underdrains will be required (as described in Table B.4.3). The underdrains can be embedded in the aggregate reservoir layer or in a gravel trench below the reservoir layer, as shown in Figure B.4.11. For information on designing an underdrain system, see Common Design Elements. IWS should be included in all infiltrating systems to enhance infiltration (Wardynski et al. 2013). The elevation of the upturned underdrain outlet dictates the volume of water retained in the profile, which should be greater than or equal to the water quality volume (as determined in step 5). An example permeable pavement profile containing IWS is provided in Figure B.4.12.

TABLE B.4.3 DECISION TABLE FOR DETERMINING UNDERDRAIN AND IMPERMEABLE LINER REQUIREMENTS

IMPERMEABLE LINERS MUST BE USED IF...	UNDERDRAINS MUST BE USED IF...
• Site is in Edwards Aquifer Recharge Zone or Transition Zone • Soil contamination is expected or present • Karst geology presents risk of sinkhole formation • Runoff may unintentionally be received from a stormwater hotspot • Site is within 100 feet of a water supply well or septic drain field • Site is within 10 feet of a structure/foundation • Infiltrated water may interfere with utilities	• An impermeable liner is needed • Infiltration rate of underlying soils is less than 0.5 in/hr • Site is within 50 feet of a steep, sensitive slope

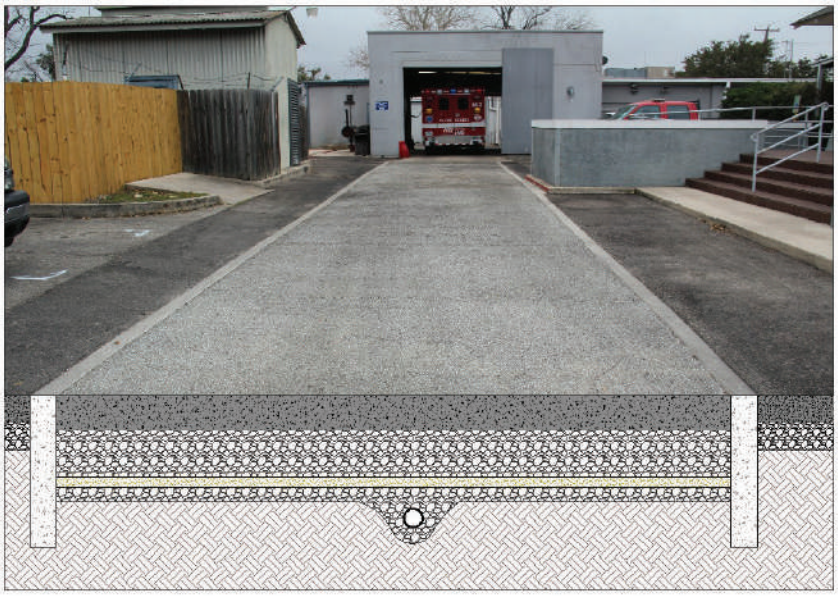


FIGURE B.4.11 EXAMPLE OF PERMEABLE PAVEMENT WITH TRENCHED UNDERDRAIN.
Alamo Heights Fire Station, Alamo Heights, Texas. Source: Tetra Tech

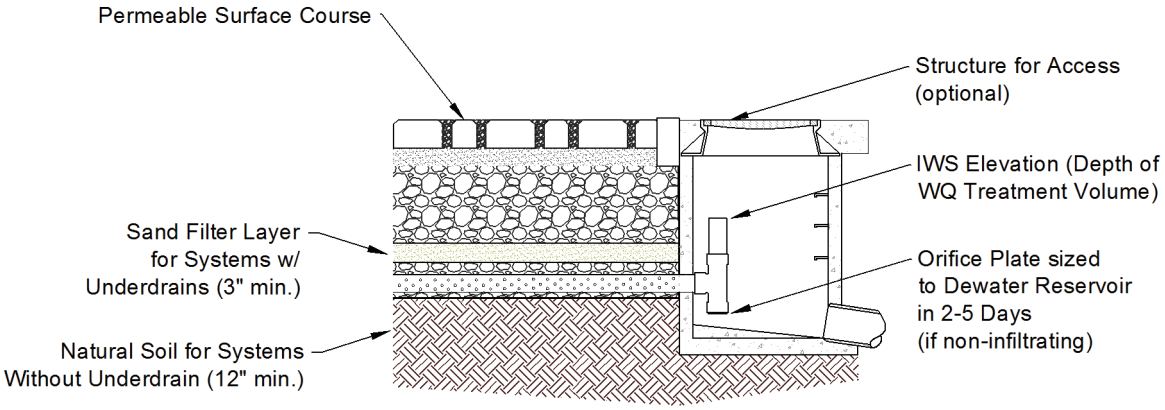


FIGURE B.4.12 EXAMPLE PERMEABLE PAVEMENT PROFILE FEATURING IWS.

If infiltration is disallowed, the system should be lined with a hydraulic restrictive layer. Factors prescribing an impermeable liner are provided in Table B.4.3. Non-infiltrating systems are also known as detention systems and can be designed similar to other detention structures. Outflow should be regulated in accordance with water quality (releasing water over the course of 2 to 5 days) and flood control requirements for detention structures (discussed in Step 6).

Observation Wells

Design drawings should specify installation of observation wells to monitor the drawdown rate of permeable pavement reservoir layers. Wells should be constructed of perforated PVC pipe (4-inch diameter or greater) and should be designed to prevent damage from vehicular traffic. If necessary, observation wells can be installed at an angle and daylight in adjacent landscape areas (as long as the

well extends the full depth of the reservoir layer).Wells should be securely sealed with watertight caps. Figure B.4.13 provides examples of observation wells installed in permeable pavement applications.



FIGURE B.4.13 OBSERVATION WELL INSTALLED IN PERMEABLE PAVEMENT.
Source: NCSU-BAE

Design Subgrade Slope and Specify Geotextile

The subgrade slope should not exceed 0.5 percent to ensure that the design volume is captured and evenly distributed and to maintain structural integrity. Baffles can be installed along the subgrade to provide grade control if necessary. In fully lined systems, a drawdown orifice should be provided in each baffle to allow dewatering between storm events.

A geotextile should be placed beneath the reservoir media and along the perimeter of the cut in all infiltrating systems. A needled, non-woven, polypropylene geotextile conforming to the specifications in Table B.4.4 should be specified. It is important to line the entire trench area, including the sides, with a geotextile before placing the aggregate. The geotextile serves an important function by inhibiting soil from migrating into the reservoir layer and reducing storage capacity.

TABLE B.4.4 GEOTEXTILE LAYER SPECIFICATIONS

GEOTEXTILE PROPERTY	VALUE	TEST METHOD
Grab tensile strength (lbs)	≥ 120	ASTM D4632
Mullen burst strength (lbs/sq. in.)	≥ 225	ASTM D3786
Permeability (gpm/sq. ft.)	≥ 125	ASTM D4491
Apparent opening size (sieve size)	#70–#80 (min)	ASTM D4751

The geotextile apparent opening size selection is based on the percent passing the No. 200 sieve in A Soil subgrade, using FHWA or AASHTO selection criteria.

Step 5. Design the Profile

The permeable pavement profile must be designed to capture the water quality treatment volume and filter it through soil or a sand filter layer. In fully lined (non-infiltrating systems), the treatment volume should ideally be detained for 2 to 5 days. Additionally, the profile must provide structural support for the anticipated vehicular loading.

Temporary Surface Ponding Depth (Edwards Aquifer Zones)

When permeable pavement is used in the Edwards Aquifer protection zones, surface ponding must be provided (by curb and gutter) such that the design storm volume will be retained onsite if the permeable pavement surface clogs (Barrett 2005). Curb edging and driveways should be elevated such that the design water quality volume ponds on the surface and does not flow offsite. This is not typically a requirement outside the Edwards Aquifer protection zones.

Specify Sand/Soil Filter Layer

Percolating runoff through native soils is the most effective way to improve water quality. When no underdrains are required (when subsoil infiltration rates greater than 0.5 in/hr), a minimum of 12 inches of native soil should be provided at the subgrade to filter captured stormwater before infiltration (for soil specifications, see Common Design Elements). If underdrains are used, or if subsoils are not suitable for stormwater filtration, a minimum of 4 inches of ASTM C-33 washed sand should be included above the aggregate of the underdrain drainage layer. A layer of choking stone might be needed between the sand filter layer and the gravel drainage layer, as discussed in Common Design Elements.

Calculate Surface Area and Reservoir Media Depth

The aggregate base course should be designed to store at a minimum the water quality treatment volume determined in Appendix E. For infiltrating systems, this volume should be retained in the profile using IWS (as described in step 4). The stone aggregate used should be washed, angular, crushed stone, 0.75 to 2.5 inches in diameter with a void space of about 40 percent (No. 57 stone). ASTM No. 2 stone may be used as a subbase layer below the base course for additional storage. Aggregate contaminated with soil and typical *crusher run* stone should not be used because those materials will clog the pores at the bottom of the pavement.

If the area of permeable pavement is known, the following equation can be used to determine the depth of storage layer (aggregate base course) needed to capture the water quality treatment volume:

$$d = \frac{V}{An} \qquad \text{[Equation B.4.1]}$$

where

d = aggregate layer depth (ft)

V = water quality volume (ft^3)

A = surface area (square ft)

n = porosity (use actual laboratory measured porosity of material)

Structural Design Requirements

If permeable pavement will be used in a parking lot or other setting that involves vehicles, the pavement surface must be able to support the maximum anticipated traffic load. The structural design process will vary according to the type of pavement selected, and the manufacturer's specific recommendations should be followed. The thickness of the permeable pavement and reservoir layer must be sized to support structural loads and to temporarily store the design storm volume (e.g., the water quality, channel protection, and flood-control volumes). On most new development and redevelopment sites, the structural support requirements will dictate the depth of the underlying stone reservoir.

The structural design of permeable pavements involves considering four main site elements:

- Total traffic
- In situ soil strength
- Environmental elements
- Bedding and reservoir layer design

The resulting structural requirements can include the thickness of the pavement, filter, and reservoir layer. Designers should note that if the underlying soils have a low California Bearing Ratio (less than 4 percent), the soil might need to be compacted to at least 95 percent of the Standard Proctor Density, which generally rules out their use for infiltration.

Designers should determine structural design requirements by consulting transportation design guidance sources, such as the following:

- AASHTO Guide for Design of Pavement Structures (1993)
- AASHTO Supplement to the Guide for Design of Pavement Structures (1998)

Step 6. Design for Safe Bypass/Conveyance of Large Storm

Permeable pavement systems, as with any other stormwater SCM, must be designed to safely route runoff in excess of the intended design flow. The method of large-storm routing is largely site specific and depends on the type of permeable surface course. When poured in place, surface courses are used (pervious concrete or porous asphalt), volume in excess of the system storage capacity can be allowed to bubble up through the profile and run off the site safely as surface flow. Catch basins or slot drains could be installed around the perimeter of the permeable pavement to drain any overflow; inlets can be specified slightly above the elevation of the finished surface to allow some surface ponding, if allowable.

Modular paving systems (PICP, CGP, or plastic grid systems) should *not* be designed to overflow in this manner, however, because upflowing water could dislodge and carry away aggregate from the bedding course. When surface overflow is not a feasible or preferred option, the system can be designed to (1) completely store the 25-year storm volume in the aggregate reservoir and exfiltrate into underlying soils, (2) convey larger storms safely through the system using underdrains (equipped with orifices, if required), or (3) use other internal controls to allow bypass of larger storms. Large storm routing can be designed to satisfy detention requirements, per local requirements.

Step 7. Edge Restraints and Intersections

Providing separation between permeable pavements and adjacent impermeable surfaces serves multiple purposes, including the following:

1. Clearly identifying for maintenance personnel the transition between permeable and impermeable surfaces
2. Restraining modular (block) pavers and porous asphalt to prevent lateral shifting or unraveling of edges
3. Creating a hydraulic restriction layer to prevent lateral seepage of runoff below adjacent pavements and structures
4. Delineating parking zones with clean, aesthetically pleasing lines

Restraints for flexible pavements are typically composed of standard concrete curbs (elevated or at grade, depending on application) or specially designed monolithic concrete walls. At intersections between permeable and impermeable surfaces, a hydraulic restriction layer (typically a geomembrane) is installed along the entire length of the cut and at least 2 feet laterally along the subgrade and under the impermeable surface. Figure B.4.14 shows an example of an edge restraint.



FIGURE B.4.14 AN 8-INCH CONCRETE TRANSITION STRIP IS USED AS AN EDGE RESTRAINT BETWEEN PICP AND IMPERMEABLE ASPHALT.
San Antonio, Texas, Source: Tetra Tech.

Step 8. Design Signage

It is good practice to specify signage on engineering plans; signage educates the public and identifies permeable pavements to maintenance personnel. Prohibited practices, such as stockpiling soils or mulch, should be clearly displayed to protect permeable pavements from premature clogging. Signage will also prevent poured in place permeable pavements from being mistaken as impermeable and then paved over during repair.

Step 9. Design for Multi-Use Benefits

Permeable pavements inherently provide multi-use benefits because the facilities double as parking lots and transportation corridors. In addition to these benefits, permeable pavement can be enhanced by incorporating the following design elements:

5. Enhanced pavement textures, colors, and patterns can calm traffic, increase aesthetic appeal, enhance pedestrian safety, and draw attention to multi-use stormwater practices.
6. Stormwater reuse systems can be installed to harvest and use captured runoff for non-potable use (irrigation, ornamental water features, and such).
7. Permeable pavers can be used to maintain the character of historic districts while providing stormwater management solutions.
8. Educational kiosks and signage raise public awareness of stormwater issues.

B.4.4: CONSTRUCTION CONSIDERATIONS

Notes on construction plans should specify that tracked vehicles (versus wheeled vehicles) be used whenever practicable to minimize compaction of subsoils. Construction specifications should also include notes requiring the testing of subgrade infiltration rates before installing aggregate (for infiltrating systems). This step ensures that captured water will draw down in the required duration. If subgrade infiltration rates are drastically lower than design values, the subgrade should be treated by scarifying, ripping, or trenching according to the recommendations in bioretention. If infiltration rates remain lower than required, the profile depth must be changed to provide additional storage or the drainage configuration must be altered to regulate the drawdown.

Careful inspection of several construction steps can prevent costly errors. Construction of permeable pavement systems should be performed only by a contractor with experience in permeable pavement installation and that is certified by the Interlocking Concrete Pavement Institute or the National Ready Mix Concrete Association. Lists of certified contactors are at http://www.icpi.org or http://www.nrmca.org. In addition to the general considerations available online, the following practices should be completed by the designer or a trained inspector.

B.4.4.1: Inspect Aggregate upon Delivery

Stone aggregate bedding, base, and subbase courses should be thoroughly washed to prevent fines from clogging the subsoil interface or underdrains (Fassman and Blackbourne 2010). Before placement, the furnished aggregate should appear free of fines and leave no substantial dust on the skin when handled. Unwashed aggregate should be replaced or washed onsite using proper construction site sediment control practices.

B.4.4.2: Inspect Elevations

Elevation discrepancies during grading or placing pipe inverts can result in undersized (and underperforming) systems. Verifying the average subgrade elevation and the elevation of the outlet invert will help ensure that the specified reservoir storage volume has been provided.

B.4.4.3: Test Actual Subgrade Infiltration Rate

After excavation and before installing aggregate, the actual in situ infiltration rate should be measured using the methods in Common Design Elements. This is a critical step to determine the level of compaction experienced during construction so that adequate mitigation practices can be recommended.

B.4.4.4: Mitigate Soil Compaction to Enhance Exfiltration

If exfiltration rates (as determined in previous step) are substantially lower than design values, the subgrade should be treated according to Table B.4.5 to mitigate compaction. If subgrade exfiltration rates are substantially lower than original design rates, it may be necessary to provide additional aggregate reservoir depth to accommodate storage and exfiltration of subsequent rainfall events.

TABLE B.4.5 SUBGRADE TREATMENTS FOR INFILTRATION ENHANCEMENT

SUBGRADE COMPACTION	MINIMUM SUBGRADE TREATMENT	SPECIFICATION
Low	Scarification	Loosen the top 6 to 9 inches of subgrade using the teeth of an excavator bucket (or comparable). This can be achieved by excavating the final 1 foot using a toothed bucket.
Low-Medium	Ripping	Using a subsoil ripper or metal bar, rip the subgrade to a depth of 9 to 12 inches, every 3 feet (on center). When operating in silty, loamy, or clay soils, fill ripped areas with coarse sand to maintain free-flowing trenches.
High	Trenching	Excavate 1-foot-deep by 1-foot wide trenches into the subgrade, every 6 feet (on center). Fill the bottom of the trench with one-half inch of coarse sand, and top off trench with washed aggregate (No. 57 stone or comparable).

Source: Based on findings in Tyner et al. 2009 and Brown and Hunt 2010

B.4.4.5: Inspect Surface Course Placement and Curing

Poured in place surface courses should be inspected during placement to ensure proper mix characteristics. After screeding and compaction, inspectors should ensure that the surface of pervious concrete is not smeared (particularly when placing plastic over the surface for curing).

B.4.5: INSPECTION CRITERIA

To maintain the effectiveness of structural SCMs, regular inspection is essential. Generally, SCM inspection and maintenance can be categorized as routine and as needed. Routine activities, performed regularly (e.g., monthly) ensure that the SCM is in good working order and continues to be aesthetically pleasing. Routine inspection is an efficient way to prevent potential nuisance situations from developing and reduce the need for repair or maintenance. Routine inspection also reduces the chance of degrading the quality of the effluent by identifying and correcting potential problems regularly. Property maintenance personnel should be instructed to inspect SCMs during their normal routines.

In addition to routine inspections, as-needed inspection/maintenance of all SCMs should be performed after any event or activity that could damage the SCM, particularly after every large storm event. Post-storm inspections should occur after the expected drawdown period for the SCM, when the inspector can determine if the SCM is draining correctly.

In general, individual SCMs can be described with minimum performance expectations, design criteria, structural specifications, date of implementation, and expected life span. Recording such information will help the inspector determine whether a SCM's maintenance schedule is adequate or requires revision and will allow comparison between the intended design and the as-built conditions. Checklists also provide a useful way for recording and reporting whether major or minor renovation or routine repair is needed. The effectiveness of a SCM might be a function of the SCM's location, design specifications, maintenance procedures, and performance expectations. Inspectors should be familiar with the characteristics and intended function of the SCM so they can recognize problems and know how they should be resolved.

Routine and as-needed SCM inspections consist of technical and non-technical activities as summarized below:

- Inspect the general conditions of the SCM and areas directly adjacent.
- Maintain access to the site including the inlets, side slopes (if applicable), forebay (if one exists), SCM area, outlets, emergency spillway, and so on.
- Examine the overall condition of vegetation.
- Eliminate any possibility of public hazards (vector control, unstable public access areas).
- Check the conditions of inflow points, pretreatment areas (if they exist), and outlet structures.
- Inspect and maintain the inlet and outlet regularly and after large storms.
- Ensure that the pretreatment areas meet the original design criteria.
- Check the encroachment of undesirable plants in vegetated areas. This could require more frequent inspections in the growing season.
- Inspect water quality improvement components. Specifically, check the stormwater inflow, conveyance, and outlet conditions.
- Inspect hydrologic functions such as maintaining sheet flow where designed, ensuring functional pretreatment, maintaining adequate design storage capacity, and verifying proper operation of outlet structures.
- Check conditions downstream of the SCM to ensure that flow is properly mitigated below the facility (e.g., excessive erosion, sedimentation).

In every inspection, whether routine or as needed, the inspector should document whether the SCM is performing correctly and whether any damage has occurred to the SCM since the last inspection. Ideally, the inspector will also identify what should be done to repair the SCM if damage has occurred. Documentation is very important in maintaining an efficient inspection and maintenance schedule, providing evidence of ongoing inspection and maintenance, and detecting and reporting any necessary changes in overall management strategies.

Inspection frequency for bioretention facilities is included in Table B.4.6 in Section B.4.6.

Table B.4.6 Operation and Maintenance Tasks for Permeable Pavement

Task	Frequency	Indicator maintenance is needed	Maintenance notes
Catchment inspection	Weekly or biweekly during routine property maintenance	Sediment accumulation on adjacent impervious surfaces or in voids/ joints of permeable pavement	Stabilize any exposed soil and remove any accumulated sediment. Adjacent pervious areas might need to be graded to drain away from the pavement.
Miscellaneous upkeep	Weekly or biweekly during routine property maintenance	Trash, leaves, weeds, or other debris accumulated on permeable pavement surface	Immediately remove debris to prevent migration into permeable pavement voids. Identify source of debris and remedy problem to avoid future deposition.
Preventative vacuum/ regenerative air street sweeping	Twice a year in higher sediment areas	N/A	Pavement should be swept with a vacuum power or regenerative air street sweeper at least twice per year to maintain infiltration rates.
Replace fill materials	As needed	For paver systems, whenever void space between joints becomes apparent or after vacuum sweeping	Replace bedding fill material to keep fill level with the paver surface.
Restorative vacuum/ regenerative air street sweeping	As needed	Surface infiltration test indicates poor performance or water is ponding on pavement surface during rainfall	Pavement should be swept with a vacuum power or regenerative air street sweeper to restore infiltration rates.

B.4.6: RECOMMENDED MAINTENANCE

Maintenance of permeable pavement systems is critical to the overall and continued success of the system. Specific maintenance activities are listed in Table B.4.6. Key maintenance procedures consist of the following:

1. Adjacent areas that drain to the permeable pavement area should be permanently stabilized and maintained to limit the sediment load to the system.
2. Vacuum sweeping should be typically performed a minimum of four times per year. Adjust the frequency according to the intensity of use and deposition rate on the permeable pavement surface.
3. Any weeds that grow in the permeable pavement should be sprayed with pesticide immediately. Weeds should not be pulled, because doing so can damage the fill media.
4. Mowing and trimming turf grass used with permeable pavers and plastic grid systems must be performed regularly according to site conditions. Grass should be mowed at least once a month in the growing season. All vegetated areas must be inspected at least annually for erosion and scour.

B.4.7: REFERENCES

American Association of State Highway and Transportation Officials (AASHTO). 1993. *Guide for Design of Pavement Structures, 4th Edition*. Washington, DC.

American Association of State Highway and Transportation Officials (AASHTO). 1998. Supplement to *Guide for Design of Pavement Structures, 4th Edition.* Washington, DC.

ASTM (American Society for Testing and Materials) C936-13. http://www.astm.org/Standards/C936.htm

Barrett, M.E. 2005. *Complying with the Edwards Aquifer Rules. Technical Guidance on Best Management Practices*. RG-348. Prepared for Texas Commission on Environmental Quality, Field Operations Division, Austin, TX.

Bean, E.Z., W.F. Hunt, and D.A. Bidelspach. 2007. Field survey of permeable pavement surface infiltration rates. *Journal of Irrigation and Drainage Engineering* 133(3):247–255.

Borgwardt, S. 2006. Long-term in situ infiltration performance of permeable concrete block pavement, In *Proceedings of the 8th International Conference on Concrete Block Paving*, Interlocking Concrete Pavement Institute, Washington, DC.

Brown, R.A., and W.F. Hunt. 2010. Impacts of construction activity on bioretention performance. *Journal of Hydrologic Engineering* 15(6):386–394.

CAPA (Carolina Asphalt Paving Association). n.d. *Porous Paving Parking Lots Guide Specifications*. Raleigh, NC.

Collins, K.A., W.F. Hunt, and J.M. Hathaway. 2010. Side-by-side comparison of nitrogen species removal for four types of permeable pavement and standard asphalt in eastern North Carolina. *Journal of Hydrologic Engineering* 15(6):512-521.\

Contech. 2011. Urban Green Grass Pavers. Contech Construction Products, Inc. Scarborough, ME.

Dong, Q., H. Wu,B. Huang, and S. Xiang. 2010. D*evelopment of a Simple and Fast Test Method for Measuring the Durability of Portland Cement Pervious Concrete*. SN3149, Portland Cement Association, Skokie, IL.

Eck, B.J., R.J. Winston, W.F. Hunt, and M.E. Barrett. 2012. Water Quality of Drainage from Permeable Friction Course. *Journal of Environmental Engineering* 138(2):174–181.

Fassman, E.A., and S.D. Blackbourn. 2010. Urban runoff mitigation by a permeable pavement system over impermeable soils. *Journal of Hydrologic Engineering* 15(6):475–485.

Fassman, E.A., and S.D. Blackbourn. 2011. Road runoff water-quality mitigation by permeable modular concrete pavers. *Journal of Irrigation and Drainage* 137(11):720–729.

Ferguson, B.K. 2005. *Porous Pavements*. CRC Press, Boca Raton, FL.

GCPA (Georgia Concrete and Products Association). 2006. *Guide for Construction of Portland Cement Concrete Pervious Pavement*. Georgia Concrete and Products Association, Tucker, GA. <http://www.gcpa.org/>. Accessed June 23, 2010.

Invisible Structures. 2001. *Grasspave2TM Technical Specifications*. Invisible Structures, Inc., Golden, CO.

ICPI (Interlocking Concrete Pavement Institute). 2004. *Tech Spec 8, Concrete Grid Pavements*. Interlocking Concrete Pavement Institute, Washington, DC.

National Asphalt Pavement Association (NAPA). 2008. *Porous Asphalt Pavements for Stormwater Management. Design, Construction and Maintenance Guide.* Information Series 131. Lanham, MD.

NCHRP (National Cooperative Highway Research Program). 2009. *Construction and maintenance practices for permeable friction courses.* Rep. 640, Transportation Research Board, Washington, DC.

NRMCA (National Ready Mixed Concrete Association). 2004. *CIP 38 – Pervious Concrete. Concrete in Practice*. National Ready Mixed Concrete Association, Silver Spring, MD. <http://www.nrmca.org/GreenConcrete/CIP%2038p.pdf>. Accessed June 23, 2010.

Roseen, R.M., and T.P. Ballestero. 2008. Porous asphalt pavements for stormwater technology. *Hot Mix Asphalt Technology* pp. 26–34.

Smith, D. 2011. *Permeable Interlocking Concrete Pavements.* Fourth Edition. Herndon, VA.

TCEQ (Texas Commission on Environmental Quality). 2012. *Addendum Sheet to Complying with the Edwards Aquifer Rules. Technical Guidance on Best Management Practices*. RG-348 (Revised July 2005). July 5, 2012. Austin, TX.

TxDOT (Texas Department of Transportation. 2004. *Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges.* June 1, 2004. Austin, TX.

Tyner, J.S., W.C. Wright, and P.A. Dobbs. 2009. Increasing exfiltration from pervious concrete and temperature monitoring. *Journal of Environmental Management* 90:2636–2641.

Wardynski, B.J., R.J. Winston, and W.F. Hunt. 2013. Internal water storage enhances exfiltration and thermal load reduction from permeable pavement in the North Carolina mountains. *Journal of Environmental Engineering* 139(2):187–195.

B.5: RAINWATER HARVESTING



RAINWATER CATCHMENT SYSTEM, MCKENNA CENTER, NEW BRAUNFELS, TEXAS. SOURCE: TETRA TECH

B.5.1: DESCRIPTION

Cisterns or their smaller counterpart, rain barrels, are containers that capture runoff and store it for future use. With control of the timing and volume, the captured stormwater can be more effectively released for irrigation or alternative grey water uses between storm events. Rain barrels tend to be smaller systems, less than 100 gallons. Cisterns are larger systems that can be self-contained aboveground or belowground systems generally larger than 100 gallons. Below ground systems often require a pump for water removal. For San Marcos and surrounding areas, cisterns and rain barrels primarily provide control of stormwater volume; however, water quality improvements can be achieved when cisterns and rain barrels are used in a treatment train with SCMs such as bioretention areas. Water in cisterns or rain barrels can be controlled by permanently open outlets or operable valves depending on project specifications. Cisterns and rain barrels can be a useful method of reducing stormwater runoff volumes in urban areas where site constraints limit the use of other SCMs. Advantages and limitations of rainwater harvesting are outlined in Table B.5.1.

TABLE B.5.1 ADVANTAGES AND LIMITATIONS OF RAINWATER HARVESTING

ADVANTAGES	LIMITATIONS
• Provides peak flow mitigation for frequent and infrequent storm events • Aids in infiltration by delaying runoff • Variable configurations to meet site constraints • Can reduce the size of infiltration SCMs • Can be designed for high visibility to raise stormwater awareness or can be hidden from view • Effective where underground utilities or other constraints preclude use of surface/subsurface storage SCMs • Can be designed to supplement or replace nonpotable water supplies (for nonresidential uses) or for irrigation (residential or nonresidential) • Rainwater harvesting equipment is exempt from sales tax under Texas Tax Code 151.355	• Requires regular maintenance of inlet filters and mosquito control screens • Can require structural support • Reuse systems may require filtration and disinfection per intended use and local plumbing codes

B.5.2: SELECTION CRITERIA

Rainwater harvesting systems are typically placed near roof downspouts such that flows from existing downspouts can be easily diverted into the cistern. Runoff enters the cistern near the top and is filtered to remove large sediment and debris. Collected water exits the cistern from the bottom or can be pumped to areas more conducive for infiltration. Cisterns can be used as a reservoir for temporary storage or as a flow-through system for peak flow control. Cisterns are fitted with a valve that can hold the stormwater for reuse, or they release the stormwater from the cistern at a rate below the design storm rate. Regardless of the intent of the storage, an overflow must be provided if the capacity of the cistern is exceeded. The overflow system should route the runoff to a SCM for treatment or safely pass the flow into the stormwater drainage system. The overflow should be conveyed away from structures. The volume of the cistern should be allowed to slowly release, preferably into a SCM for treatment or into a landscaped area where infiltration has been enhanced.

Rainwater harvesting systems must be located downstream of a rooftop or other impervious surface. They must be located on stable ground and must not interfere with a building functionality, or with any other SCMs.

B.5.3: DESIGN SPECIFICATIONS

The design of a cistern or rain barrel can be broken down into an eight-step process, as listed in Table B.5.2. Additional resources are provided in TCEQ (2011), Texas Water Development Board (2005), and Texas A&M AgriLife Extension Services (2013).

Step 1. Determine the Volume of Water or Flow to Treat

The volume of water to be treated will help managers determine the appropriate cistern size and configuration. Methods for calculating the volume required for treatment are outlined in Appendix E. The treatment volume must be treated on-site and can be treated by multiple SCMs. Cisterns will typically be part of a treatment system that would include cisterns and other SCMs including bioretention or pervious pavement. The cistern could be included to reduce the size of another SCM. Peak runoff flow rates should also be calculated using the methods in Appendix E such that pipes can be sized accordingly to allow overflow or bypass of the 100-year peak discharge.

Step 2. Determine SCM Configuration

Cisterns are available commercially in numerous sizes, shapes, and materials. Many are made to custom fit the available space and can be short and wide, tall and narrow, round, rectangular, and almost any size imaginable. They can be made from multiple materials but are primarily constructed of plastic or metal. Plastic cisterns can be covered with wood facades to provide a more finished appearance or can be painted with any image desired.

TABLE B.5.2 ITERATIVE DESIGN STEP PROCESS

DESIGN STEP		DESIGN COMPONENT/ CONSIDERATION	GENERAL SPECIFICATION
1	**Determine SCM Size**	Use Appendix E	
2	**Determine SCM Configuration**	Based on volume and desired alternative uses, incorporate next to buildings or underground. A foundation of gravel should be provided if the weight of the cistern at capacity is less than 2,000 pounds, otherwise a concrete foundation should be provided.	
3	**Select and Size Inlet Configuration**	Conveyance type	Runoff should be conveyed to the cistern such that no backwater onto roofs occurs during the 100-year event. Two types of inlet configurations are available: • Dry conveyance: conduit freely drains to cistern with no water storage in pipe • Wet conveyance: a bend in the conduit retains water between rainfall events
4	**Design Inlet Pretreatment Configuration**	Inlet filter	A self-cleaning inlet filter should be provided to strain out large debris such as leaves. Some systems incorporate built-in bypass mechanisms to divert high flows.
		First flush diverter	A passive first flush diverter should be incorporated in areas with high pollutant loads to capture the first washoff of sediment, debris, and pollen during a rainfall event. First flush diverters are typically manually dewatered between events.
5	**Select and Size Appropriate Outlet and Overflow/ Bypass Method**	Low-flow outlet	An outlet should be designed to dewater the water quality storage volume to a vegetated area in 2 days minimum. The elevation of the outlet depends on the volume of water stored for alternative purposes.
		Overflow or bypass	Emergency overflow (set slightly below the inlet elevation) or bypass must be provided to route water safely out of the cistern when it reaches full capacity.
6	**Specify Cautionary Signage, Pipe Color, and Locking Features**	Signage	Signage indicating: "Caution: Reclaimed Water, Do Not Drink" (preferably in English and Spanish) must be provided anywhere cistern water is piped or outlets.
		Pipe color and locking features	All pipes conveying harvested rainwater should be Pantone color #512 and be labeled as reclaimed water. All valves should feature locking features.
7	**Design for Multi-Use Benefits**	Harvested rainwater should be used to offset potable water uses, such as irrigation, toilet flushing, car washing, etc. Additionally, educational signage and aesthetically pleasing facades should be specified.	
8	**Additional Design Specifications**	Vector control	All inlets and outlets to the cistern must be covered with a 1-mm or smaller mesh to prevent mosquito entry/egress
		Routing water for use	Regardless of gravity or pumped flow, adequate measures must be taken to prevent contamination of drinking water supplies
		Makeup water supply	A makeup water supply can be provided to refill the cistern to a desired capacity when harvested water has a dedicated use.
		Cistern material	Tanks should typically be opaque to prevent algal growth.

Cisterns are usually intended to capture runoff from elevated surfaces, such as rooftops, and, therefore, must be next to structures where runoff can be collected. Cisterns are typically designed to capture runoff from concentrated sources or collection systems such as a downspout. Multiple cisterns placed around a structure can be hydraulically connected to take advantage of maximum storage capacity. The typical components of a cistern are shown in Figure B.5.1.

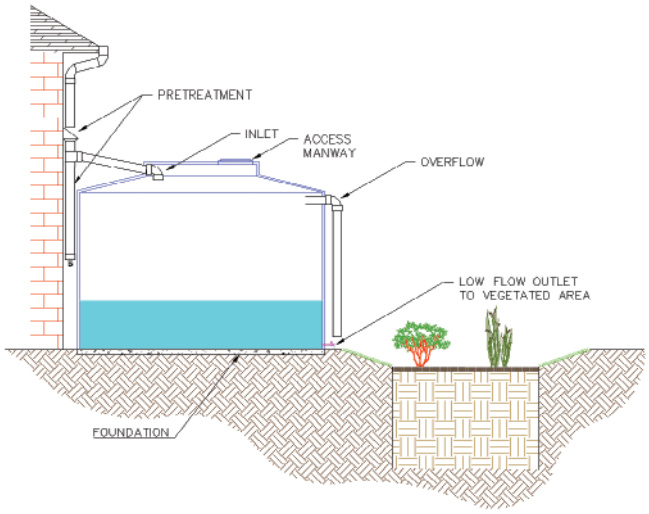


FIGURE B.5.1 MINIMUM DESIGN COMPONENTS OF RAINWATER HARVESTING SYSTEM.

The conditions or layout of the site could determine if the foundation can be excavated and what materials will be used to support the cistern. Cisterns, especially large systems, must have a proper foundation to support the weight when they are at capacity. Two options exist for foundations (Jones and Hunt 2008):

- **Cisterns exerting less than 2,000 pounds per square foot:** The foundation of the cistern should be cleared and leveled. The foundation should be at least 6 inches of No. 57 gravel or concrete, depending on the stability of the underlying soils.
- **Cisterns exerting greater than 2,000 pounds per square foot:** The area beneath the cistern should be cleared and leveled. Concrete should be poured such that gravity flow can be maintained and the cistern can be drained to the level of the outlet valve.

The threshold where a concrete pad is required will vary depending on the soil type. If the structural capacity of the site to support a full cistern is in doubt, a geotechnical evaluation should be performed to determine the structural capacity of the soils. Figures B.5.2 to Figure B.5.4 show the foundation options.



FIGURE B.5.2 CISTERN LESS THAN 2,000 PSI ON A GRAVEL FOUNDATION.
Fort Davis, Texas. Source: Tetra Tech



FIGURE B.5.3 CISTERN GREATER THAN 2,000 PSI ON A CONCRETE FOUNDATION.
Source: Tetra Tech



FIGURE B.5.4 CISTERN ON A CONCRETE FOUNDATION.
Hardberger Park, San Antonio, Texas. Source: Tetra Tech

Step 3. Select and Size Inlet Configuration

Inlet connections can feature either dry conveyance or wet conveyance. The following subsections describe each configuration.

Dry Conveyance

When downspouts freely drain to the cistern without any trapped water, the system uses dry conveyance. Connections can be made through the top of the cistern as shown in Figure B.5.5 and Figure B.5.6 or through the sides of the vertical portion formed for the opening of the cistern, often referred to as the manway, as shown in Figure B.5.7. Inlet connections made through the top of the cisterns can also include a basket filter as an inlet filter option. Inlet connections through the sides with the proper gaskets are recommended for ease of maintenance and access to the cistern.

When designing dry conveyance, downspout pipes should be sized to convey the 100-year discharge without causing any backwater on the roof.

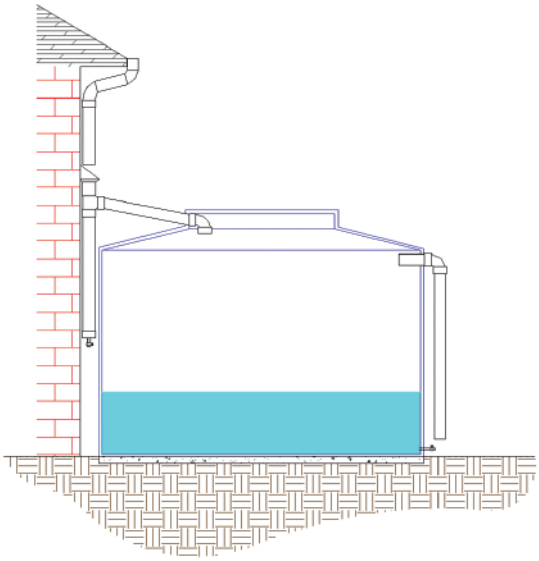


FIGURE B.5.5 DRY CONVEYANCE INLET CONFIGURATION.



FIGURE B.5.6 INLET IN THE TOP OF THE CISTERN.
Texas A&M University at San Antonio, San Antonio, Texas. Source: San Antonio River Authority



FIGURE B.5.7 INLET IN THE SIDES OF THE MAN WAY.
Greensboro, North Carolina. Source: Tetra Tech

Wet Conveyance

When the downspout features a bend, causing water to be trapped between runoff events, this system is known as wet conveyance (Figure B.5.8 and Figure B.5.9). Wet conveyance systems with buried downspouts can allow for cisterns to be placed further from buildings and might be preferable for aesthetic or overhead clearance purposes. When designing wet conveyance systems, the 100-year discharge from the catchment must be conveyed without any backwater onto the rooftop (considering all head losses through the pipe). Because water will permanently be stored in the downspout, watertight connections must be used to prevent leakage. A drain at the lowest elevation of the downspout can be installed, if desired, for dewatering and emergency maintenance.

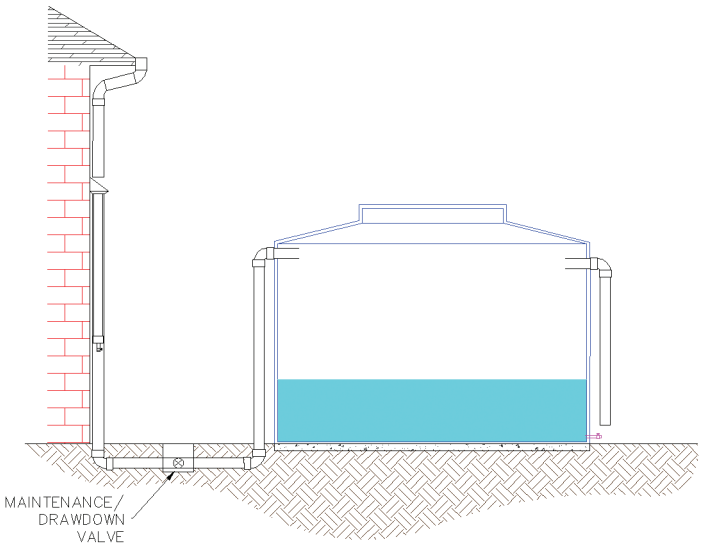


FIGURE B.5.8 CISTERN WITH WET CONVEYANCE FEATURING A DRAWDOWN VALVE FOR MAINTENANCE.



FIGURE B.5.9 CISTERN WITH A WET CONVEYANCE INLET CONFIGURATION.
Dallas, Texas. Source: NCSU-BAE

Step 4. Design Inlet Pretreatment Configuration

Stormwater runoff must be filtered before it enters the cistern to remove debris and particles that could clog the outlet. Two types of systems can be used: inlet filters and first-flush diverters. The following subsections discuss each pretreatment configuration in greater detail.

Inlet Filters

Inlet filters are designed to remove particles as runoff passes through the filters before entering the cistern; many filter options are available. The size and type of filter used will depend on the size of the area draining to the downspout. The filters can be installed at the gutter as shown in Figure B.5.10 or at the end of the downspout as shown in Figure B.5.11 depending on the configuration of the downspouts. Flow through filters that force all the runoff through the filter can be used for smaller drainage areas (less than 1,500 square feet). Filters capable of bypassing larger event flow could be required for larger drainage areas (1,500 to 3,000 square feet). A self-cleaning screen used for inlet filters should provide a minimum angle of declination of at least 45 degrees from horizontal, but angles of more than 45 degrees tend to enhance self-cleaning and prevent clogging (Nel 1996). Examples of two types of filters are shown in Figure B.5.12 and Figure B.5.13.



FIGURE B.5.10 INLET FILTERS AT THE GUTTER.
Greensboro, North Carolina. Source: Tetra Tech



FIGURE B.5.11 INLET FILTERS AT THE DOWNSPOUT.
Wilmington, North Carolina. Source: Tetra Tech



FIGURE B.5.12 FLOW-THROUGH INLET FILTER.
Source: Tetra Tech

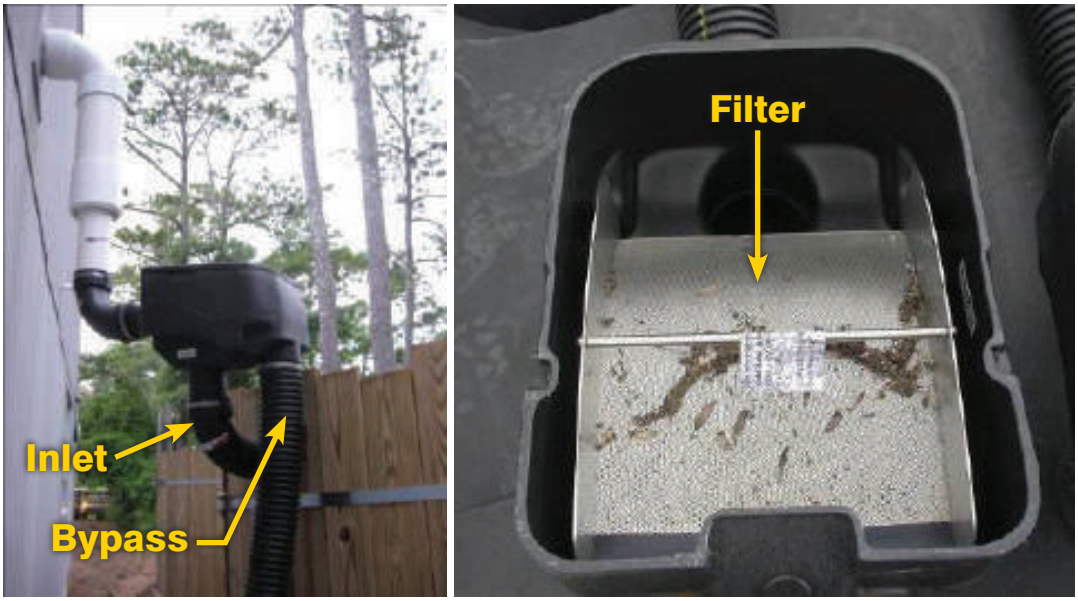


FIGURE B.5.13 SELF-FLUSHING FILTER WITH A BYPASS.
Source: Tetra Tech

First-Flush Diverter

First-flush diverters can be installed after the inlet filter and are designed to divert an initial volume of water away from the cistern to prevent small particles—initially washed off of the roof—from clogging the outlet. First-flush diverters are typically attached to the inlet or, in some cases, the inlet filter with a 4- to 6-inch diameter pipe with a small relief valve from which water can be diverted. The volume of water diverted away from the cistern depends on the length of the pipe. Once the diverter is full, a valve closes and water flows into the cistern. A first-flush diverter is not always required and inclusion is up to the designer depending on site conditions. A first-flush diverter is recommended for sites where pollen or other fine particles might not be removed by an inlet filter. Diverters must be routinely drained to provide capacity for the next runoff event.



FIGURE B.5.14 VALVE FOR A FIRST-FLUSH DIVERTER.
Source: NCSU BAE



FIGURE B.5.15 FIRST FLUSH DIVERTER CONFIGURATION.
Source: NCSU BAE

Step 5. Select and Size the Appropriate Outlet and Overflow/Bypass Method

Low-Flow Outlet

The outlet of the cistern should be designed to release the volume of captured runoff at a rate below the design storm rate at its maximum capacity. The outlet of the cistern should be directed to a bioretention area or other pervious surface with enhanced infiltration capacity as demonstrated in Figure B.5.16. Irrigation area requirements for the Edwards Aquifer Recharge, Contributing, and Transition Zones are presented in Table B.5.3; these requirements are applicable to all areas.

The elevation of the low-flow outlet depends on the demand for alternative water use. When water demand and use is high (such as when the cistern is being used for toilet flushing, car washing,



FIGURE B.5.16 CISTERN OUTLET.
San Diego, California. Source: Tetra Tech.

TABLE B.5.3 IRRIGATION AREA REQUIREMENTS FOR CISTERNS IN THE EDWARDS AQUIFER RECHARGE, CONTRIBUTING, AND TRANSITION ZONES (APPLICABLE TO ALL AREAS)

IRRIGATION/INFILTRATION AREA REQUIREMENTS
• 12 inches of soil cover, according to geotechnical investigation
• 100 feet from wells, septic systems, natural wetlands, and streams
• No sensitive or geologic features that could allow water to directly enter the aquifer
• Coarse soil material (diameter greater than 0.5 inch) does not make up more than 30% of the soil volume
• Slopes less than 10%
• Soil permeability and surface area sufficient to produce no runoff

Source: Barrett 2005

or consistent irrigation), the low-flow outlet can be placed such that half of the tank remains full for use. If stormwater management is the sole purpose of the cistern, the low-flow outlet should be placed at the bottom so that the tank can dewater and provide maximum capacity for storage of subsequent rain events. Figure B.5.17 illustrates example low flow outlet placement. Regardless of where the outlet is placed, temporary storage must be provided above the outlet elevation to capture the design storm volume. Models, such as the *Rainwater Harvester Design Model* (North Carolina State University 2008), can be used to optimize orifice placement.

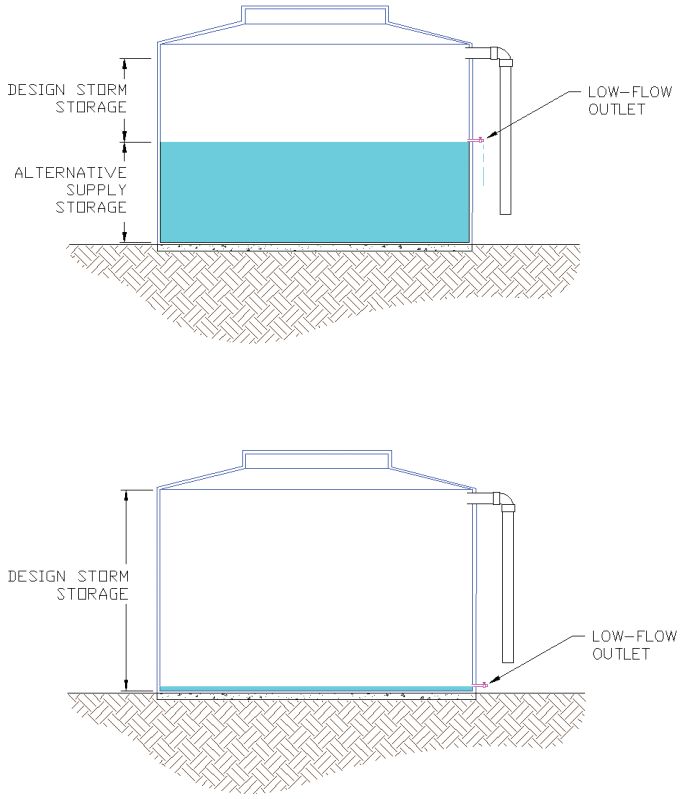


FIGURE B.5.17 TOP: A LOW FLOW OUTLET IS PLACED TO PROVIDE EQUAL PARTS DETENTION STORAGE AND STORAGE FOR ALTERNATIVE USE. BOTTOM: PLACING THE LOW FLOW OUTLET AT THE BOTTOM OF THE CISTERN ENSURES MAXIMUM DESIGN STORM STORAGE.

Overflow or Bypass

All cisterns should have an overflow for runoff volumes that exceed the capacity of the cistern. The overflow should be set slightly below the inlet. Overflow connections should be connected to the tank using appropriate watertight gaskets. An additional bypass can be incorporated using an appropriate inlet filter. Examples of an overflow discharging to vegetated areas are provided in Figure B.5.18.



FIGURE B.5.18 TOP: A CHANNEL DIRECTS OVERFLOW AWAY FROM BUILDING. BOTTOM: A CISTERN OVERFLOWS TO AN ADJACENT BIORETENTION AREA.

Top: Chihuahuan Desert Nature Center, Fort Davis, Texas.
Bottom: 100 E. Guenther, San Antonio, Texas. Source: Tetra Tech

All overflow and outlet volumes should be directed safely away from all structural foundations and any areas where infiltration could have an adverse effect. Overflow and bypass mechanisms should be sized to safely convey the 100-yr discharge without any backwater onto the adjacent roof. Calculation of 100-yr conveyance should account for head losses through all pipe sections, elbows, entrances, and exits.

Step 6. Specify Cautionary Signage, Pipe Color, and Locking Features

Part I, Subpart A, Chapter 86 of the City of San Marcos City Code details the requirements for the City of San Marcos utilities. Signage for the rainwater harvesting facilities is not described in this code. Designers should follow the City of San Marcos Plumbing Code or International Plumbing Code requirements for non-potable water signage.

In addition to preventing accidental ingestion, signage on cistern manways should restrict access to individuals with appropriate confined space credentials.



FIGURE B.5.19 TWO CISTERNS WITH PIPE CONNECTIONS.
Hardberger Park, San Antonio, Texas. Source: Tetra Tech

Step 7. Design for Multi-Use Benefits

By design, rainwater harvesting practices offer multi-use benefits by providing an alternative non-potable water source while controlling runoff volume and rate. In addition to hydrologic and water quality benefits, cisterns and rain barrels can be designed for multi-use benefits by:

- Providing irrigation for landscape beds and vegetated stormwater practices
- Offsetting non-potable water supplies used for toilet flushing, car washing, swimming pools, street sweeping, and other uses (nonresidential cisterns only)
- Incorporating aesthetically pleasing colors, murals, or facades
- Incorporating creative downspout designs for small practices (rain chains)
- Raising public awareness of stormwater issues with signage

Step 8. Additional Design Specifications

The following considerations relevant to safety and water reuse should be included in design plan notes and specifications.

Vector Control

The inlets and outlets of cisterns and rain barrels should be covered with a simple piece of filter material, such as a screen or wire mesh, to prevent mosquito breeding. A 1 mm or smaller mesh is recommended. Screens at the inlet should be placed downstream of debris filters to prevent clogging by leaves. Overflow/bypass openings should be covered with a non-clogging configuration, such as a screen mesh flap that hangs across the pipe opening—the bottom of the flap should be weighted or attached with small magnets such that it remains closed when no flow is present, but can easily open to allow overflow when the tank is full.

Routing Water for Use

The method of routing water depends on the intended use. For basic irrigation, gravity can often be used to route harvested rainwater to nearby vegetation beds or infiltrating stormwater practices. To route water for use inside nonresidential structures or for greater distances from the cistern, a pump might be required. Submersible water pumps are commonly used, but pumps can also be installed in utility boxes next to the cistern. Pipes conveying harvested water may not be placed in the same trench as potable water pipes, a 2-foot horizontal separation must be maintained between harvested

and potable water at all times. Buried potable water pipes that cross harvested water pipes must be at least 12 inches above the harvested water and must have a PVC sleeve that extends horizontally 2 feet to either side of the crossing. Harvested water should also be protected from contamination by sewer pipes in the same manner as potable water pipes.

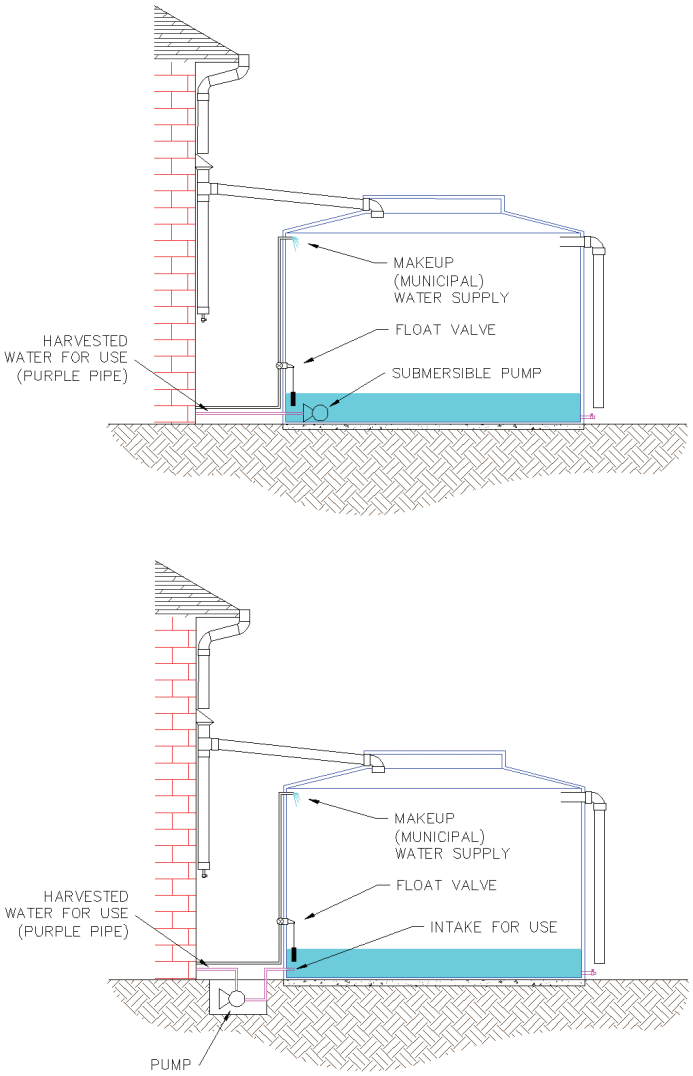


FIGURE B.5.20 TOP: CONCEPTUAL SCHEMATIC OF CISTERN WITH SUBMERSIBLE PUMP. BOTTOM: CONCEPTUAL SCHEMATIC OF CISTERN WITH EXTERNAL PUMP.

Makeup Water Supply for Dedicated Use

If the cistern will be used to offset non-potable water demand of nonresidential buildings (such as for toilet flushing) a makeup, or backup, water supply system is typically installed to maintain a minimum volume of storage water in the cistern for dedicated use. A number of different makeup systems are available, most of which use floats and valves similar to toilet tank components. When the cistern level drops below the minimum capacity, the valve is opened and municipal water supply is used to fill the tank to a specified level. When municipal water is used as a makeup supply for cisterns, both an air gap and a reduced pressure backflow device must be installed, and access to backflow preventers must be maintained.

Cistern Material Specification

Rainwater harvesting tanks are typically constructed of plastic, metal, or concrete. The specified material will affect the quality of captured runoff, aesthetics, configuration, installation, and cost. Plastic tanks can experience algal growth if not completely opaque. In general, cisterns are expected to last 20 to 50 years (Kowalsky and Thomason 2011). A detailed description of cistern materials is provided in Texas Water Development Board (2005).

B.5.4: CONSTRUCTION CONSIDERATIONS

Cisterns and rain barrels can present safety hazards if improperly designed. Engineers should direct contractors to implement appropriate OSHA health and safety protocol when installing cisterns. Elevated rain barrels and tall cisterns should be securely anchored to prevent toppling and subsequent injury.

B.5.5: INSPECTION CRITERIA

To maintain the effectiveness of structural SCMs, regular inspection is essential. Generally, SCM inspection and maintenance can be categorized as routine and as needed. Routine activities, performed regularly (e.g., monthly) ensure that the SCM is in good working order and continues to be aesthetically pleasing. Routine inspection is an efficient way to prevent potential nuisance situations from developing and reduce the need for repair or maintenance. Routine inspection also reduces the chance of degrading the quality of the effluent by identifying and correcting potential problems regularly. Property maintenance personnel should be instructed to inspect SCMs during their normal routines.

In addition to routine inspections, as-needed inspection/maintenance of all SCMs should be performed after any event or activity that could damage the SCM, particularly after every large storm event. Post-storm inspections should occur after the expected drawdown period for the SCM, when the inspector can determine if the SCM is draining correctly.

In general, individual SCMs can be described with minimum performance expectations, design criteria, structural specifications, date of implementation, and expected life span. Recording such information will help the inspector determine whether a SCM's maintenance schedule is adequate or requires revision and will allow comparison between the intended design and the as-built conditions. Checklists also provide a useful way for recording and reporting whether major or minor renovation or routine repair is needed. The effectiveness of a SCM might be a function of the SCM's location, design specifications, maintenance procedures, and performance expectations. Inspectors should be familiar with the characteristics and intended function of the SCM so they can recognize problems and know how they should be resolved.

Routine and as-needed SCM inspections consist of technical and non-technical activities as summarized below:

- Inspect the general conditions of the SCM and areas directly adjacent.
- Maintain access to the site including the inlets, side slopes (if applicable), forebay (if one exists), SCM area, outlets, emergency spillway, and so on.
- Examine the overall condition of vegetation.
- Eliminate any possibility of public hazards (vector control, unstable public access areas).
- Check the conditions of inflow points, pretreatment areas (if they exist), and outlet structures.
- Inspect and maintain the inlet and outlet regularly and after large storms.
- Ensure that the pretreatment areas meet the original design criteria.
- Check the encroachment of undesirable plants in vegetated areas. This could require more frequent inspections in the growing season.
- Inspect water quality improvement components. Specifically, check the stormwater inflow, conveyance, and outlet conditions.
- Inspect hydrologic functions such as maintaining sheet flow where designed, ensuring functional pretreatment, maintaining adequate design storage capacity, and verifying proper operation of outlet structures.
- Check conditions downstream of the SCM to ensure that flow is properly mitigated below the facility (e.g., excessive erosion, sedimentation).

In every inspection, whether routine or as needed, the inspector should document whether the SCM is performing correctly and whether any damage has occurred to the SCM since the last inspection. Ideally, the inspector will also identify what should be done to repair the SCM if damage has occurred. Documentation is very important in maintaining an efficient inspection and maintenance schedule, providing evidence of ongoing inspection and maintenance, and detecting and reporting any necessary changes in overall management strategies.

Inspection frequency for bioretention facilities is included in Table B.5.4 in Section 5.6.

B.5.6: OPERATION AND MAINTENANCE

Cisterns require regular maintenance during the rainy season to ensure proper function. Table B.5.4 lists specific tasks which are described below:

1. The main source of debris in the cistern is leaf litter and other detritus collected in the gutter system. The gutter systems should be inspected and cleaned. Any leaks should be immediately repaired.
2. Check inlet filters to prevent clogging and debris accumulation to allow for proper flow into the cisterns. Clean as needed to ensure proper operation.
3. Outlet pipes and fittings should be inspected to verify proper flows from the cistern. Cisterns should empty within 24 to 48 hours.
4. Overflow systems should direct water away from any structural foundations.
5. Cisterns should be checked for structural stability and secured as necessary.
6. It is possible for some sediment and debris to accumulate in the bottom of the cistern. Access to the cistern should be maintained, and it is necessary to conduct a visual inspection to verify debris in the cistern.

When harvested rainwater is used to replace non-potable water demand, inspections must occur upon installation and annually to ensure proper function of backflow preventers.

TABLE B.5.4 INSPECTION AND MAINTENANCE TASKS FOR CISTERNS

TASK	FREQUENCY	INDICATOR MAINTENANCE IS NEEDED	MAINTENANCE NOTES
Gutter and rooftop inspection	Biannually and before heavy rains	Inlet clogged with debris	Clean gutters and roof of debris that have accumulated, check for leaks
Remove accumulated debris	Monthly	Inlet clogged with debris	Clean debris screen to allow unobstructed stormwater flow into the cistern
Structure inspection	Biannually	Cistern leaning or soils slumping/ eroding	Check cistern for stability, anchor system if necessary
Structure inspection	Annually	Leaks	Check pipe, valve connections, and backflow preventers for leaks
Add ballast	Before any major wind-related storms	Tank is less than half-full	Add water to half full
Miscellaneous upkeep	Annually		Make sure cistern manhole is accessible, operational, and secure

B.5.7: REFERENCES

Barrett, M.E. 2005. *Complying with the Edwards Aquifer Rules. Technical Guidance on Best Management Practices*. RG-348. Prepared for Texas Commission on Environmental Quality, Field Operations Division, Austin, TX.

City of San Antonio. 2009. Appendix C Gray Water Recycling Systems and Reclaimed/Recycled Water Systems. *2009 International Plumbing Code and 2009 International Fuel Gas Code/Local Amendments.* San Antonio, TX.

Jones, Matthew P and Hunt, W.F. 2008. *Rainwater Harvesting: Guidance for Homeowners.* North Carolina Cooperative Extension, Raleigh, NC.

Kowalsky, G., and K. Thomason. 2011. *Cistern Design Considerations for Large Rainwater Harvesting Systems.* http://rfcd.pima.gov/pdd/lid/pdfs/cistern-design-for-water-harvesting.pdf.

Nel, C. 1996. "Die ontwikkeling van 'n struktuur vir die verwydering van vaste besoedeling uit stormwateraflope". Unpublished DTechEng thesis, Technikon Pretoria.

North Carolina State University. 2008. *Rainwater Harvester Design Model*. North Carolina State University, Department of Biological and Agricultural Engineering, Stormwater Engineering Group. http://www.bae.ncsu.edu/topic/waterharvesting/RainwaterHarvester_2.0.zip.

Texas A&M AgriLife Extension Service. 2013. Rainwater Harvesting (online). Accessed 20 June 2013 at http://rainwaterharvesting.tamu.edu/.

TCEQ (Texas Commission on Environmental Quality). *2011. Rainwater Harvesting with Rain Barrels. A "Take Care of Texas" Guide.* GI-383. Austin, TX.

Texas Water Development Board. 2005. *The Texas Manual on Rainwater Harvesting.* Third Edition. Austin, TX.

B.6: ADDITIONAL LID SCMS

B.6.1: VEGETATED FILTER STRIPS

Vegetated filter strips (VFSs) are bands of dense, permanent vegetation with a uniform slope, designed to provide pretreatment of runoff generated from impervious areas before flowing into another SCM as part of a treatment train. Vegetated filter strips on highly permeable soils can also provide infiltration, improving volume reduction. Increased infiltration can decrease the necessary horizontal length. Such characteristics make it ideal to use vegetated filter strips as a SCM around roadside shoulders or safety zones.

Vegetated filter strips are implemented for improving stormwater quality and reducing runoff flow velocity. As water sheet flows across the vegetated filter strip, the vegetation filters out and settles the particulates and constituents, especially in the initial flow of stormwater. Removal efficiency often depends on the slope, length, gradient, and biophysical condition of the vegetation in the system.

While VFS are applicable in many different areas, there are two primary applications for vegetative filter strips. Roadways and small parking lots are ideal locations where runoff that would otherwise discharge directly to a receiving waterbody, passes through the filter strip before entering a conveyance system. Properly designed roadway medians and shoulders make effective vegetated filter strips. The second application is land maintained in the natural condition adjacent to perimeter lots in subdivisions that will not drain via gravity to other stormwater treatment systems. The catchment area must have sheet flow to the filter strips without the use of a level spreader. VFS often require a large amount of space relative to other SCMs so they can be restricted in some areas beyond those two examples mentioned above.

Successful performance of filter strips relies heavily on maintaining shallow dispersed flow. If runoff is flowing over the VFS too fast, or in a concentrated manner, it will likely lead to rill erosion or scouring. To avoid flow channelization and maintain performance, a filter strip should:

- Contain dense vegetation with a mix of erosion resistant, soil binding species;
- Engineered vegetated filter strips should be graded to a uniform, even and a slope of less than 20% (5:1);
- Natural vegetated filter strip slopes should not exceed 10% (10:1) on average, providing that there are no flow concentrating areas on the strip; and
- Laterally traverse the contributing runoff area.

Filter strips can be used upgradient from watercourses, wetlands, or other water bodies, along toes and tops of slopes, and at outlets of other stormwater management structures. The most important criteria for selection and use of this SCM are space and slope.

Advantages and limitations of filter strips are outlined in Table B.6.1.

TABLE B.6.1 ADVANTAGES AND LIMITATIONS OF FILTER STRIPS

ADVANTAGES	LIMITATIONS
• Good pretreatment SCM	• Must be sited next to impervious surfaces
• Simple to install (often requiring only minimal earthwork and planting)	• Might not be suitable for industrial sites or large drainage areas
• Simple, aesthetically pleasing landscaping	• May require large footprint for sufficient treatment
• Low cost/maintenance	• Requires sheet flow across vegetated area (cannot treat high velocity flows)
	• Application in arid areas is limited because of the need for thick vegetation
	• Does not provide attenuation of peak flows



FIGURE B.6.1 VEGETATED FILTER STRIP THAT PRETREATS ROADWAY RUNOFF.
Apex, North Carolina. Source: Tetra Tech



FIGURE B.6.2 VEGETATED FILTER STRIP SURROUNDING A BIORETENTION AREA IN A PARKING LOT.
Raleigh, North Carolina. Source: Tetra Tech



FIGURE B.6.3 VEGETATED FILTER STRIP NEXT TO A PARKING LOT.
San Antonio, Texas. Source: Tetra Tech

B.6.2: VEGETATED SWALES

Vegetated swales are shallow, open grass channels that are LID alternatives to traditional curbs and gutters. Swales are designed to convey runoff while providing limited pollutant removal by sedimentation and horizontal filtration through vegetation. Swales are effective for pretreatment of concentrated flows before discharge to a downstream SCM. **Vegetated swales should not be confused with bioswales, which rely on vertical filtration of runoff through subsurface bioretention media.** Compared with other LID practices, vegetated swales have a relatively low construction cost, a moderate maintenance burden, and require only a moderate amount of surface area.

Vegetated swales require shallow slopes and soils that drain well. Pollutant removal capability is related to channel dimensions, longitudinal slope, and amount of vegetation. Optimum design of these components will increase contact time of runoff through the swale and improve pollutant removal rates.

Grassy swales are primarily stormwater conveyance systems. They can provide sufficient control under light to moderate runoff conditions, but their ability to control large storms is limited. Therefore, they are most applicable in low to moderate sloped areas or along

highway medians as an alternative to ditches and curb and gutter drainage. Grassy swales can be used as a pretreatment measure for other downstream facilities, such as bioretention areas. Enhanced grassy swales utilize engineered soils and an underdrain to provide filtration of pollutants.

Swales can be more aesthetically pleasing than concrete or rock-lined drainage systems and are generally less expensive to construct and maintain. Swales can slightly reduce impervious area and reduce the pollutant accumulation and delivery associated with curbs and gutters. The disadvantages of this technique include the possibility of erosion and channelization over time, and the need for more ROW than a storm drain system.

Advantages and limitations of vegetated swales are outlined in Table B.6.2.

TABLE B.6.2 ADVANTAGES AND LIMITATIONS OF VEGETATED SWALES

ADVANTAGES	LIMITATIONS
• Combines limited stormwater treatment with runoff conveyance • Often less expensive than curb and gutter • Provides limited peak flow reduction • Can be installed in narrow, marginal spaces along roadways and parking lots to convey runoff to downstream SCMs	• Higher maintenance than curb and gutter • Impractical in areas with very flat grades or steep topography (can cause nuisance standing water and vector issues) • Not as effective for high flow volumes/velocities • Not effective for volume reduction



FIGURE B.6.4 VEGETATED SWALE IN AN INSTITUTIONAL SETTING.
James Madison High School Agriscience Building, San Antonio, Texas. Source: Bender Wells Clark Design



FIGURE B.6.5 ROADSIDE VEGETATED SWALE.
San Antonio, Texas. Source: Tetra Tech

B.6.3: GREEN ROOFS

Green roofs reduce runoff volume and rates by intercepting rainfall in a layer of rooftop growing media. Rainwater captured in rooftop media then evaporates or is transpired by plants back into the atmosphere. Rainwater in excess of the media capacity is detained in a drainage layer before flowing to roof drains and downspouts. Green roofs are highly effective at reducing or eliminating rooftop runoff from small to medium storm events, which can reduce downstream pollutant loads; however, green roofs do not typically improve the quality of captured rainwater. In addition to stormwater volume reduction, green roofs offer an array of benefits, including extended roof lifespan (due to additional sealing, liners, and insulation), improved building insulation and energy use, reduction of urban heat island effects, opportunities for recreation and rooftop gardening, noise attenuation, air quality improvement, bird and insect habitat, and aesthetics (Tolderlund 2010; Berndtsson 2010; Getter and Rowe 2006). Green roofs can be designed as extensive, shallow-media systems or intensive, deep-media systems depending on the design goals, roof structural capacity, and available funding. Extensive green roofs in the San Marcos region may require drip irrigation to sustain vegetation through hot summer months, but air conditioner condensate or harvested rainwater can be used for this purpose. To improve vegetation resistance and resilience, a biodiverse, locally-adapted plant palette should be used. Even with careful plant selection, many "green" roofs will remain brown during much of the year. *Blue roofs* are another form of rooftop runoff management also known as *rooftop ponding areas* or *rooftop detention* that can be effective for volume and flow control. Brown roofs are another form of rooftop runoff management focused on grasses or other "brown" vegetation rather than succulents, although this manual focuses on vegetated roofs because of their multi-use benefits. Additional information and design recommendations for blue roofs and brown roofs can be found in *Guidelines for the Design and Construction of Stormwater Management Systems* from the New York City Department of Environmental Protection and New York City Department of Buildings. Table B.6.3 describes the advantages and limitations of green roofs.

B.6.3.1: Extensive Green Roof

Green roofs with shallow, lightweight media are generally known as extensive. Media depths typically range from 4 to 6 inches to minimize loading on structures. Extensive green roofs are typically implemented solely for stormwater management, although alternative benefits are often realized (including reduced energy costs, improved roof lifespan, and pollinator habitat). An example of an extensive green roof is provided in Figure B.6.6.

TABLE B.6.3 ADVANTAGES AND LIMITATIONS OF GREEN ROOFS

ADVANTAGES	LIMITATIONS
• Reduces stormwater volume and peak flow through evapotranspiration	• Structural constraints could preclude use
• Independent of site soils and geological setting	• Installation can be challenging in certain locations
• Can be used to reduce size of downstream SCMs	• Tend to be costly compared to other stormwater volume reduction practices
• Improve building energy use and reduce energy costs	• Although total stormwater volume is reduced, tend to export high nutrient concentrations and possibly pathogens (Berndtsson 2010)
• Enhance roof lifespan	• Roof slopes steeper than 45° tend to require special design
• Provide rooftop recreation and gardening opportunities	• May require irrigation for maintenance of vegetation during summer months (depends on plant selection and design goals)
• Reduce noise and air pollution	
• Provide urban bird and insect habitat	
• Improve aesthetics and increase property values (if visible)	



FIGURE B.6.6 EXTENSIVE GREEN ROOFS REDUCE STORMWATER RUNOFF WHILE PROVIDING COOLING EFFECTS, HABITAT FOR POLLINATORS, AND AESTHETIC VALUE.
Live Roof System, Hipolito F. Garcia Federal Building, San Antonio, Texas. Source: Joss Growers

B.6.3.2: Intensive Green Roof

Roof gardens and rooftop parks with media deeper than 6 inches are commonly known as intensive green roofs. Unlike extensive green roofs, intensive green roofs are typically installed primarily for recreational and aesthetic purposes and provide stormwater benefits as an auxiliary function. Because deep media depth exerts high loads on underlying structures, implementation of intensive green roofs is common on the top level of parking decks, high-rise buildings, and other structures specifically designed for extreme loading. Example of an intensive green roof is shown in Figure B.6.7.



FIGURE B.6.7 INTENSIVE GREEN ROOFS PROVIDE RECREATIONAL, AESTHETIC, AND EDUCATIONAL OPPORTUNITIES IN ADDITION TO STORMWATER BENEFITS.
James Madison High School Agriscience Building, San Antonio, Texas. Source: Tetra Tech

B.6.4: CONSTRUCTED WETLANDS AND WET PONDS

Stormwater wetlands are engineered, shallow-water ecosystems designed to treat stormwater runoff. Commonly implemented in low-lying areas, stormwater wetlands are well suited to areas along river corridors where water tables are higher. Sediment and nutrients are efficiently reduced by stormwater wetlands by means of sedimentation, chemical and biological conversions, and uptake. Stormwater wetlands provide flood control benefits by storing water and slowly releasing it over 2 to 5 days. In addition to stormwater management, stormwater wetlands provide excellent plant and wildlife habitat and can often be designed as public amenities. Research has indicated that a home located next to stormwater wetlands can have a 20 to 30 percent higher selling price (Russell et al. 2012).

Wet ponds, also called stormwater ponds, retention ponds, wet extended detention ponds, differ from constructed wetlands primarily in having a greater average depth. Wet ponds treat incoming stormwater runoff by settling and biological uptake. The primary removal mechanism is settling as stormwater runoff resides in this pool, but pollutant uptake, particularly of nutrients, also occurs to some degree through biological activity in the pond. Wet ponds are among the most widely used stormwater practices. While there are several different versions of the wet pond design, the most common modification is the extended detention wet pond, where storage is provided above the permanent pool in order to detain stormwater runoff and promote settling.

One common denominator of each of these facilities is that they require the design and installation of a forebay. The forebay is designed to remove the majority of sediment and debris before it enters the main treatment portion of the facility. Wet ponds, retention ponds, extended detention ponds, and wetlands all require the installation of a sediment marker to determine the amount of sediment that enters a facility. Maintenance is required to remove the sediment accumulated, and the sediment and debris should be disposed of according to local regulations.

Advantages and limitations of stormwater wetlands are outlined in Table B.6.4.



FIGURE B.6.8 A LARGE LINEAR STORMWATER WETLAND.
Lenoir, North Carolina. Source: Tetra Tech

TABLE B.6.4 ADVANTAGES AND LIMITATIONS OF STORMWATER WETLANDS

ADVANTAGES	LIMITATIONS
• Excellent sediment and nutrient reduction • Useful in low-lying areas, areas with high water tables, or where infiltration is otherwise restricted/discouraged • Construction and design techniques similar to conventional detention ponds • Provide multi-benefit uses by enhancing biodiversity and providing recreational/educational opportunities • Typically require fewer vector control efforts than unvegetated ponds because properly maintained habitat supports mosquito predators (dragonflies and fish) • High habitat potential and aesthetic value	• Limited use in semi-arid climates where supplemental water would be required to maintain water level (a site-specific water balance must be performed to justify implementation) • Systems are end-of-pipe stormwater treatment systems



FIGURE B.6.9 SMALL WETLANDS ALONG THE PERIMETER OF A NEIGHBORHOOD.
Wilmington, North Carolina. Source: Tetra Tech

B.6.5: PROPRIETARY SYSTEMS

Currently, there are many proprietary systems on the market designed to meet stormwater management goals. Suppliers of these systems all have specific design and maintenance criteria available if this a desired option for a project site.

B.6.5.1: Benefits

There are many benefits to these and other systems currently on the market. First, many of these systems, like the other structural systems described above, can be custom designed for a specific project with regard to media mix and vegetation. Secondly, they can be a good choice for highly urban areas where space is limited or where retrofits to existing storm drains are desired. Lastly, they can be efficient to implement and often offer guarantees against performance and structural failure.

B.6.5.2: Limitations

While proprietary systems have certain advantages, they also have several limitations. First, under current regulations, several proprietary systems are not allowed in certain jurisdictions or over aquifer recharge or contributing zones. Thus, it is important to investigate any local or regional regulatory obstacles that may exist which prohibit or prescribe their application. Second, these systems can be limited in their ability to address site performance goals and the regional ecological conditions to the fullest degree. Many of the proprietary systems are designed to reach certain performance targets, such as volume reductions or solely filtration purposes, or a combination. If this approach is chosen, it is important to understand the various performance goals for each system to assess whether these match the performance goals of a project in San Marcos.

B.7: TREATMENT TRAINS

Using multiple treatment processes either in individual or multiple SCMs is called a *treatment train*. Meeting targeted treatment objectives can usually be achieved using a series of LID SCMs in a treatment train. Treatment trains can often be designed along rights-of-way, in parking lots, underground, or incorporated into landscaped areas. LID site planning should result in a treatment train of LID strategies and SCMs to meet treatment and water quality goals.

There are a number of reasons why this type of configuration is preferred. First, implementing a number of practices provides the opportunity to include a variety of unit processes (sedimentation, filtration, biological uptake, etc.) to treat the runoff, which optimizes the pollutant removal. Secondly, the use of multiple systems provides a level of redundancy so that at least partial treatment is being achieved even if one system is not functioning properly.

Probably the biggest benefit is the reduction in maintenance costs that can be achieved by using a dry system, such as a swale, upstream of ponds or other permanently wet facilities. Removal of accumulated sediment, trash, and debris from a dry swale is far easier and less expensive than removal of the same material once it enters a pond.

SCMs can be used singularly or in combination, or shared by multiple drainage areas, pursuant to local regulatory criteria (depending on project location and its jurisdiction).

An example of how SCMs can be implemented in combination to provide the maximum potential treatment for a site configuration includes a treatment train utilizing vegetated filter strips draining to a vegetated swale that then convey the stormwater to a bioretention area where stormwater is infiltrated or filtered through a soil media. An example of a treatment train is shown in Figure B.7.1. Such a treatment train can be integrated into the site to maximize hydrologic and water quality treatment using the unit processes of each SCM type. Effectiveness of individual or multiple integrated practices can be compared in terms of removing substances or groups of pollutants. Typical sources present an average of water quality performance data collected from multiple storm events over a multi-year period. SCMs sized to treat the volume produced by wet weather events will have the capacity to treat the smaller volume produced by dry weather flow with the same water quality performance. Water quality data is typically a combination of effluent and overflow samples.



FIGURE B.7.1 TREATMENT TRAIN FEATURING A PERMEABLE PAVEMENT CAPTURING RUNOFF BEFORE ENTERING A GRASSED BIORETENTION AREA.
Yuxi, China. Source: Tetra Tech

When no specific pollutant has been targeted for removal, regulators should work with designers to address pollutant removal through flow- or volume-based requirements or both. Under such circumstances, cost may become the most important deciding factor in SCM selection rather than pollutant removal performance.

B.8: COMMON DESIGN ELEMENTS

Many SCMs have similar elements or standards. Those common elements and associated design standards are outlined in this section.

B.8.1: GEOTECHNICAL INVESTIGATION

The design of most SCMs will rely on an initial geotechnical investigation. Performing soil tests early in the conceptual and preliminary design phases will ensure that the proposed system is optimized to actual site conditions and to prevent costly change orders resulting from poorly estimated soil parameters.

The investigation should include both desktop and field analyses to fully characterize the structural and hydrologic characteristics of a site. Desktop analyses can be used to generate a conceptual site design but should always be verified with field investigation. The following parameters can be determined by desktop analyses:

- Underlying geology (especially presence of karst geology or shallow bedrock)
- Site location with respect to Edwards Aquifer Recharge Zone
- Proximity to steep slopes
- Proximity to structural foundations, roadway subgrades, utilities, and other infrastructure
- Proximity to water supply wells
- Proximity to septic drain fields

Field investigations should be performed by a licensed soil scientist or geotechnical engineer. All soil testing should be performed at the depth of the initially proposed subgrade because this is the soil strata where infiltration could occur. If a detention (non-infiltrating system) is proposed, soil tests must still be performed to determine structural requirements and to identify the elevation of the seasonal high water table.

Sufficient test pits or borings should be done to adequately characterize the site soil conditions, but, at a minimum, the greater of 2 samples or 1 sample per 50,000 square feet of SCM area should be collected. Soils should be investigated to a depth of at least 3 feet below the proposed SCM subgrade. The following key parameters should be determined or verified by field investigation:

- The infiltration rate of the soils at the potential subgrade (ASTM D 3385 *Standard Test Method for Infiltration Rate of Field Soils Using Double-Ring Infiltrometer,* or a comparable method)
- The depth and texture of subsoils
- The depth to the seasonal high groundwater table
- Structural capacity of soils (if surface SCM, such as a rainwater harvesting facility or planter box, is intended)
- Presence of expansive clay minerals
- Presence of compacted or restrictive layers
- Underlying geology (especially presence of karst geology or shallow bedrock)
- Proximity to steep slopes
- Proximity to structural foundations, roadway subgrades, utilities, and other infrastructure
- Proximity to water supply wells
- Proximity to septic drain fields

In the Edwards Aquifer Recharge, Contributing, and Transition zones, at least 12 inches of natural soil must be provided wherever a practice is intended to discharge stormwater for infiltration (e.g. permeable pavement in the Contributing Zone or irrigation with harvested water from a cistern). Fill material may be used, but it must have a texture comparable to natural site soils. All soils should contain no wastes, debris, deleterious material, or material that can leach contaminants. Soils should contain less than 30 percent coarse material by volume, which is defined as material larger than 0.5 inch in diameter.

B.8.2: CURB CUTS

When SCMs are incorporated into highly impervious areas, such as parking lots and in road rights-of-way, curb cuts can be required to allow surface runoff to enter the SCM. Curb cuts are designed such that the design storm can pass through the curbing without causing water to pond in the travel lanes. Example curb cuts are shown in Figure B.8.1 and Figure B.8.2.

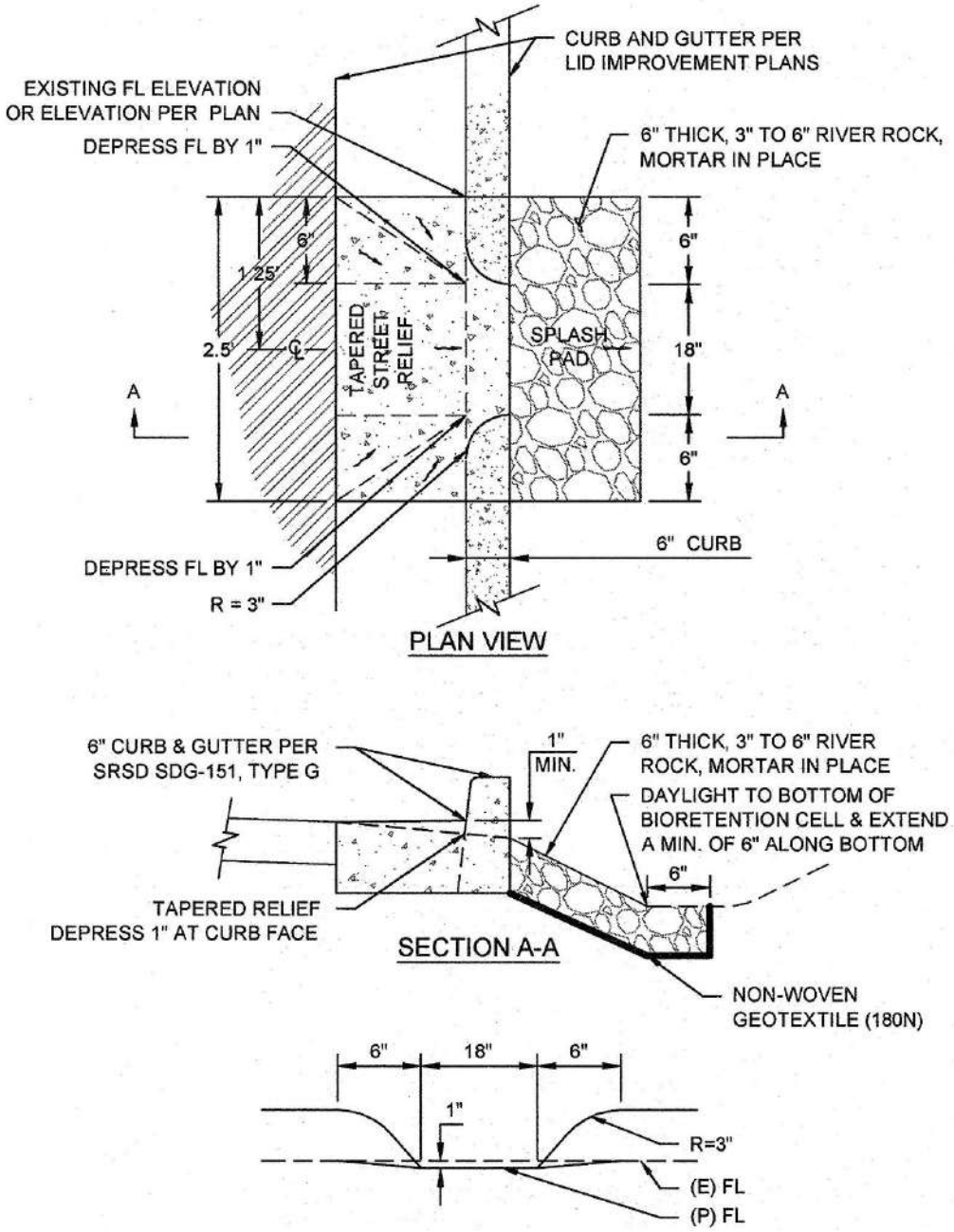


FIGURE B.8.1 TYPICAL CURB CUT DIAGRAM.



FIGURE B.8.2 A TYPICAL CURB CUT.
LA Zoo, Los Angeles, California. Source: Tetra Tech.



FIGURE B.8.3 MULTIPLE, SMALL CURB CUTS DISTRIBUTE STREET RUNOFF TO BIORETENTION AREAS WITHOUT EROSIVE FORCE.
Bioretention in residential neighborhood, Toledo, Ohio. Source: Tetra Tech

Designs have the following recommendations:

- The opening should be at least 18 inches wide at the base to prevent clogging and to provide dispersed flow.
- The curb cut can have vertical sides or have chamfered sides at 45 degrees.
- Slope the bottom of the concrete curb cut toward the stormwater facility.
- Provide a minimum 2-inch drop in grade between the curb cut entry point and the finished grade of the stormwater facility.
- The curb cut must pass the design storm flow without causing backup that would disrupt normal travel in the lane.
- The curb cut opening should be armored to prevent erosion. Concrete, stone, or sod can be used to armor the flow path to the base of the bioretention area. If a vegetated filter strip is provided downstream from the curb cut, a turf reinforcement mat may be required to stabilize the soil if flows are expected to exceed 3 ft/sec.

Some pretreatment flow reduction can be provided by using multiple, smaller curb cuts to minimize the flow at each opening and by armoring the curb opening from the back of the curb to the base elevation of the bioretention area (Figure B.8.3).

Figure B.8.4 and Figure B.8.5 show examples of potential curb cut configurations. Figure B.8.6 shows a covered curb cut that would be appropriate in areas experiencing high levels of pedestrian traffic. Inlets can be covered or protected for pedestrians or other traffic using a covered curb cut. Covered curb cuts, such as the one shown in Figure B.8.6, are preferred over other curb inlet methods including the use of pipes or linear cuts in the curbing for ease of maintenance. Covering the inlet with a removable grate allows for easy visual inspection of the inlet and can reduce the effort required for maintenance. Such curb cuts can also be modified with a small sump or lip to capture coarse sediments and trash. Armoring the curb opening from the back of the curb to the base elevation of the bioretention will reduce inlet velocities, preventing scour and erosion in the SCM.



FIGURE B.8.4 CURB CUTS AND VEGETATED FILTER STRIP FOR ROADSIDE BIORETENTION AREA.
Apex, North Carolina. Source: Tetra Tech.



FIGURE B.8.5 ARMORED CURB CUTS.
(Left) Raleigh, North Carolina. Source: Tetra Tech. (Right) Downey, California. Source: Tetra Tech.



FIGURE B.8.6 COVERED CURB CUT WITH SUMP.
Source: City of San Diego LID Design Manual

B.8.3: STABILIZATION AND ENERGY DISSIPATION

In some cases, the inlet or outlet can be a pipe with concentrated flow. Flow dissipation is difficult yet critical in such situations. Several options can be used for dissipating flow from a pipe. The flow can be discharged into a shallow forebay. Energy dissipation can be implemented at the outlet of the pipe, such as by using sod or stones, to slow the flow as shown in Figure B.8.7. All stone armoring should be sized such that it is not mobilized during high flows and should be underlain with appropriate geotextile fabric to prevent scour of underlying soils. Another option to dissipate energy from small pipes would be to install an elbow at the end of the pipe, with stable materials around the elbow, to slow the flow and allow the water to cascade onto a gravel pad. A small weep hole should be used to prevent water from permanently ponding in the elbow. An example of a constructed energy dissipater is shown in Figure B.8.8, and an upturned elbow used for energy dissipation is shown in Figure B.8.9.

Sediment markers are required at the inflow points for facilities with forebays, or at facilities where a high volume of sediment is expected. This includes bioretention facilities, wet ponds, extended detention ponds, and wetlands. The sediment markers will indicate the volume of sediment within a facility, and help promote prompt maintenance of each of the facilities.



FIGURE B.8.7 STONE FLOW DISSIPATER/FOREBAY.
Cary, North Carolina. Source: Tetra Tech.



FIGURE B.8.8 CONCRETE ENERGY DISSIPATER.
University of Texas at San Antonio, San Antonio, Texas. Source: Tetra Tech



FIGURE B.8.9 UPTURNED ROOF DOWNSPOUT ENERGY DISSIPATER.
San Antonio, Texas. Source: Tetra Tech, Inc.

Level spreaders are another technique for converting concentrated flows into diffuse, sheet flow. Runoff is distributed through a dead-end channel (sometimes called a *blind swale*) along the upslope side of the vegetated filter strip and evenly dispersed onto the vegetated filter strip along the level spreader as shown in Figure B.8.10 and Figure B.8.11. It is important that the lip of the level spreader be accurately level across the entire length and that a minimum 2-inch drop is provided from the lip to the gravel pad below. Level spreaders can be installed in an "arced" configuration if necessary, but the arc should always be convex such that flow is never concentrated (Figure B.8.11). Weir overflow equations can be used to determine the required level spreader length to produce nonerosive flows (Chow 1959).

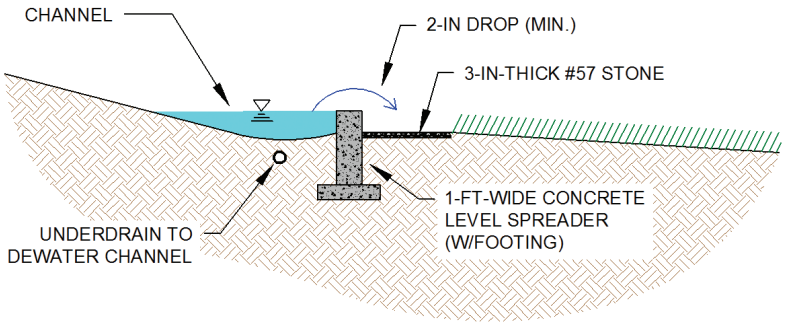


FIGURE B.8.10 TYPICAL LEVEL SPREADER PROFILE VIEW.

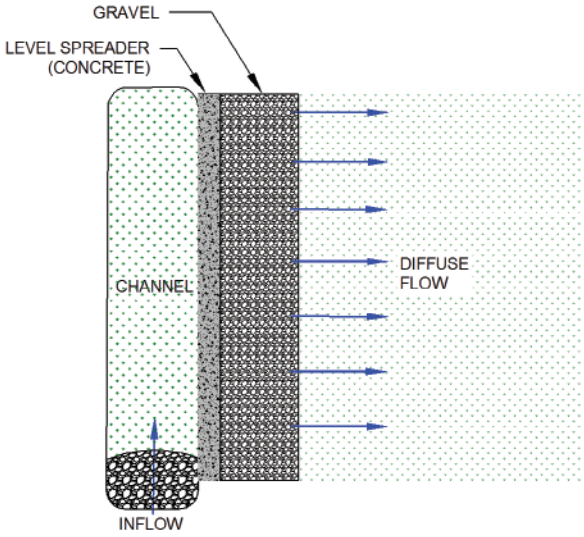


FIGURE B.8.11 TYPICAL LEVEL SPREADER PLAN VIEW.

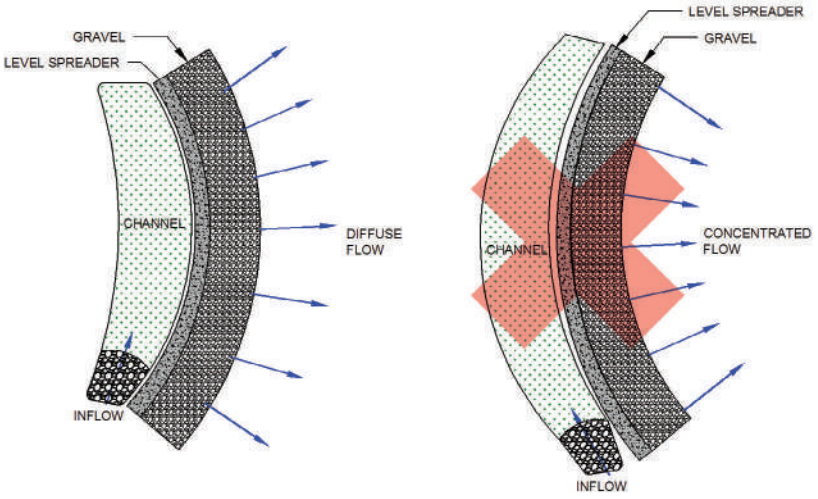


FIGURE B.8.12 FIGURE ILLUSTRATION PROPER INSTALLATION OF ARCED LEVEL SPREADER (LEFT) AND IMPROPER LEVEL SPREADER ARC (RIGHT)

B.8.4: UNDERDRAIN DESIGN

Underdrains are common design elements in bioswales, planter boxes, and sand filters. Soil testing should be performed at the site by a licensed soil scientist or geological engineer to determine the infiltration rate of the soils and the depth to the seasonally high groundwater table. If the infiltration rate of the soils where the infiltrating practice will be installed is less than 0.5 in/hr, or if a site is near a steep slope, underdrains will be required. If underdrains are used, they should meet the recommended specifications in Table B.8.1.

A barrier to separate the soil media from the drainage layer should be installed. Two options can be used for providing the separation from the soil media and the drainage layer:

- Option 1: Place a thin, 2- to 4-inch layer of pure sand and a thin layer (nominally 2 inches) of choking stone (such as ASTM No. 8) between the soil media and the drainage stone as shown in Figure B.8.13.
- Option 2: The drainage stone should be a washed No. 57 stone, or similar alternative that has been washed to remove all fines. The drainage stone should be used to provide a gravel blanket and bedding for the underdrain pipe. Place the underdrain on a 3-foot-wide bed of the drainage stone 6 inches deep and cover with the same drainage stone to provide a 16-inch minimum depth around the bottom, sides, and top of the slotted pipe.

TABLE B.8.1 UNDERDRAIN SPECIFICATIONS (BARRETT 2005)

COMPONENT	SPECIFICATION
Diameter	4-inch minimum
Material	Perforated Schedule 40 PVC
Perforation Type	Slotted or round, although slotted underdrains provide greater intake capacity, clog resistant drainage, and reduced entrance velocity into the pipe, thereby reducing the chances of solids migration
Perforation Spacing and Placement	The maximum spacing between perforations should not exceed 6 inches, but spacing of perforations is typically not critical to the function of the SCM as long as the total opening area provides capacity for the expected underdrain flow and does not limit infiltration through the soil media. The perforations can be placed closest to the invert of the pipe to achieve maximum potential for draining the facility. If an anaerobic zone is intended, the perforation can be placed at the top of the pipe.
Slope	1% minimum slope toward outlet
Cleanout Access	Rigid, unperforated observation pipes with a diameter equal to the underdrain diameter should be connected to each individual underdrain (every 250 to 300 feet in larger systems) to provide a cleanout port and an observation well to monitor dewatering rates. The wells/cleanouts should be connected to the perforated underdrain with the appropriate manufactured connections. The wells/cleanouts must extend 6 inches above the mulch or sod layer and be capped with a screw cap to avoid damage from maintenance and vandalism. The ends of upgradient, lateral underdrain pipes not terminating in an observation well/cleanout must also be capped.
Outfall	The underdrain can be connected to a vegetated swale, to another filter cell as part of a connected treatment system, daylight to a vegetated dispersion area using an effective flow dispersion device, stored for reuse, or to a stormwater drainage system.

A geotextile fabric should be placed between the soil media and the drainage layer as shown in Figure B.8.14. If a geotextile fabric is used, it must meet the minimum materials requirements listed in Table B.8.2.

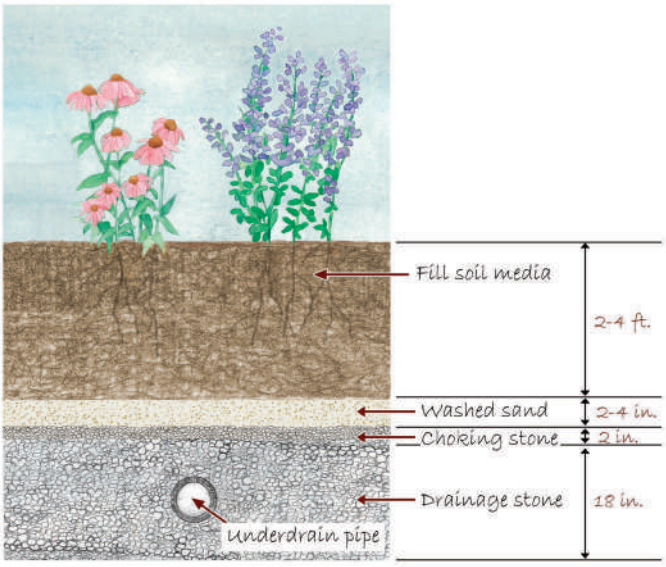


FIGURE B.8.13 UNDERDRAIN BARRIER OPTION 1: SOIL MEDIA BARRIER.
Source: City of San Diego LID Design Manual

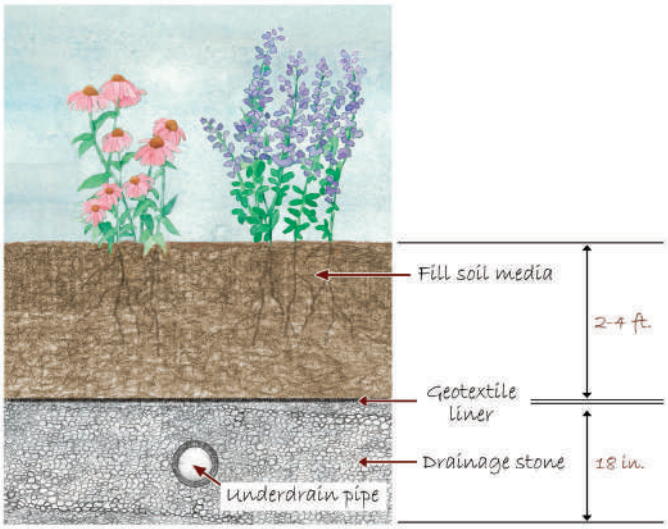


FIGURE B.8.14 UNDERDRAIN BARRIER OPTION 2: GEOTEXTILE LINER.
Source: City of San Diego LID Design Manual

Option 2 is a common method; however, geotextile clogging and biofouling has been observed in field investigations. In situations where there is concern of clogging around the geotextile, Option 1 is recommended.

TABLE B.8.2 GEOTEXTILE LAYER SPECIFICATIONS (BARRETT 2005)

GEOTEXTILE PROPERTY	VALUE	TEST METHOD
Trapezoidal tear (lbs)	40 (min)	ASTM D4533
Permeability (cm/sec)	0.2 (min)	ASTM D4491
AOS (sieve size)	#60–#70 (min)	ASTM D4751
Ultraviolet resistance	70% or greater	ASTM D4355

B.8.5: TRASH RACKS

Non-clogging intake designs should be specified whenever litter or debris pose a risk of clogging drawdown pipes. For stormwater facilities that require ponding, an intake pipe with downturned opening extending 6 to 12 inches below the permanent pool (Figure B.8.15) or enclosing the drawdown orifice (Figure B.8.16) will reduce the risk of clogging by floating debris. Providing a downward slope on the entire intake pipe can also reduce deposition of sediment within the pipe itself.



FIGURE B.8.15 DRAWDOWN PIPE WITH A DOWNTURNED ELBOW.
Source: NCSU BAE



FIGURE B.8.16 OUTLET STRUCTURE WITH A TRASH RACK AND PROTECTED DRAWDOWN ORIFICE.
Wilmington, North Carolina. Source: NCSU BAE

When additional exclusion of trash and debris is required (such as in sand filter sedimentation chambers or for emergency/maintenance dewatering intakes in stormwater wetlands) a trash rack or other debris exclusion device should be specified. A simple trash rack can be designed by nesting a perforated riser pipe within a wire mesh cage. The bottom portion of the pipe should be enveloped in a cone of washed stone (ASTM No. 57 stone is adequate) as shown in Figure B.8.17. The specific trash rack configuration will depend on site conditions and design goals, but regardless of configuration all trash rack should allow for safe bypass of high flows. For further guidance on trash rack design, see Barrett (2005) and UDFCD (2010).

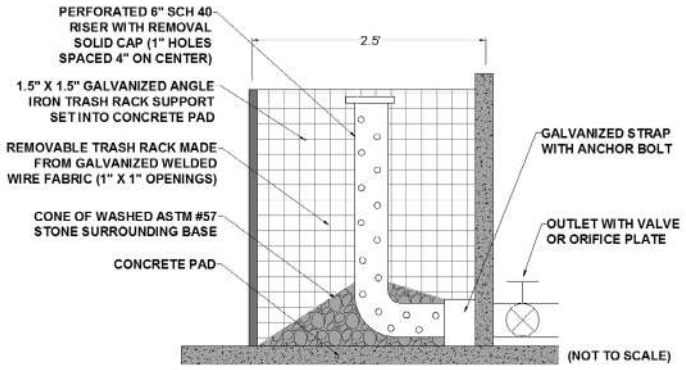


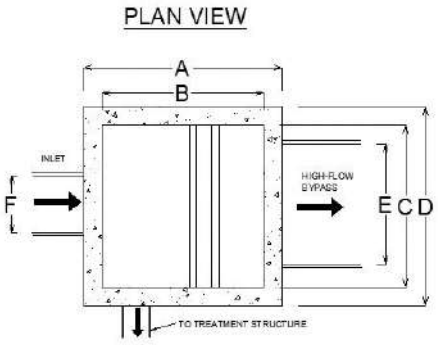
FIGURE B.8.17 SCHEMATIC OF A TRASH RACK FOR DEWATERING SEDIMENTATION CHAMBERS AND STORMWATER WETLANDS (ADAPTED FROM BARRETT 2005).

B.8.6: DIVERSION STRUCTURES

If a SCM is designed to be an offline system, a structure will be required to divert the design volume into the SCM. Figure B.8.18 shows an example of a typical diversion structure. When the capacity of the SCM is exceeded or the flow exceeds the capacity of the diversion pipe, the flow bypasses over the weir and flows directly to the stormwater drainage system. The bypass pipe should be sized to limit the flow into the SCMs to non-erosive flows. When flows through a SCM could exceed the recommended maximum flow rates, regardless of whether a system is online or offline, a diversion structure is recommended to prevent erosion in the SCM.

The flow velocity in a mulched system should not exceed 1 ft/sec. Flow in a grassed system should not exceed 3 ft/sec. Flows can be greater (up to 14 ft/sec) with the use of reinforced turf matting and will depend on the matting selected. A diversion structure should be used to ensure that flows through the system do not exceed the recommended design flow. More information on determining erosive flows can be found in TxDOT (2011).

In situations where stormwater is collected in a pipe and routed to a SCM, a diversion structure should be designed at the inlet of the SCM to divert flows that exceed the volume or flow capacity of the SCM.



	DESCRIPTION
A	OUTSIDE WIDTH OF DIVERTER BOX
B	INSIDE WIDTH OF DIVERTER BOX
C	INSIDE WIDTH OF DIVERTER BOX
D	OUTSIDE WIDTH OF DIVERTER BOX
E	DIAMETER OF OUTLET PIPE
F	DIAMETER OF INLET PIPE
G	WIDTH OF DROP INLET GRATE
H	OVERHANG
J	DISTANCE TO WEIR
K	TOP WIDTH OF WEIR
L	DISTANCE TO WEIR
M	HEIGHT OF WEIR
N	DIAMETER OF PIPE TO TREATMENT
P	DEPTH OF BOX
Q	DISTANCE TO WEIR
R	BOTTOM WIDTH OF WEIR

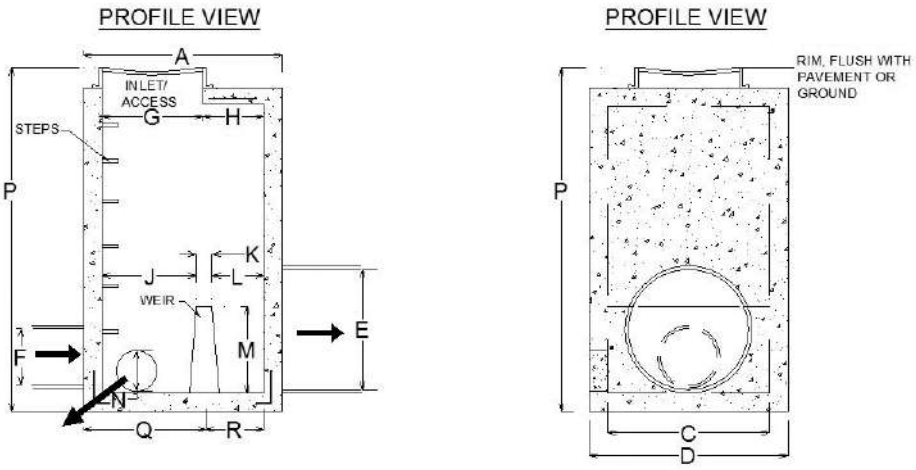


FIGURE B.8.18 TYPICAL DIVERSION STRUCTURE.

B.8.7: IMPERMEABLE LINERS AND HYDRAULIC RESTRICTION BARRIERS

The most ideal configuration, from a stormwater pollutant-removal perspective, is to infiltrate as much runoff as possible. Types of clay that have a high potential for expansion when saturated should be protected from moisture in load bearing conditions; however, expansive clays do not preclude infiltration. When infiltrating SCMs are hydraulically isolated from structures (by vertical or horizontal distance or by using hydraulic restriction layers), systems installed in tight clays soils can still experience significant volume reductions (Fassman and Blackbourn 2010). In situations where conditions require limiting infiltration, two basic options can be used for hydraulic restriction layers.

The preferred option is to restrict lateral flow while allowing for deep percolation infiltration of stormwater. To allow infiltration, the bottom of the bioretention area should remain unlined. The hydraulic restriction layer should extend the full depth of the media to the base of the drainage layer in situations where underdrains are required. In situations where underdrains are not required, the hydraulic restriction layer should extend to a depth where saturation will not affect any adjacent load-bearing soils. Areas that have a potential for settling under saturated conditions should be protected from lateral flows. An example is shown in Figure B.8.19.

In situations where infiltration is not possible, because of limiting soil capacity or aquifer protection (i.e., Edwards Aquifer Recharge, Contributing, and Transition zones), the entire perimeter of the soil media should be lined to prevent infiltration into the existing soils while gaining some pollutant removal from the soil media. Infiltration pathways might also need to be restricted using impermeable barriers because of the close proximity of roads, foundations, other infrastructure, or hotspot locations as determined in the geotechnical investigation. A full geotechnical investigation should be performed by a licensed soil scientist or geotechnical engineer. That should be done for all sites to determine the effect of infiltration, including the appropriate depth and type of the hydraulic restriction layer.

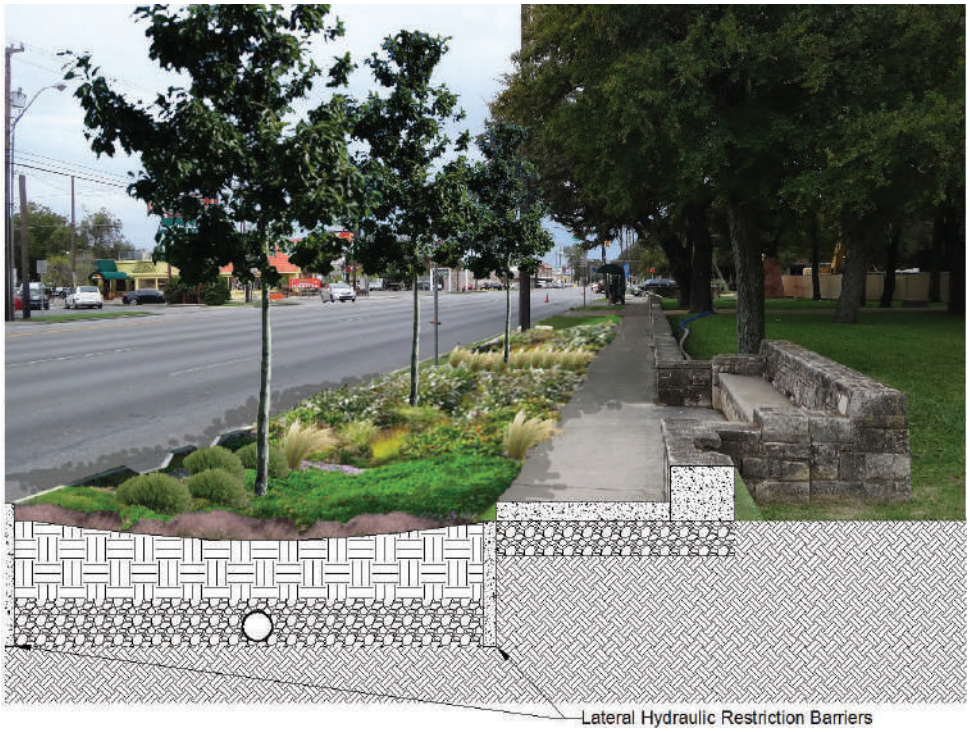


FIGURE B.8.19 LATERAL HYDRAULIC RESTRICTION LAYERS IN A ROADSIDE BIORETENTION AREA PREVENT HORIZONTAL SEEPAGE WHILE ALLOWING INFILTRATION AT A SAFE DEPTH.
Broadway Street, Witte Museum, San Antonio (rendering). Source: Bender Wells Clark Design

In the Edwards Aquifer Recharge, Transition, and Contributing zones, three types of hydraulic restriction layers are recommended: clay liners, concrete, or geomembranes (Barrett 2005). Specifications for clay liners are provided in Table B.8.3.

If geomembrane is used, it should be a minimum of 30 mils thick and ultraviolet resistant. A suitable geotextile fabric should be placed on the top and bottom of the membrane for puncture protection and the liners covered with a minimum of 6 inches of compacted topsoil. The topsoil should be stabilized with appropriate vegetation. The geotextile fabric (for protection of geomembrane) should be nonwoven geotextile fabric and meet the specifications in Table B.8.4. Construction plans should specify the method for sealing the seams of the geomembrane (per manufacturer recommendations). Seams are typically head sealed by the manufacturer but can be sealed in the field following ASTM D7408 standards and all manufacture requirements. An example of a geomembrane liner is shown in Figure B.8.20.

TABLE B.8.3 CLAY LINER SPECIFICATIONS (BARRETT 2005)

PROPERTY	TEST METHOD	UNIT	SPECIFICATIONS
Thickness	--	inch	12
Permeability	ASTM D-2434	cm/sec	1×10^{-6}
Plasticity Index of Clay	ASTM D-423 & D-424	%	Not less than 15
Liquid Limit of Clay	ASTM D-2216	%	Not less than 30
Clay Particles Passing	ASTM D-422	%	Not less than 30
Clay Compaction	ASTM D-2216	%	95% of Standard Proctor Density

Source: Barrett 2005

TABLE B.8.4 PROTECTIVE GEOTEXTILE FABRIC SPECIFICATIONS (BARRETT 2005)

PROPERTY	TEST METHOD	UNIT	SPECIFICATIONS
Unit weight	--	oz/yd^2	8
Filtration rate	ASTM D-423 & D-424	0.08	0.08
Puncture strength	ASTM D-751*	lb	125
Mullen burst strength	ASTM D-751	psi	400
Tensile strength	ASTM D-1682	lb	200
Equiv. opening size	US Standard Sieve	No.	80

Source: Barrett 2005

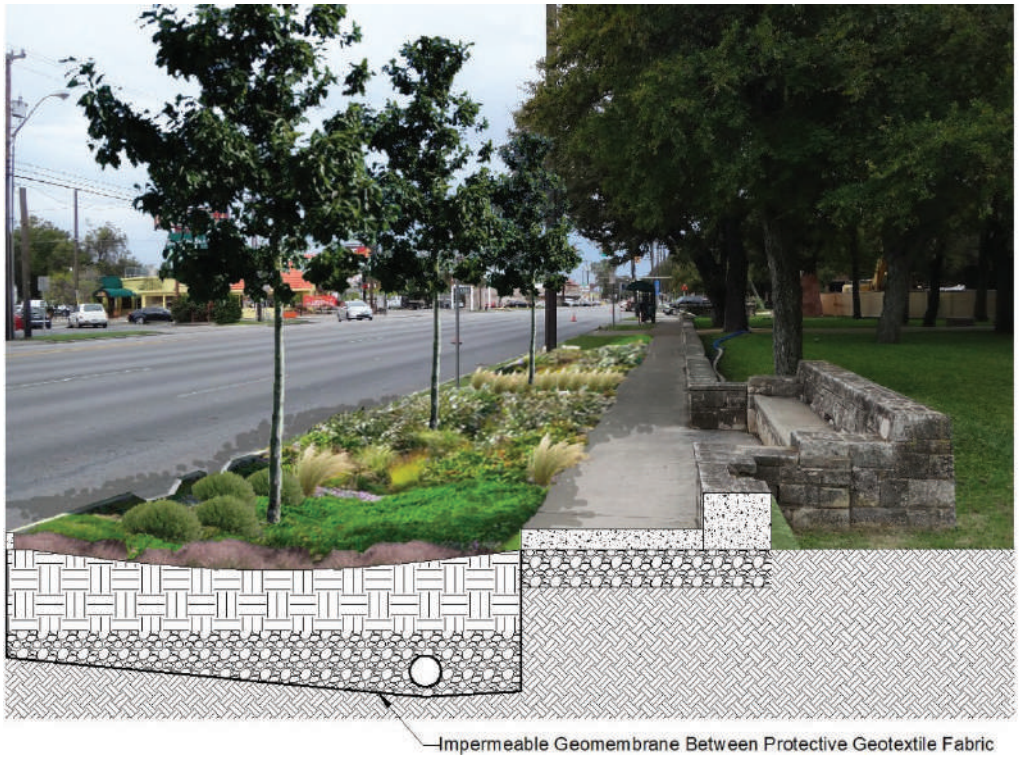


FIGURE B.8.20 BIORETENTION AREA WITH GEOMEMBRANE LINER.
Broadway Street, Witte Museum, San Antonio (rendering)

B.8.8: OUTLET STRUCTURES

Stormwater control measures shall be designed to reduce the post-development peak rates of discharge to existing pre-development peak rates of discharge for the 2-, 10-, 25-, and 100-year storm events at each point of discharge from the project or development site. If off-site flows are conveyed through a SWM facility, the outlet structure must be designed to safely pass the 100-year fully developed off-site flows in accordance with the Safety Criteria set forth in Appendix E: Stormwater Detention Guidance, Section 3.1.3. Detention ponds are typically designed with an outfall structure and either an emergency spillway or an overflow structure. The emergency spillway or overflow structure will release runoff from a rainfall event larger than the design storm or in the event of a blocked outfall pipe.

Sediment markers are required at the outflow points for facilities with forebays, or at facilities where a high volume of sediment is expected. This includes bioretention facilities, wet ponds, extended detention ponds, and wetlands. The sediment markers will indicate the volume of sediment within a facility, and help promote prompt maintenance of each of the facilities.

B.8.8.1: Outfall Control Structures

There are two basic types of outlet control structures: those incorporating orifice flow and those incorporating weir flow. Rectangular and V-notch weirs are the most common types. For detailed sizing guidelines, see Appendix E: Stormwater Detention Guidance, Section 3.1.4. The outlet of a stormwater control facility should be located at the opposite end from the inlet. This will minimize the approach velocity of the overflow, and will also allow for more water quality treatment within the facility.

Outlet pipes materials include corrugated metal or reinforced concrete. Figure B.8.21 presents a possible outflow structure configuration for extended detention facilities. A reverse slope outflow pipe design is preferred if a second stage micropool is provided in the facility. Otherwise, the facility's drawdown time should be regulated by a gate valve or orifice plate located downstream of the primary outflow opening. In general, the outflow structure should have a trash rack or other acceptable means of preventing clogging at the entrance to the outflow pipes.

The outflow structure should be sized to allow for complete drawdown of the water quality volume in 48 hours. No more than 50% of the water quality volume should drain from the facility within the first 24 hours. A valve or orifice can be used to regulate the rate of discharge from the basin.

The facility should have a separate drain pipe with a manual valve that can completely or partially drain the pond for maintenance purposes. To allow for possible sediment accumulation, the submerged end of the pipe should be protected, and the drain pipe should be sized one pipe schedule higher than the calculated diameter needed to drain the pond within 24 hours. The valves should be located at a point where they can be operated in a safe and convenient manner.

For online facilities, the principal and emergency spillways must be sized to provide 1.0 foot of freeboard during the 25-year event and to safely pass the flow from 100-year storm.

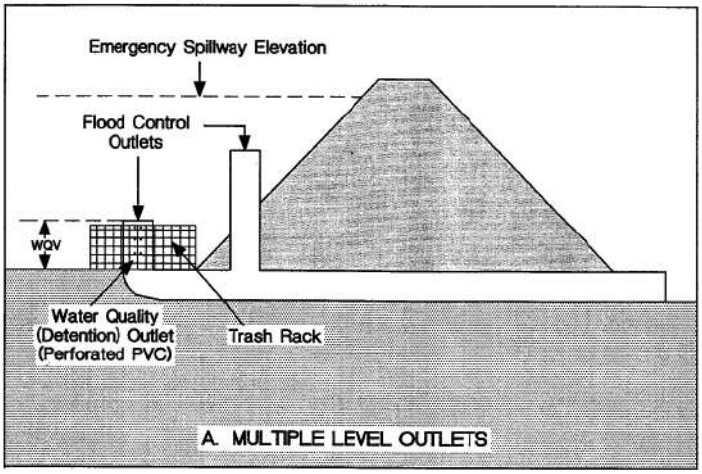


FIGURE B.8.21 SCHEMATIC OF DETENTION BASIN OUTLET STRUCTURE.

Detention pond outlets require detailed analysis to determine the size and type of outlet required based on the rate and volume of the outfall. Several types of modeling software exist that can evaluate basin outfalls. For instance, the United States Army Corps of Engineers Hydraulic Engineering Center published a Hydrologic Modeling System (HEC-HMS) that can evaluate the tailwater of detention pond or basin. Additional tailwater analysis, as well as determining the size and material of the outfall pipe, can be performed using the Federal Highway Administration's HY-8 software.

There are many different types of tailwater, depending upon the detention basin location and the presence or absence of a pump station. HEC-HMS allows the user to select the type of tailwater present in order to accurately depict the behavior of the detention

basin. The program also allows for analysis of an outfall structure, including inputs for the pipe material, size, and orifice.

Discharge from a SCM should be non-erosive. Riprap or other stabilization measures are recommended to prevent erosion and scour. In addition, the discharge from the outflow structure, overflow structure, and emergency spillway should not cause adverse downstream impacts to adjacent properties and/or structures.

B.8.8.2: Overflow and Emergency Spillways

The purpose of an emergency spillway is to provide a controlled overflow relief for storm flows in excess of the design discharge for the storage facility. A suitable emergency spillway section for a detention facility is a broad crested weir, cut through the original ground next to the embankment. The transverse cross section of the weir is typically trapezoidal in shape. The invert of the spillway at the outfall should be at an elevation 1 to 2 feet above the maximum design storage elevation.

Overflows and emergency spillways can also be modeled in HEC-HMS. Inputting the overflows and spillways into the model is a way to accurately determine the detention basin outfall rate and volume, and can be utilized to ensure the safety of the structure.

Depending on the size of the basin embankment, the detention basin embankment may qualify as a dam. In those cases, the basin needs to be approved by the Dam Safety Team of the TCEQ for safety.

B.8.9: UTILITIES

When implementing SCMs, avoid utilities where possible. In many cases, the SCM can be shifted in the landscape to prevent implementation over utilities. In cases where utilities cannot be avoided, take care to prevent effects from infiltration or saturation by using hydraulic restricting layers to direct infiltration away from the utility. The utility should pass through the hydraulic restriction layer, and the liner should be appropriately sealed to prevent any lateral seepage from the SCM. Liners can be easily sealed by using a patch that adheres to the utility line and sealed directly to the liner. Local plumbing codes should be reviewed for restrictions pertaining to water and sewer utilities.

The location of future utilities should also be considered in the site layout and location of SCMs. Long, linear SCMs, such as a bioretention area or bioswale in the right-of-way, should have periodic breaks to allow for future utility trenches. At least one access point should be placed along any SCM for each parcel where there is a separation or break in the liner for a utility trench. SCMs in such a scenario should be designed as separate systems with separate hydraulic restriction layers, but they could be connected at the subsurface through the underdrain or at the surface by a trench with a grate similar to a covered curb cut. For more details, see Connectivity.

B.8.10: CONNECTIVITY

When SCMs are implemented in the right-of-way and parking lots, it is important to maintain pedestrian access routes to prevent disturbance to the SCM, prevent harm to the public, and provide connections for future utilities. It is also important that sections of the SCM remain hydraulically connected to fully use as much of the SCM as possible. SCMs should be connected by open channels covered with an appropriate grate to allow visual inspection of the channel and ease of maintenance. Culverts can be used for larger facilities, but they should be inspected regularly for blockages. Figure B.8.22 shows pedestrian access over SCMs while maintaining appropriate hydraulic connectivity.



FIGURE B.8.22 ACCESS OVER LINEAR SCMS.
(Top left) Los Angeles, California. Source: Tetra Tech, Inc. (Top right) Bender Wells Clark Design, San Antonio, Texas. (Bottom) Raleigh, North Carolina. Source: Tetra Tech, Inc.

B.8.11: ADA REQUIREMENTS

SCMs typically require surfaces with little to no slope, therefore, Americans with Disabilities Act (ADA) requirements are rarely an issue. However, in areas with high levels of pedestrian traffic, some effort should be made to delineate the SCM. Several options—including low-level and decorative fencing, such as the one shown in Figure B.8.23, or a low-profile curb, as shown in Figure B.8.24, can often be used to delineate the space around the SCM and alert pedestrians of the change in grade.



FIGURE B.8.23 LOW-LEVEL FENCING.
Portland, Oregon. Source: Portland BES



FIGURE B.8.24 LOW-PROFILE CURBING.
Greensboro, North Carolina. Source: Tetra Tech

B.8.12: REFERENCES

Barrett, M.E. 2005. *Complying with the Edwards Aquifer Rules. Technical Guidance on Best Management Practices*. RG-348. Prepared for Texas Commission on Environmental Quality, Field Operations Division, Austin, TX.

Fassman, E.A. and S. Blackbourn. 2010. Urban runoff mitigation by a permeable pavement system over impermeable soils. *Journal of Hydrologic Engineering* 15(6):475-485.

TxDOT (Texas Department of Transportation). 2011. Chapter 13, Section 2. Soil Erosion Control Considerations. *Hydraulic Design Manual.* Austin, TX.

UDFCD (Urban Drainage and Flood Control District). 2010. T-12 Outlet Structures. *Urban Storm Drainage Criteria Manual, Volume 3 – Best Management Practices*. Denver, CO. http://www.udfcd.org/downloads/down_critmanual_vollll.htm.

APPENDIX C. STORMWATER DETENTION GUIDELINES

1: INTRODUCTION.....C:2
1.1: GENERAL C:2
1.2: DETENTION SCMS C:2
1.2.1: TYPES C:2
1.2.2: FUNCTIONALITY C:3
1.2.3: LIMITATIONS C:3
1.2.4: SAFETY CRITERIA FOR SWM PONDS C:3
1.2.5: OPERATION AND MAINTENANCE C:5
1.2.6: MAINTENANCE ACCESS C:5

2: AESTHETICALLY ENHANCED DETENTION BASINS.....C:6
2.1: BACKGROUND C:6
2.1.1: WHY ARE AESTHETIC STORMWATER DETENTION AND WATER QUALITY FACILITIES IMPORTANT TO DEVELOPERS? C:6
2.1.2: STORMWATER BASIN ZONES C:7
2.2: TOOLS FOR SUCCESS C:8
2.2.1: SITING AND FUNCTIONALITY C:8
2.3: PHYSICAL CHARACTER AND ARCHITECTURAL ELEMENTS C:9
2.4: LANDSCAPE DESIGN C:10
2.5: GUIDELINES FOR SITE ENHANCEMENT C:11

1: INTRODUCTION

Hydrology is the study of water, its source, distribution, quantity, quality, and movement. For the purpose of this Appendix, the hydrology guidance will be limited to surface hydrology; the portion of the hydrologic cycle that deals specifically with precipitation, infiltration, and surface runoff.

This appendix describes the specific precipitation data which has been defined by federal and state agencies and regionalized to Hays County. This chapter will also address infiltration and surface runoff by providing guidance on Methods of Analysis that range from small local analysis (i.e. Rational Method) to the hydrograph methods as well as guidance on probable maximum precipitation, with equation parameters specific for this region. The selection of these methods will be determined by drainage area size and purpose of the study. The proper application of these methods will generate discharge values that may be used for planning, design, mitigation, or regulation.

1.1: GENERAL

Stormwater Management (SWM) ponds may be of two basic types: on-site/off-site and regional. In general, on-site or off-site ponds are those which are located off-channel and provide stormwater management for a particular project or development. Regional ponds are designed to provide stormwater management in conjunction with other improvements on a watershed-wide basis. SWM ponds may be further classified as retention or detention ponds and may incorporate water quality stormwater control measures as defined in this manual. The performance and safety criteria in this section apply to all ponds which provide management of peak rates of stormwater runoff regardless of type.

1.2: DETENTION SCMS

Detention SCMs are structural facilities that are constructed on a site to capture stormwater runoff from a project area and reduce downstream water quantity and the frequency and intensity of flood events. Detention SCMs are primarily ponds and can be designed to be either a wet pond or dry pond. Detention SCMs require detailed engineering design. Guidance for this design can be found in *TCEQ Edwards Aquifer Technical Guidance on Best Management Practices* however, general information is found in the following sections.

Rate of runoff and facility design features must be designed using the hydrologic methods described in Section III B.1 of this manual. Computation of the rate of runoff must be based on an assumption of a fully developed contributing drainage area or watershed. Detention will be waived for single-family residential and small projects with less than 5,000 square feet of impervious cover, including buildings, parking lots and sidewalks.

To facilitate detention basin appearance, designers are required to follow the guidance found in this appendix for Aesthetically Enhanced Detention Basins.

1.2.1: Types

1.2.1.1: Wet Ponds

Wet ponds, also known as wet basins, detain stormwater using an in-line permanent pool or pond. Wet ponds are primarily used to detain stormwater runoff and prevent downstream flooding. These basins minimize peak flood events by releasing stormwater runoff slowly, often after the storm event has peaked. Wet ponds can allow for some water quality improvements by settling sediment, organic nutrients, and trace metals. Biological processes that naturally occur within a wet pond can also remove these pollutants.

Wet ponds and wet basins require a dependable water source. This source can be surface water or groundwater, but a permanent pool elevation must be maintained and potable water should not be used for filling the pond. This type of stormwater control measure can treat a large drainage area when properly designed and can also provide an amenity for the community.

1.2.2.2: Dry Ponds

Dry ponds, or extended detention dry ponds, are similar to wet ponds. The primary difference is that dry ponds do not permanently store water. Instead, dry ponds are designed to be periodically inundated with water during storm events. Dry ponds are intended to reduce the peak runoff from storm events and have the potential to decrease downstream flood intensity and duration.

Dry ponds can also be designed to provide treatment of stormwater to enhance water quality. However, the primary function of dry ponds is to detain stormwater for flood control. When dry detention facilities are utilized for water quality treatment, additional SCMs such as grassy swales or vegetated filter strips are required in order to reduce levels of suspended solids.

Dry detention facilities are often used in housing developments and can be used in business and retail developments. They are capable of receiving stormwater from a large drainage area when properly designed.

1.2.2: Functionality

Deciding on whether the detention SCM should be a wet pond or dry pond depends on the SCM site characteristics. Soil type and permanent water source are the two most important factors to keeping water in a wet pond year round. Both wet and dry ponds require engineered design in order to ensure that they are sized as required and that stormwater runoff will be properly detained. Stormwater from the facility should be released in a controlled manner to prevent flooding downstream.

Wet ponds and facilities with permanent pools should be designed to prevent the congregation of mosquitos and other vectors. Consideration of permanent pond elevation should be given to areas where evapotranspiration greatly exceeds precipitation (which is most of the Edwards Aquifer region).

1.2.1.1: Protection of Downstream Channels

Detention SCMs protect downstream resources by controlling and improving the quantity and quality of runoff. Depending on whether the SCM is designed with an outflow or not, the runoff is either infiltrated or slowly released after the passing of the storm event. Regardless, the SCM serves as a temporary or permanent flow-reduction device and decreases the volume of the peak storm event at locations downstream. Reducing the downstream volume decreases the severity of storm events. It also reduces the potential for downstream erosion due to scour

1.2.2.2: Groundwater Recharge

Many post-construction detention SCMs rely on infiltration to treat and manage stormwater runoff. By allowing stormwater runoff to infiltrate into the ground, SCMs can replenish groundwater and increase the base flow of streams. With an increasing demand for water resources, ensuring that stormwater runoff re-enters the local water supply system helps to replenish our limited supply. This benefit is only applicable to detention SCMs that are constructed without a liner.

1.2.3: Limitations

Limiting factors for where to locate detention SCMs can include water table depth, site slope, soil characteristics, space requirements, existing utilities, depth to bedrock, existing trees, evapotranspiration rates, and pest control when siting and designing detention SCMs for stormwater management and flood control.

Soil conditions are very important to consider when selecting locations for the facilities. Areas in Karst topography or areas with highly erodible soils will require a geomembrane or clay liner to prevent scour and potential contamination of groundwater. A list of highly erodible soils is provided in Appendix B, but designers should perform soil infiltration tests and soil profiling at the site before designing the stormwater facility.

1.2.4: Safety Criteria for SWM Ponds

All ponds shall meet or exceed all specified safety criteria. Use of these criteria shall in no way relieve the engineer of the responsibility for the adequacy and safety of all aspects of the design of the SWM pond.

A. The spillway, outfall, embankment, and appurtenant structures shall be designed to safely pass the design storm hydrograph with the freeboard shown in the table below. All contributing on-site drainage areas, and off-site areas which are routed through the SWM pond, shall be assumed to be fully developed to properly size the spillway, outfall, embankment and appurtenant structures. Any orifice with a dimension smaller than or equal to 12 inches shall be assumed to be fully blocked. For all spillways (especially enclosed conduits), the ability to adequately convey the design flows must take into account any submergence of the outlet, any existing or potential obstructions in the system and the capacity of the downstream system.

DETENTION POND CLASS	DESIGN STORM EVENT	FREEBOARD ON TOP OF EMBANKMENT, FEET
On-site/Off-site		
Small (DA $<$ 25 acres)	100-year	0
Large (25 $\leq$ DA $<$ 64 acres)	100-year	1.0
Regional DA $\geq$ 64 acres	100-year	2.0

B. Any SWM pond with a storage size and dam height that meets the Texas Commission of Environmental Quality's (TCEQ's) classification criteria for a regulated dam must comply with TCEQ's criteria set forth in TAC 30, Chapter 299.

a. The height of the hydraulic structure (dam) is measured from the top of the structure to the downstream

intersection of the structure and the natural or excavated ground, whichever is lower.

b. The PMF is computed by using the probable maximum precipitation (PMP) values.

c. Dam Safety Certification Statement:

I [name of professional engineer] Texas license number [number] certify that the design of the dam in this set of plans meets all the design and analysis criteria set forth in TCEQ's in TAC 30, Chapter 299.

d. SWM ponds that are considered may not be designed or constructed with any trees or other woody vegetation on the dam structure or within 20-feet of the upstream or downstream toe of the dam. This 20-foot clear zone must be called out on the site plan and for City maintained facilities must be part of the drainage easement dedicated for the dam facility. The toe of the dam is the junction of the constructed dam structure with the natural ground.

e. SWM ponds that are considered dams may not have permanent irrigation systems installed on the dam.

f. SWM ponds that are considered dams must be vegetated with grasses that do not exceed 12-inches in height and can be mowed as frequently as weekly. Examples include Bermuda grass and buffalo grass.

g. SWM ponds that are considered dams shall provide a fixed vertical marker on or near the emergency spillway indicating the water surface elevation relative to the top of the main embankment. The markings should be in half foot increments, viewable from the furthest point of access, and must be retro reflective as defined by the Texas Manual on Uniform Traffic Control Devices (TMUTCD).

C. The minimum embankment top width of earthen embankments shall be as follows:

TOTAL HEIGHT OF EMBANKMENT, FEET	MINIMUM TOP WIDTH, FEET
0-6	4
6-10	6
10-15	8
15-20	10
20-25	12
25-35	15

D. The constructed height of an earthen embankment shall be equal to the design height plus the amount necessary to ensure that the design height will be maintained once all settlement has taken place. This amount shall in no case be less than 5% of the total fill height. All earthen embankments shall be compacted to 95% of maximum density in accordance with City of San Marcos standard specifications.

E. Earthen embankment side slopes shall be no steeper than 3 horizontal to 1 vertical. Slopes must be designed to resist erosion to be stable in all conditions, and to be easily maintained. Earthen side slopes for regional facilities shall be designed on the basis of appropriate geotechnical analyses.

F. Detailed hydraulic design calculations shall be provided for all SWM ponds. Stage-discharge rating data shall be presented in tabular form with all discharge components, such as orifice, weir, and outlet conduit flows, clearly indicated. A stage-storage table shall also be provided. In all cases the effects of tailwater or other outlet control considerations should be included in the rating table calculations.

G. When designing ponds in series (i.e., when the discharge of one becomes the inflow of another), a licensed engineer in the State of Texas engineer must submit a hydrologic analysis, which demonstrates the system's adequacy. This analysis must incorporate the construction of hydrographs for all inflow and outflow components.

H. Storm runoff may be detained within parking lots. However, the engineer should be aware of the inconvenience to both pedestrians and traffic. The location of ponding areas in a parking lot should be planned so that this condition is minimized. Stormwater ponding depths (for the 100-year storm) in parking lots are limited to an average of eight (8) inches with a maximum of twelve (12) inches.

I. All pipes discharging into a public storm drain system shall have a minimum diameter of 18 inches and shall be constructed of reinforced concrete. In all cases, ease of maintenance and/or repair must be assured.

J. All concentrated flows into a SWM pond shall be collected and conveyed into the pond in such a way as to prevent erosion of the side slopes. All outfalls into the pond shall be designed to be stable and non-erosive.

1.2.5: Operation and Maintenance

The major goal of detention SCM operation and maintenance (O&M) is to ensure that it is meeting the specified design criteria for stormwater flow rate, volume, and water quality control functions. If detention systems are not properly maintained, they can lose their ability to detain stormwater. This can lead to downstream flooding, sedimentation and stagnation in the pond, and public nuisances such as insects, algal blooms, and unpleasant odors.

Maintenance of flood control SCMs is similar to maintenance of other SCMs. Mowing, clearing of woody debris and trash removal must be carried out on a regular basis to ensure the basin and outfalls do not get clogged. Trees and root systems should be removed to prevent structural damage to the outfall structure and to prevent compromising the stability of the embankments. Yard clippings and vegetation should never be left in the basin of an SCM. Areas that have been exposed to scour and erosion must be repaired as soon as possible. Damage to inlets, outfall structures, and other key design components should also be repaired upon discovery. Inspectors should ensure that any public safety measures are maintained properly.

The City of San Marcos LDC Section 6.1.1.4 Compliance with City and TCEQ Rules states:

> All temporary and permanent Best Management Practices (BMPs) required in the approved watershed protection plan must be constructed, operated and maintained in accordance with the standards, criteria and requirements in the Section 86.531 of the San Marcos MS4 Ordinance found in Chapter 86, Article 8, Division 2 of the San Marcos City Code, the City's Stormwater Technical Manual, TCEQ Edwards Aquifer Protection Program rules and the TCEQ Technical Guidance on Best Management Practices, RG 348.

Wet ponds and sediment forebays will eventually accumulate enough sediment to significantly reduce the storage capacity within the permanent pool. As can be expected, the accumulated sediment can reduce both the appearance and the pollutant removal ability of the pond. Sediment markers are required at the inflow and outflow of all wet pond facilities. Sediment accumulated in the forebay area should be removed from the facility every two years to prevent accumulation in the permanent pool. Dredging of the permanent pool should occur about every 5 to 10 years. However, facilities that lack a stable upstream drainage area, or are subject to erosion by other means, will likely require sediment removal methods to be utilized more frequently.

1.2.6: Maintenance Access

A 12-foot maintenance access strip is required around the perimeter of detention SCMs for single-family or duplex residential development and/or for such facilities that will be maintained by the City. This 12 foot maintenance access strip shall be outside the toe of any fill slope and the top of any cut slope and shall not have a post construction longitudinal slope greater than 15% nor a post construction transverse slope greater than 5%, shall not have a vertical grade break of greater than 12%, shall have an inside turning radius of no less than thirty-six (36) feet, shall be cleared and graded, and shall have vertical clearance from existing and proposed vegetation and all other objects of no less than 14 feet.

A setback shall be provided for all detention and retention facilities for single-family or duplex residential development and/or for such facilities that will be maintained by the City so that no such facility shall be located within 50 feet of a residential structure. All detention and facilities that will be for single-family or duplex residential development and/or for such facilities maintained by the City shall have an all-weather maintenance equipment access ramp into each basin of the facilities, with a longitudinal slope not to exceed 7:1, a transverse slope not to exceed 5%, no vertical grade break greater than 12%, no vertical curve with a grade change greater than 1% per horizontal foot, an inside turning radius of no less than 36 feet, a minimum width of 12 feet and a clear distance from the bottom of the ramp to any facility interior slope of at least 15 feet.

2: AESTHETICALLY ENHANCED DETENTION BASINS



FIGURE C.1 DETENTION BASIN WITH AESTHETIC ENHANCEMENTS, SAN ANTONIO, TX – SOURCE: TETRA TECH

2.1: BACKGROUND

Managing urban stormwater runoff, providing stormwater detention and water quality are regulatory requirements. There are many strategies that may be used to address these requirements. The most common conventional method is to use various stormwater control measures (SCMs) described in this manual. The information contained in this section is to be used as a guide to help the design professional when an above-ground stormwater detention and water quality facility is chosen as a method to control stormwater runoff.

Incorporation of detention and water quality facilities into a proposed development site should be performed early in the design process that is described in Appendix B. It is the goal of the City to have the aesthetic elements incorporated into the project at the earliest stages of design. This section will help the design professional in meeting this objective.

2.1.1: Why are aesthetic stormwater detention and water quality facilities important to developers?

Aesthetically enhanced storm drainage facilities add to property value and their positive visual appearance improves the quality of life in our community.

As a developer, there is a significant marketing opportunity to adding stormwater features. People are drawn to water and the natural environment. Instead of focusing on the minimum technical design criteria for meeting these requirements, we encourage developers to seize the opportunity to create water quality and detention ponds that will help create more livable, sustainable, and valuable developments. These enhanced facilities will offer resident's opportunities to observe nature, engage in recreation, or simply enjoy the view.

Coupled with water savings that can be achieved through the use of native plant palettes, prudent investment in detention and water quality basins has both an environmental and economic benefit for developers and the communities that surround these spaces.

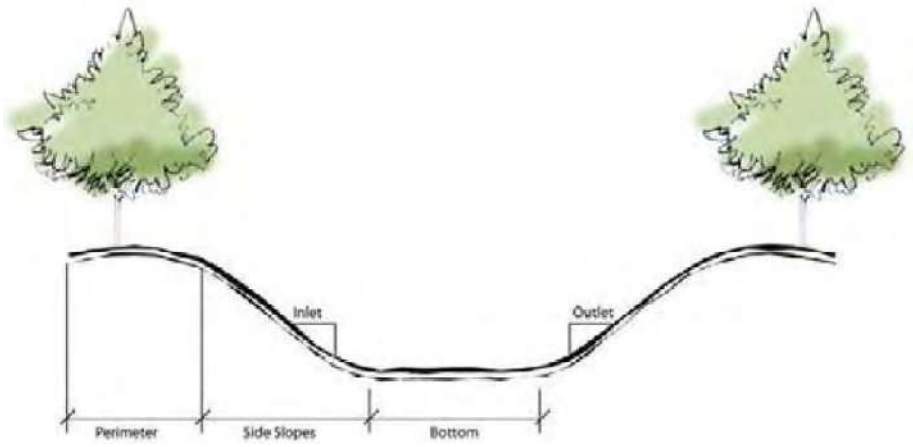


FIGURE C.2

2.1.2: Stormwater Basin Zones

There are many ways to approach the topic of aesthetics. For ease of discussion, we have chosen the approach of segregating a stormwater detention basins into zones.

The facility typically has three different zones: perimeter, side slopes, and bottom. Each zone is unique and each must be addressed to design a pond that is fully incorporated into the site in an aesthetic manner. The previous figure illustrates the difference between the three zones.

Perimeter

The zone surrounding the facility, most likely on the pedestrian grade, provides the greatest opportunity for improving visual impacts with landscaping, benches, walkways, and site furnishings. The shape of the basin should be as natural looking as practical. Avoid "boxy" and geometric features. Maximize the distance between the inlet and the outlet structures to allow for proper treatment of the stormwater runoff.

Side Slopes

This is the zone where the greatest grade changes occur. Basin side slopes should be stable and gentle to facilitate maintenance and access. Consider creating topographic changes that mimic natural conditions by including a variety of slope changes. The tops and the toes of slopes should vary, and there should be an undulation in the shape and grading of the sides of the detention area. Side slopes generally should be no steeper than 3:1. If the detention basin is deep (typically greater than 5 feet), safety benches are typically installed into the side slopes.

Bottom

This is the lowest portion of the facility, and it will likely see the largest fluctuations in water supply. For extended detention basins, trickle or low flows need to be conveyed through this zone, which is designed to have minimum slope.

Careful selection of planting materials is vital to the performance of a stormwater detention facility. Vegetation for all three zones should be carefully selected to allow for nutrient uptake and improve overall aesthetics. The functionality of each zone should be considered when selecting plant types. Vegetation at the bottom of the facility, for instance, is frequently inundated with water, so plants that can tolerate periods of inundation should be selected for the design. Additional information on selecting vegetated material can be found in Section 3.3.



FIGURE C.3 DETENTION POND AND POCKET WETLAND INTEGRATED INTO SITE FEATURES, NORTH CAROLINA, SOURCE: TETRA TECH.

2.2: TOOLS FOR SUCCESS

2.2.1: Siting and Functionality

Goal: Stormwater detention facilities should appear as an extension of the surrounding landscape. They can be designed with attractive, natural-looking features so they become site amenities rather than eyesores.

General

One of the first issues to be addressed when designing detention basins is where it will be situated on the site.

Perimeter

- Naturally, the basin should be placed near the low point of the final graded site. However, it is preferred that the designer ensures that the perimeter of the basin is integrated into adjacent open space and neighboring properties, complements the character of the general neighborhood, and blends with the individual site the pond is serving.
- Incorporating other uses into the basin is strongly encouraged, (e.g., strategically placing a bench to provide a quiet place for reading a book or enjoying the surrounding wildlife, creating an area for a neighborhood picnic or volleyball game, etc.).

Side Slopes

- Ponds located near high pedestrian activity areas should address safety concerns. Gentle slopes are preferred versus steeper slopes or walls with protective fencing. Safety benches may be required if the detention or water quality basin is too deep.
- When a basin is adjacent to a street right-of-way or located in a highly visible area, special attention should be given to incorporating aesthetic elements into the pond design.

Bottom

- Stormwater facilities not only need to meet technical standards, but also serve as an amenity. To the maximum

extent feasible, the stormwater facility should be located, designed and managed for use as year-round open space and recreational area.

- Wherever possible, stormwater facilities should be located in pervious landscaped areas or integrated with pervious pavement under parking lots. Detention ponds located in parking lots are discouraged given the potential for inconvenience, hazards, and damages resulting from ponding and possible ice buildup.
- The design of these facilities must consider maintenance issues such as weed control measures, maximum heights of grasses/turf, mowing, and water retention/release rates that can result in soggy ground at the bottom of the ponds.

2.3: PHYSICAL CHARACTER AND ARCHITECTURAL ELEMENTS

Goal: Integrate building architecture and site elements into the pond design and drainage components to create an aesthetically pleasing community asset.

General

- Instead of being seen as unattractive left-over space, stormwater basins can be designed as attractive features of the site and overall surrounding environment. Stormwater facilities can be designed to appear as an extension of the surrounding landscape.
- Often, a visually appealing basin will require more land than the most efficient engineering design. However, it is recognized that the financial cost of this additional land can be more than offset by the increased project value with an integrated, accessible, and usable recreational or open space.
- Consider curvilinear (non-rectangular) design that can be incorporated into the surrounding area. Consider following contours, rather than creating unnatural sharp corners.
- If topography of the site dictates that the basin is located near an entrance and/or in a highly visible area, it should be integrated into the environment as a visually attractive feature. Perhaps the basin may even create a focal point for the site (e.g. water features, a sign in a retaining wall, etc.).
- The use of concrete and other impervious materials should be limited to areas necessary to prevent erosion that cannot be accomplished using other stormwater design or landscape techniques. Such erosion devices should either be buried below live plant materials or designed in a naturalized manner (i.e., textured and appropriately colored).

Perimeter

- Existing drainage patterns should be preserved whenever possible.
- Consider maximizing passive and active recreational activities, e.g. incorporating the detention pond perimeter into other on-site features such as strolling and running trails, viewpoints, and picnic and sitting areas.
- Detention ponds should be designed to avoid the need for fences. When this cannot be accomplished, consider choosing a material and design that complements the building structure and overall site. Encourage the use of ornamental fences such as wrought iron, wooden or masonry for safety along steep side slopes or changes in grade only, not around the entire pond. Chain link fencing is prohibited. Use fences for safety along steep side slopes only, not around the entire pond. When appropriate, consider native shrubs and other vegetation in lieu of fences.
- Using excessively high berms to increase pond capacity is discouraged. When necessary, berms can be contoured to take on a naturalized shape.

Side Slopes

- Engineering structures such as forebays, surface weirs, inlet structures, etc. should be strategically placed to minimize visibility. When possible, they should be embedded into the sloped embankment rather than being exposed.
- Concrete rip rap that conveys stormwater runoff from impervious surfaces (such as parking lots) to the pond bottom should be incorporated into the overall design and be designed attractively.
- Maintenance access must be considered for trash and sediment removal. Maintenance equipment must be able to safely reach the bottom of the facility and have adequate space to operate and turn. Consider using gravel or a “grasscrete” like product instead of asphalt or concrete for access roads.

Retaining Walls

One way to reduce the space required for a basin is to increase its depth. Building berms or vertical walls can reduce the pond footprint. However, retaining walls are discouraged because they often become a visual nuisance and liability for the owner. When the installation of retaining walls cannot be avoided, follow these recommendations:

- For commercial projects and projects in densely developed urban areas, use materials and forms that match or complement materials used for the main building.
- For residential projects and projects in suburban settings, use materials such as rocks and stone that complement the surrounding natural landscape.
- Walled-in or steep-sided basins should be located away from major pedestrian routes, and emergency pedestrian egress routes should be provided.
- Materials, size, and location of the walls need to be chosen to prevent graffiti. Cast-in-place concrete walls are often the most convenient surfaces for graffiti and, therefore, should be avoided. Using textured materials, reducing wall heights and complementing walls with vines and climbing plants can greatly reduce the potential for graffiti.

Bottom

- At concentrated points of inflow, energy dissipation needs to be considered to protect against erosion. When using rock for energy dissipation, the rock should graduate away from the area of hard edge into the surrounding landscape. Other non-functional matching rock should be placed elsewhere on the basin to prevent the energy dissipation from appearing out of context.
- Forebay berms, when required, can be made of soil rip-rap and vegetated.
- Trickle channels protect detention and extended detention basin bottoms from becoming boggy, and enable easier maintenance (mowing and cleanup). Although concrete trickle channels are necessary due to bottom slope, soft bottom trickle channels are encouraged and can function well with sufficient bottom slope.
- Use landscaping to soften engineering stormwater structures as much as possible.

2.4: LANDSCAPE DESIGN

Goal: Choose appropriate plant materials for each zone and maximize the amount of landscape to the greatest extent possible

The overall goals for basin aesthetics in relation to planting design include choosing appropriate plant material for each zone and maximizing the amount of landscape to the greatest degree possible. Planting design should reflect the character of the area.

- Where possible, design with a more native palette that is more drought tolerant. Once established, such ponds rarely need mowing or weeding. Use of bluegrass and other species requiring irrigation and high maintenance should be reserved for areas with high public use.
- Soil amendments may be necessary, but a soil test should be conducted first to understand if this is a necessary cost for successfully establishing plants.
- Development of a water budget can help assess potential water savings on a site. For example, an established site that irrigates one acre with native grasses versus bluegrass versus can provide a significant annual savings for the City.

Perimeter

- If tied into the larger site design, the landscape should be contiguous to, and consistent with, the rest of the plantings on site. This offers the most rewards to a community.
- When landscaping is needed for screening purposes, tree and shrub massings should be used in clusters. These clusters should be separated by a minimum of approximately 20 feet at maturity to allow some visual access for safety purposes.

Side Slopes

Within the side slope zone, grasses are often used because they are easy to maintain and do not interfere with the function of the pond.

- Cool season turf grass can be used, however, it should not be planted on slopes greater than 3:1. Also, with its shallow rooting structure, it may not be the best plant material for highly erosive areas.

- Warm season grasses allow for less maintenance and, once established, rarely need to be mowed. Therefore, these grasses could be planted on slopes up to 3:1.
- If cool season turf grass is planted within the perimeter zone, it may be much harder to establish warm season grasses on the side slopes because of the transfer of water and/or fertilizers which could encourage turf to migrate. In these situations, either plant warm season grasses on the perimeter or create a no water/ fertilizer buffer.
- Steep slopes should be planted with shrubs and/or groundcovers that don't require mowing.
- Trees are not recommended on steep side slopes. Shredded mulch may be used in these areas, but may require more maintenance.
- If mulch is used, shredded mulch is preferred over large chips in order to reduce the potential for migration, which plugs the outlet structures.

Bottom

Within the bottom zone, plant material should be based upon the use of the pond.

- Dry detention basins may be planted so that sport courts can be used in the bottoms.
- When placing woody vegetation in the bottom, strategically place trees and shrubs in low velocity areas so that they remain in place in times of flooding and do not contribute to the obstruction of downstream structures. Tree and shrub plantings can increase infiltration. Choose plant material that will survive moist soils and periodic inundation.
- Wetland plantings should not include cattails, which have a tendency to grow without encouragement. Proper selection of plant materials can also improve the infiltration potential of landscaped areas since deep-rooted plants help to build soil porosity and increase water absorption.

2.5: GUIDELINES FOR SITE ENHANCEMENT

Aesthetically pleasing storm drainage facilities have multiple benefits to the communities in which they are placed. When designing such a facility, engineers and landscape architects can propose plans that will enhance the community and surrounding landscape. The primary objective of site enhancement is to design features that look as natural as possible.

When designing detention basins, there are many factors to consider to create an aesthetically pleasing, functional facility. These factors are detailed on the following table, but a short summary is included below:

- Utilize native vegetation that are appropriate for each zone.
- Avoid steep side slopes.
- Incorporate safety benches for deep ponds to allow for safe exits in case of accidental submergence.
- Avoid sharp edges on ponds. Follow the natural contours to the maximum extent practicable.
- Sediment markers are required for all SWM ponds, water quality basins, and enhanced detention ponds. The sediment markers must be placed at the inflow and outflow points of each facility.



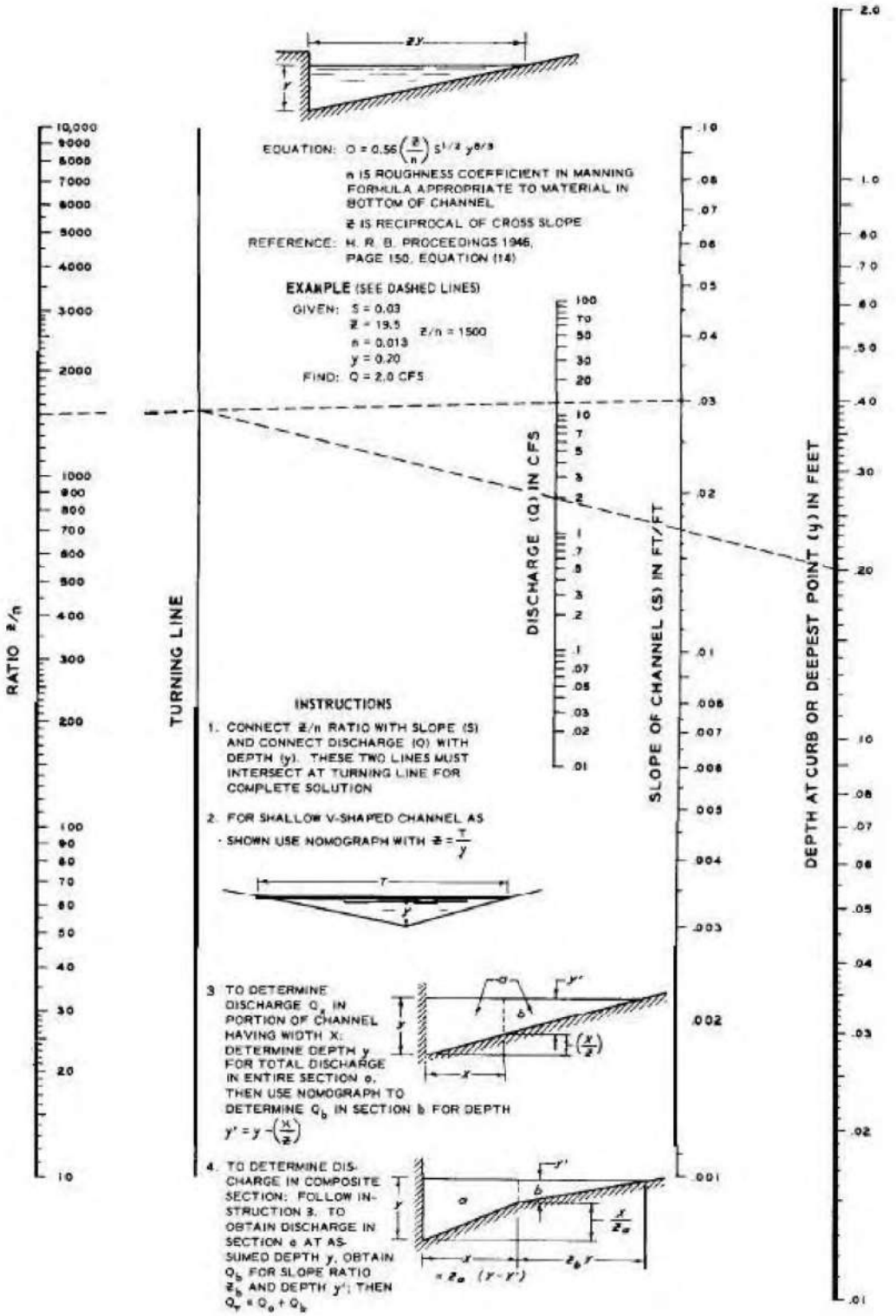
FIGURE C.4 DETENTION BASIN DESIGNED FOR PLAY ACCESS,SAN ANTONIO, TX, SOURCE: TETRA TECH.

Guidelines for Site Enhancement

	Goals	Zone 1: Perimeter	Zone 2: Side Slopes	Zone 3: Bottom	Other
Siting and Functionality	Ponds should be designed with attractive natural-looking features so they become site amenities	Integrate perimeter with the adjacent open space Incorporate recreational uses.	Use low height retaining walls to achieve natural appearance Provide gentle slopes to pond bottom Incorporate aesthetic elements into high visible areas	Use irregular form for pond bottom geometry Design for year round multi- purpose uses when appropriate	Avoid parking lot detention Design maintenance access for all areas Minimize directly connected impervious areas
Physical Character & Architectural Elements	Building architecture and site elements should be integrated into the pond design and drainage elements	Use gentle berming or transition slopes to encourage benches/site furnishings/ walkways Choose site elements that complement buildings	Use retaining wall heights less than 48 inches Separate walls a minimum of 4 feet to allow area for landscaping Design irregular side slopes Construct with natural appearing materials Use graffiti prohibitive materials	Consider outlet structures, trickle channels to prevent soggy areas Provide access ramps for trash removal or mowing Minimize visual impact of drainage structures	Allow visual access
Landscape Architecture	Plant materials should be appropriately used for each zone to maximize the amount of landscape to greatest degree possible	Tie landscape to larger site design Allow visual access for safety purpose when using landscaping for screening	Avoid cool season turf grass on slopes greater than 4:1 Consider warm season grasses to minimize mowing and water requirements Plant steep slopes with shrubs and/or groundcovers	Choose plant material based on use of the pond (active vs. passive) Use high water consumption plants (bluegrass) in active public use areas only Provide visual interest through a mix of trees, shrubs, turf, and groundcovers	Consider the broader ex. benefits education about water, natural systems, ecology, wildlife habitat, and recreation.

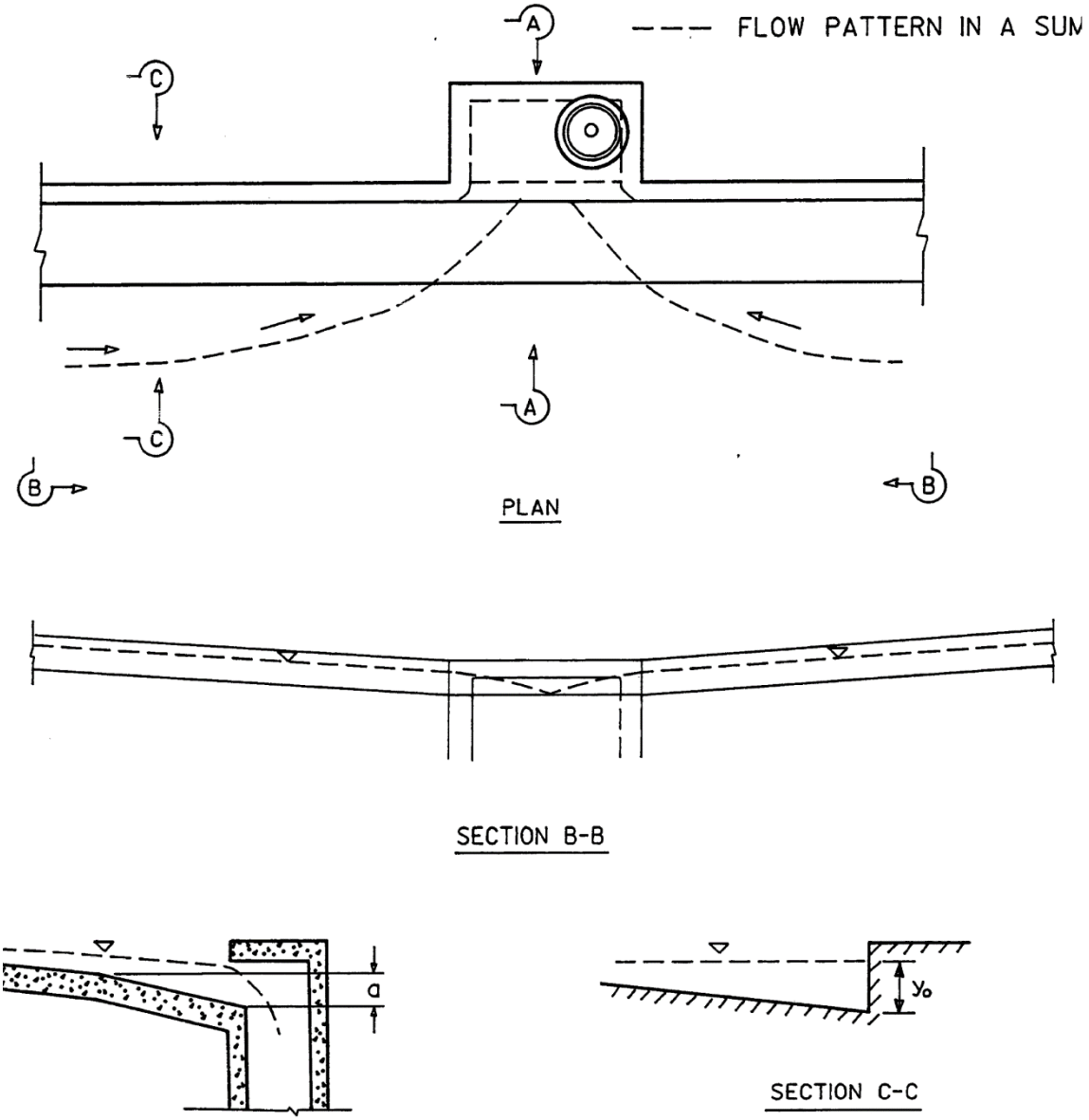
APPENDIX D. DESIGN FIGURES

FIGURE D.1 NOMOGRAPH FOR FLOW IN GUTTERS D:2
FIGURE D.2 CURB OPENING INLET IN A SUMP (TYPE S-1) D:3
FIGURE D.3 GRATE INLET IN A SUMP (TYPE S-2) D:4
FIGURE D.4 COMBINATION INLET IN A SUMP (TYPE S-3) D:5
FIGURE D.5 AREA INLET WITHOUT GRATE (TYPE S-4) D:6
FIGURE D.6 CURB OPENING, INLET ON GRADE (TYPE G-1) D:7
FIGURE D.7 GRATE, INLET ON GRADE (TYPE G-2) D:8
FIGURE D.8 COMBINATION INLET ON GRADE (TYPE G-3) D:9
FIGURE D.9 CURB OPENING INLETS D:10
FIGURE D.10 DEPRESSED CURB OPENING INLET CAPACITY IN SUM P LOCATIONS – ENGLISH UNITS D:11
FIGURE D.11 UNDEPRESSED CURB OPENING INLET CAPACITY IN SUMP LOCATIONS – ENGLISH UNITS D:12
FIGURE D.12 CURB OPENING INLET ORIFICE CAPACITY FOR INCLINED AND VERTICAL ORIFICE THROATS – ENGLISH UNITS D:13
FIGURE D.13 DEFINITION OF DEPTH D:14
FIGURE D.14 GRATE INLET CAPACITY IN SUMP CONDITIONS – ENGLISH UNITS D:15
FIGURE D.15 CURB OPENING AND SLOTTED DRAIN INLET LENGTH FOR TOTAL INTERCEPTION – ENGLISH UNITS D:16
FIGURE D.16 CURB OPENING AND SLOTTED DRAIN INLET INTERCEPTION EFFICIENCY D:17
FIGURE D.17 DEPRESSED CURB OPENING INLET D:18
FIGURE D.18 HEADLOSS COEFFICIENTS FOR CASES 2 AND 7 D:19
FIGURE D.19 HEADLOSS COEFFICIENT FOR CASE 5 D:20
FIGURE D.20 MAXIMUM DISTANCES FOR MANHOLE LOCATIONS D:21
FIGURE D.21 UNIFORM FLOW FOR TRAPEZOIDAL CHANNELS D:22
FIGURE D.22 SLOPING AND VERTICAL CHANNEL DROPS D:23
FIGURE D.23 BAFFLED APRON AND ITS DESIGN CURVE D:24
FIGURE D.24 CONCEPTUAL DESIGN OF ALTERNATIVE CHANNEL D:25
FIGURE D.25 HEADWALL ENTRANCE TYPES D:26
FIGURE D.26 CONCEPTUAL DESIGN OF DEBRIS FINS D:27
FIGURE D.27 INLET AND OUTLET CONDITIONS FOR CULVERTS D:28
FIGURE D.28 HYDRAULICS OF A CULVERT UNDER OUTLET CONTROL CONDITION D:29
FIGURE D.29 TYPES OF FLOW FOR BRIDGE DESIGN D:30



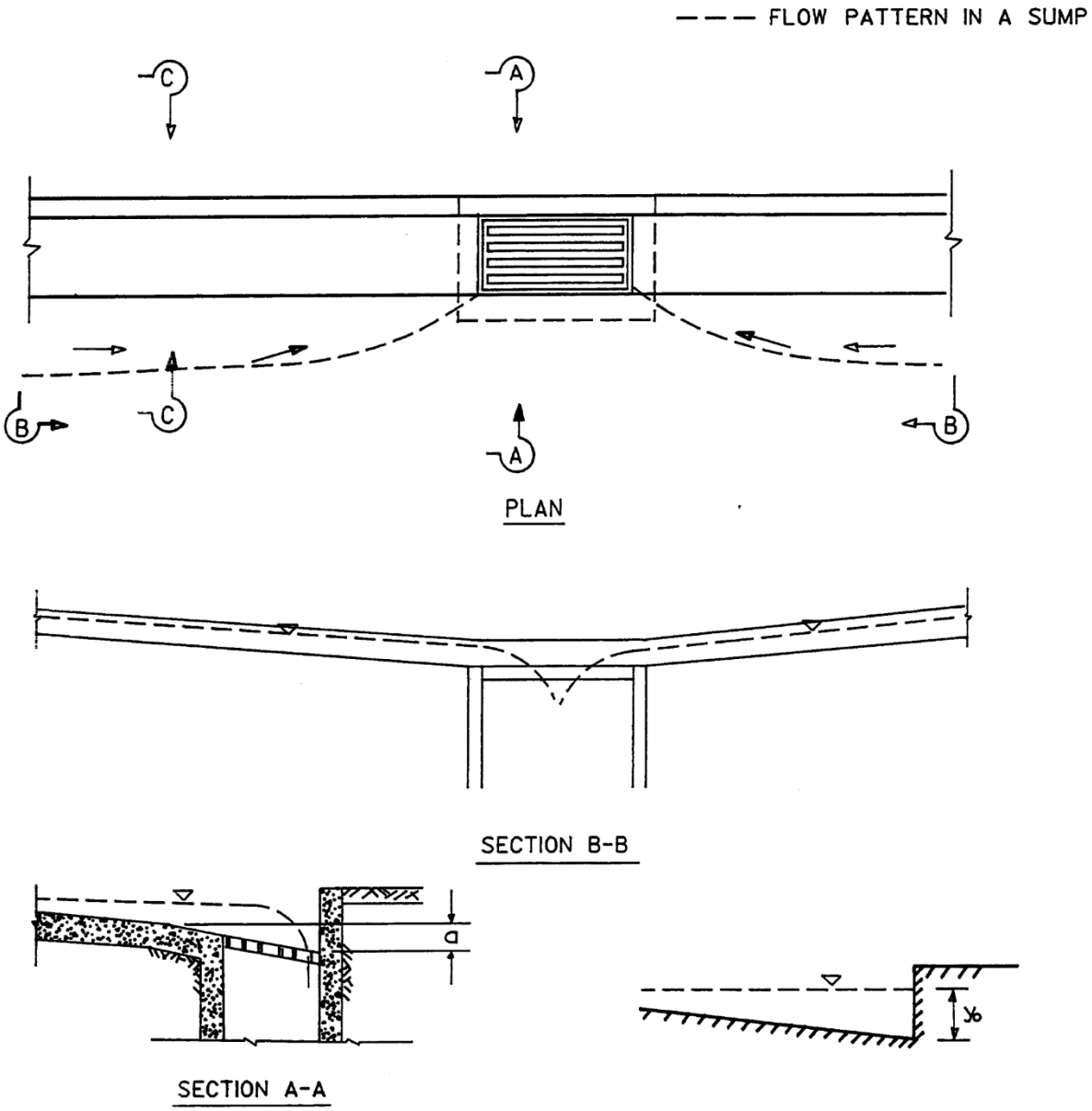
Source: US Army Corps of Engineers

FIGURE D.1 NOMOGRAPH FOR FLOW IN GUTTERS



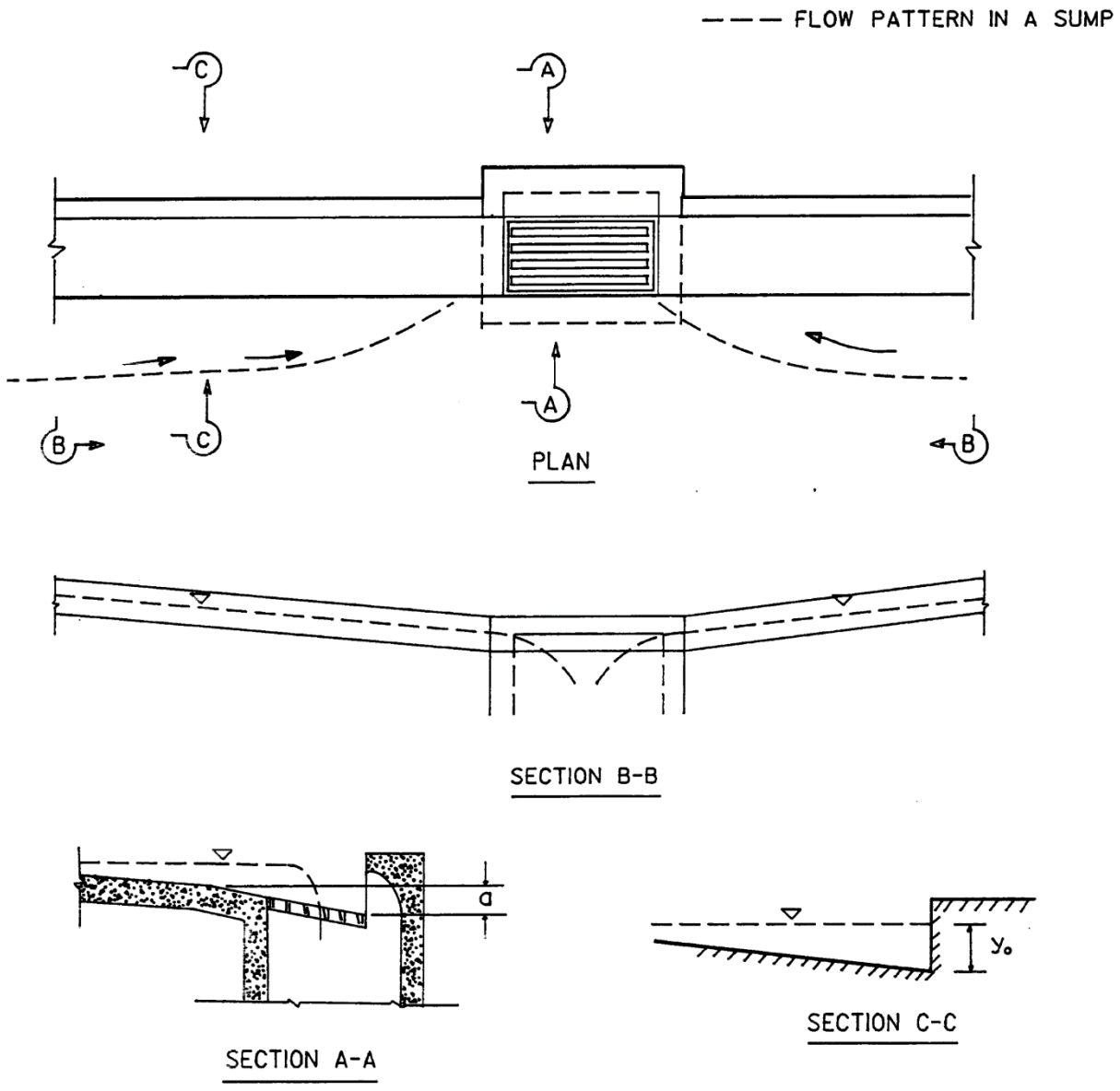
Source: City of Austin. Drainage Criteria Manual. Department of Public Works. Austin, Texas. January 1977.

FIGURE D.2 CURB OPENING INLET IN A SUMP (TYPE S-1)



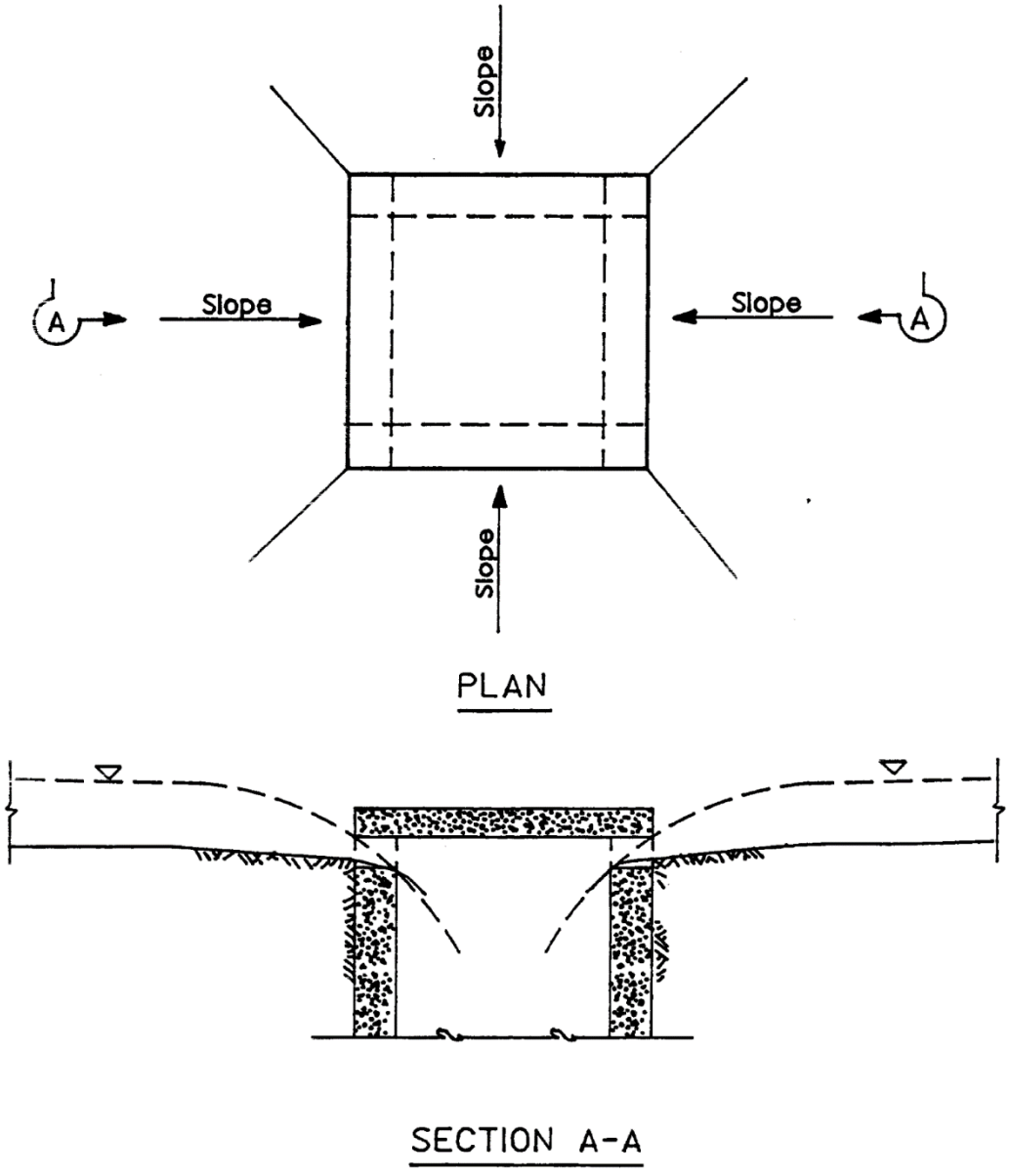
Source: City of Austin. Drainage Criteria Manual. Department of Public Works. Austin, Texas. January 1977.

FIGURE D.3 GRATE INLET IN A SUMP (TYPE S-2)



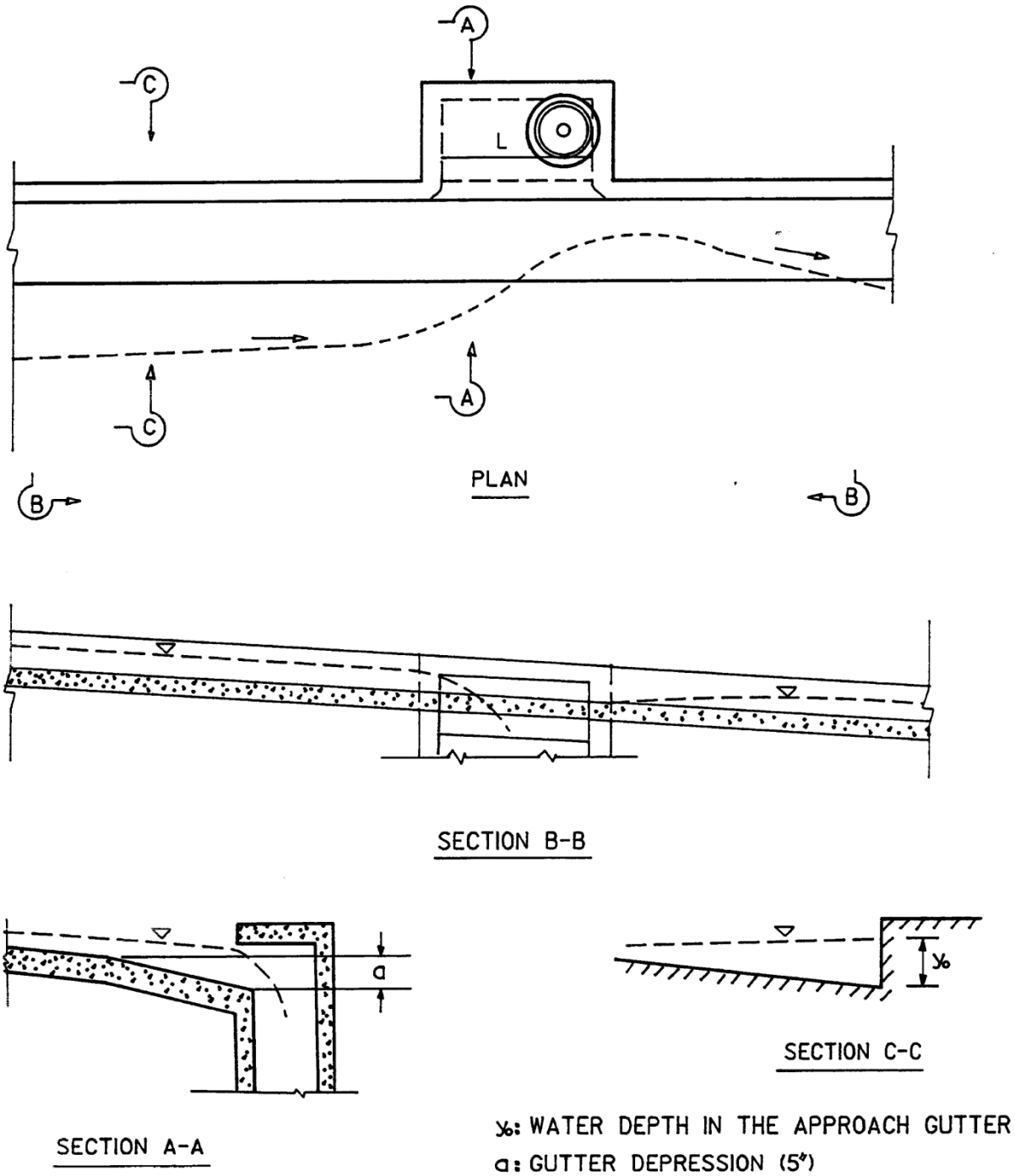
Source: City of Austin. Drainage Criteria Manual. Department of Public Works. Austin, Texas. January 1977.

FIGURE D.4 COMBINATION INLET IN A SUMP (TYPE S-3)



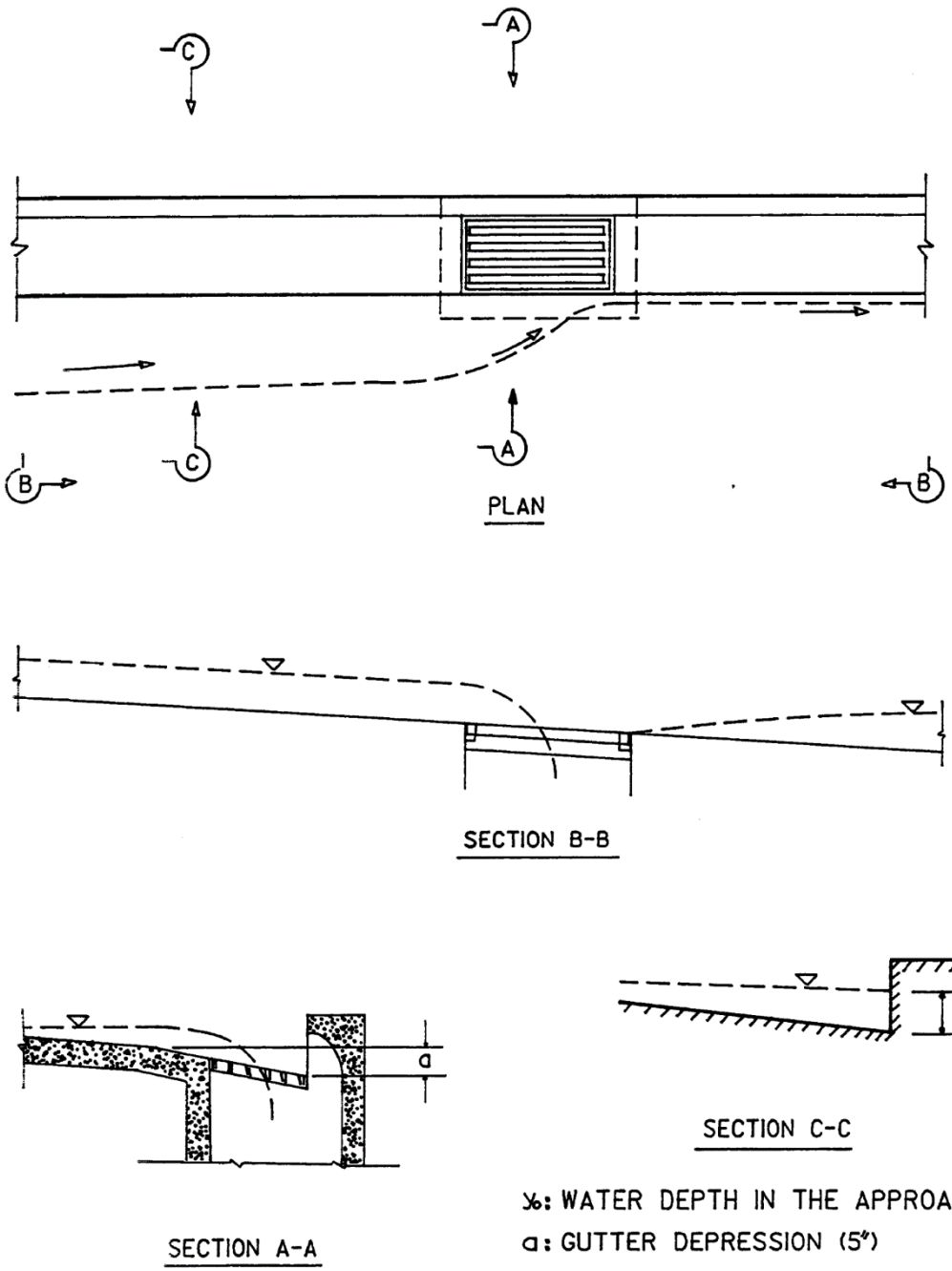
Source: City of Austin. Drainage Criteria Manual. Department of Public Works. Austin, Texas. January 1977.

FIGURE D.5 AREA INLET WITHOUT GRATE (TYPE S-4)



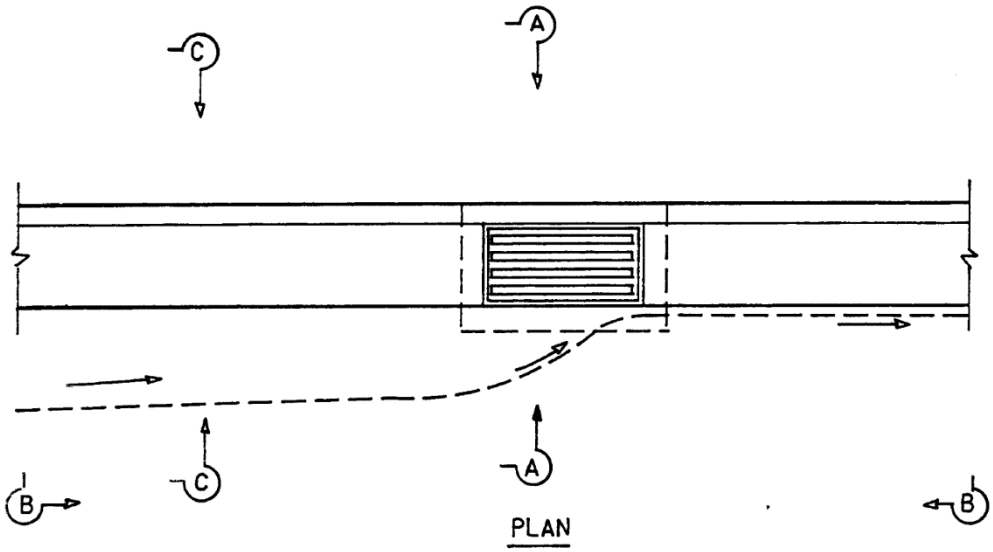
Source: City of Austin. Drainage Criteria Manual. Department of Public Works. Austin, Texas. January 1977.

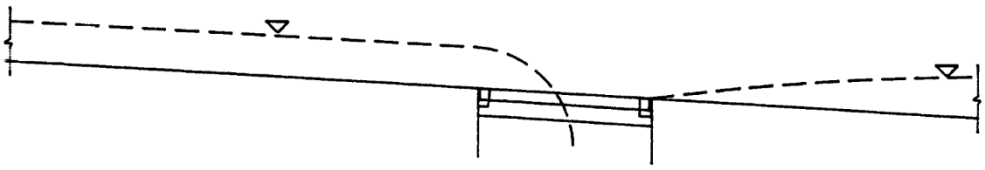
FIGURE D.6 CURB OPENING, INLET ON GRADE (TYPE G-1)

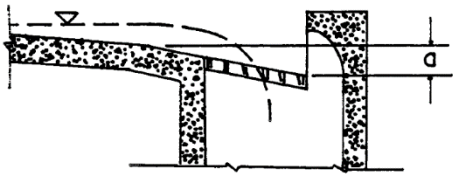


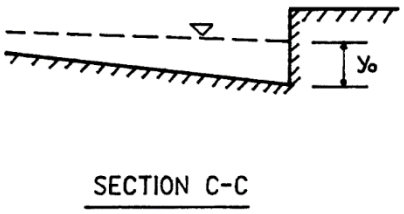
Source: City of Austin. Drainage Criteria Manual. Department of Public Works. Austin, Texas. January 1977.

FIGURE D.7 GRATE, INLET ON GRADE (TYPE G-2)









y_o: WATER DEPTH IN THE APPROACH GUTTER

a: GUTTER DEPRESSION (5")

Source: City of Austin. Drainage Criteria Manual. Department of Public Works. Austin, Texas. January 1977.

FIGURE D.8 COMBINATION INLET ON GRADE (TYPE G-3)

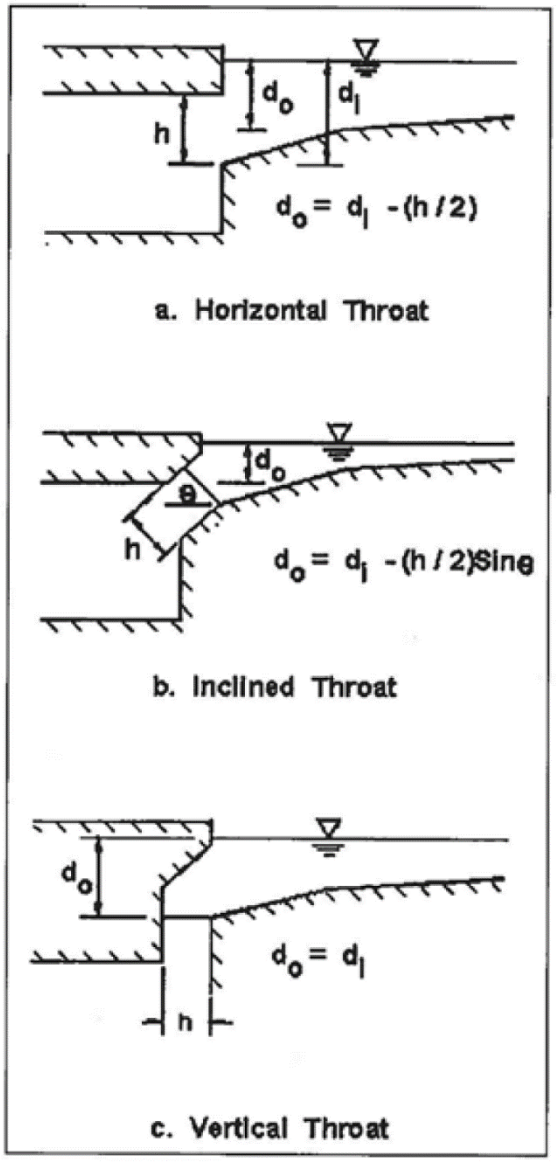


FIGURE D.9 CURB OPENING INLETS

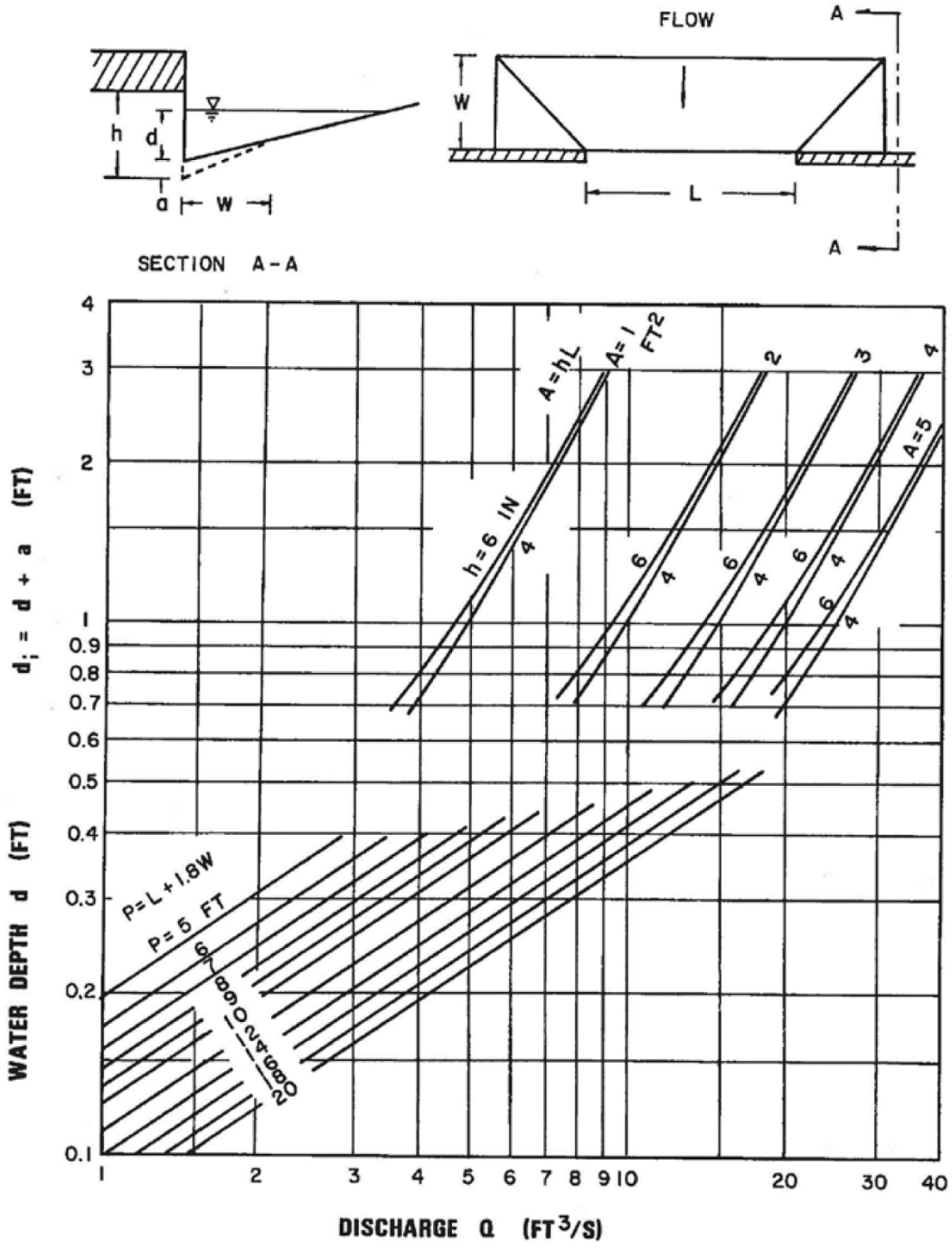


FIGURE D.10 DEPRESSED CURB OPENING INLET CAPACITY IN SUM P LOCATIONS – ENGLISH UNITS

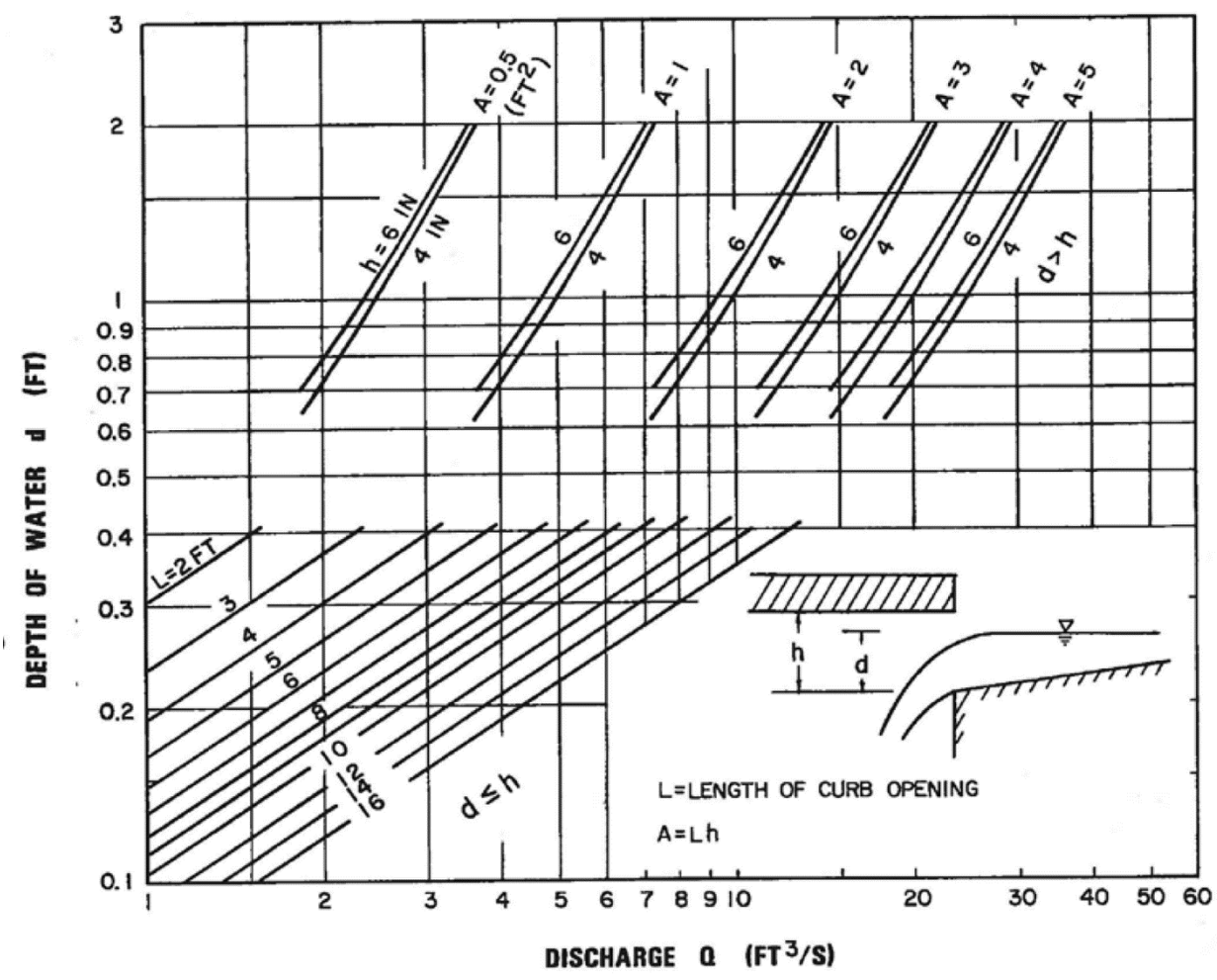


FIGURE D.11 UNDEPRESSED CURB OPENING INLET CAPACITY IN SUMP LOCATIONS – ENGLISH UNITS

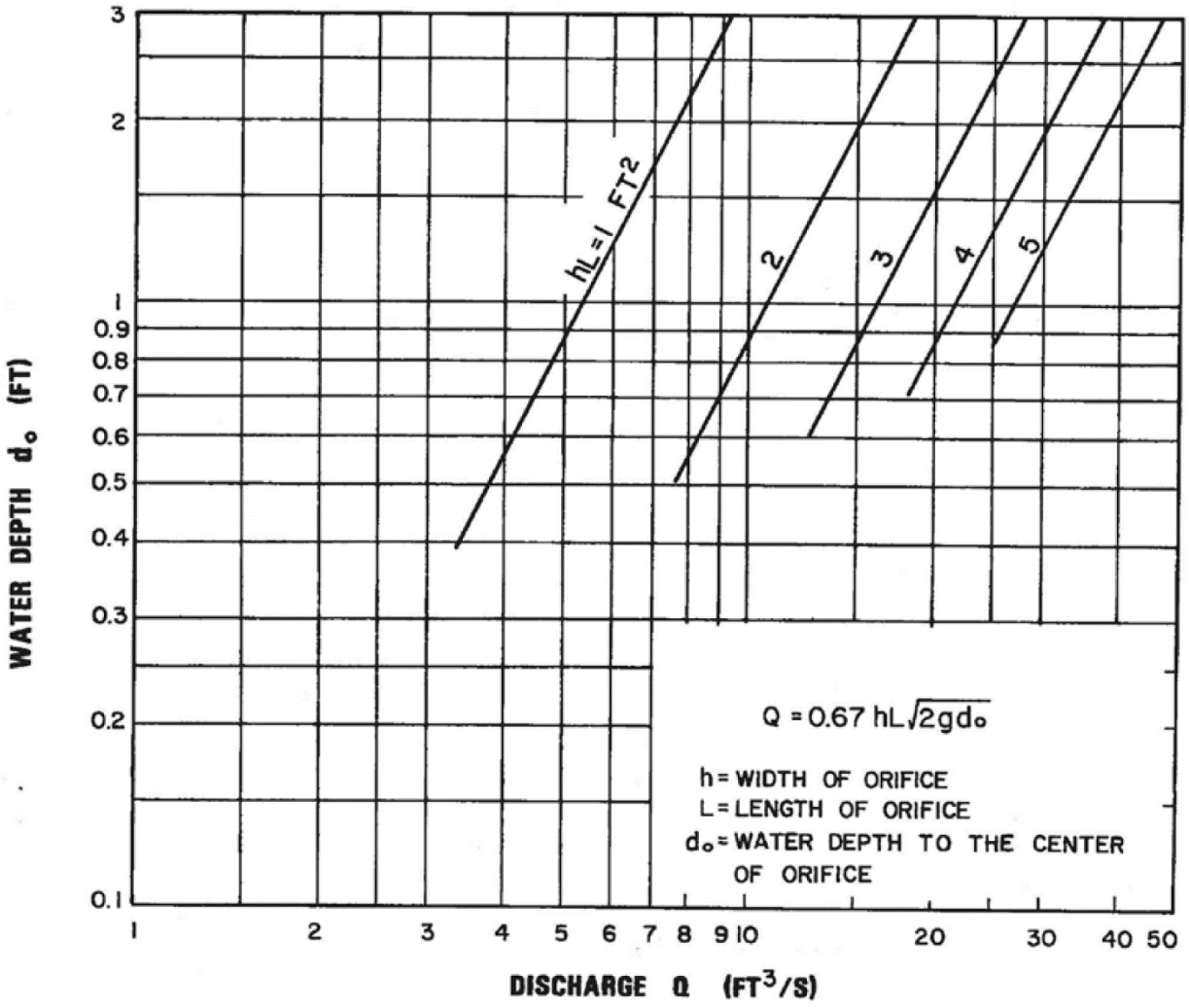


FIGURE D.12 CURB OPENING INLET ORIFICE CAPACITY FOR INCLINED AND VERTICAL ORIFICE THROATS – ENGLISH UNITS

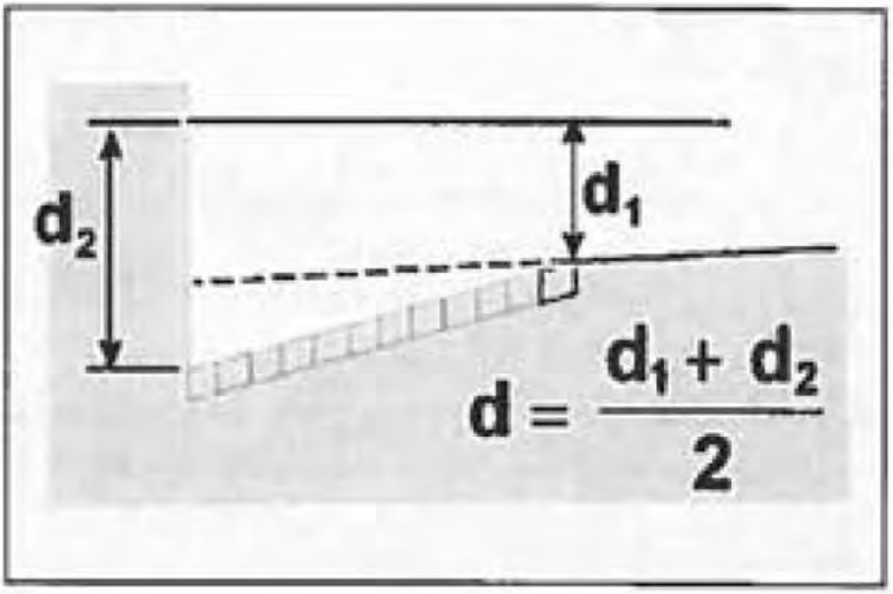


FIGURE D.13 DEFINITION OF DEPTH

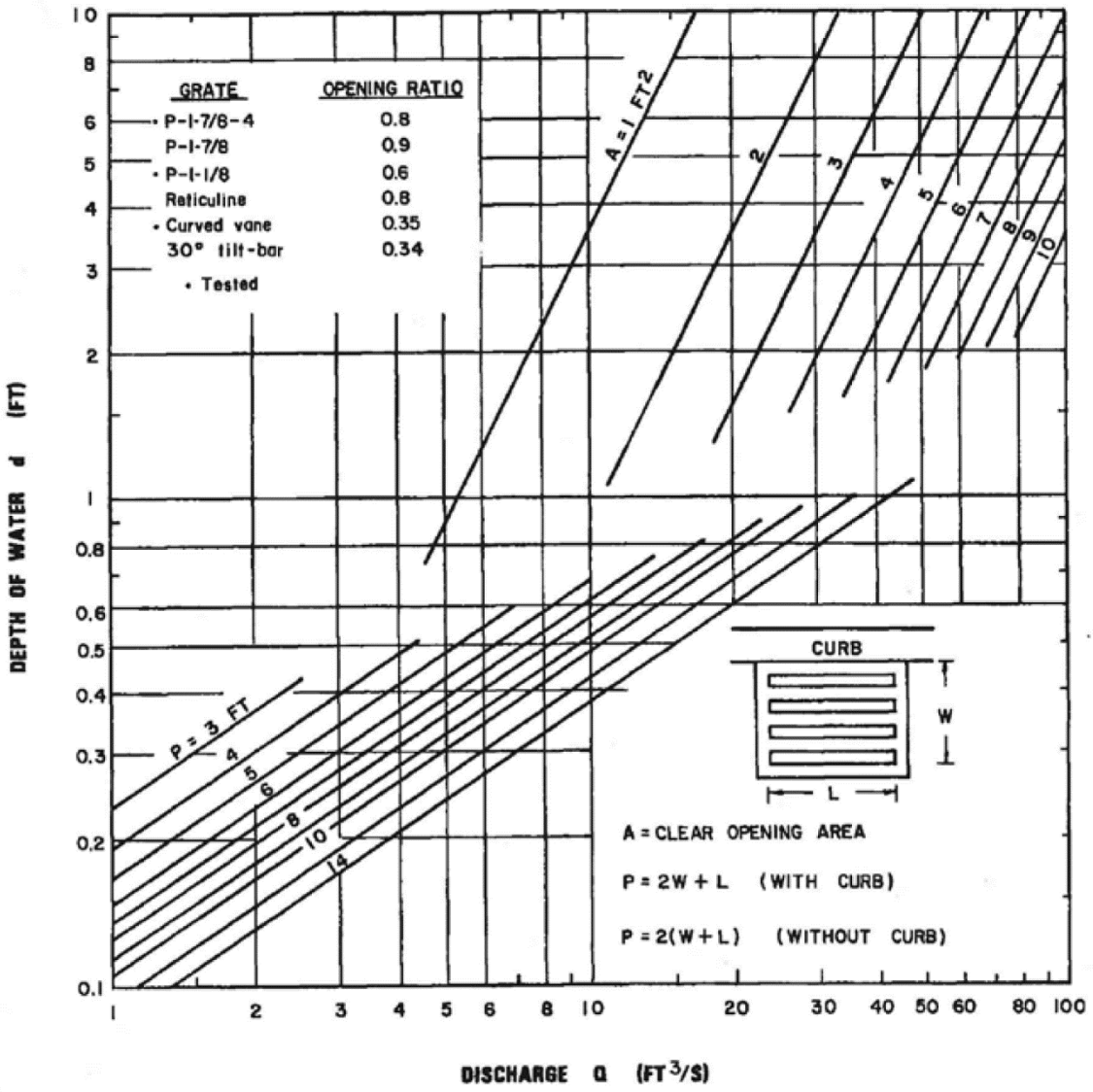


FIGURE D.14 GRATE INLET CAPACITY IN SUMP CONDITIONS – ENGLISH UNITS

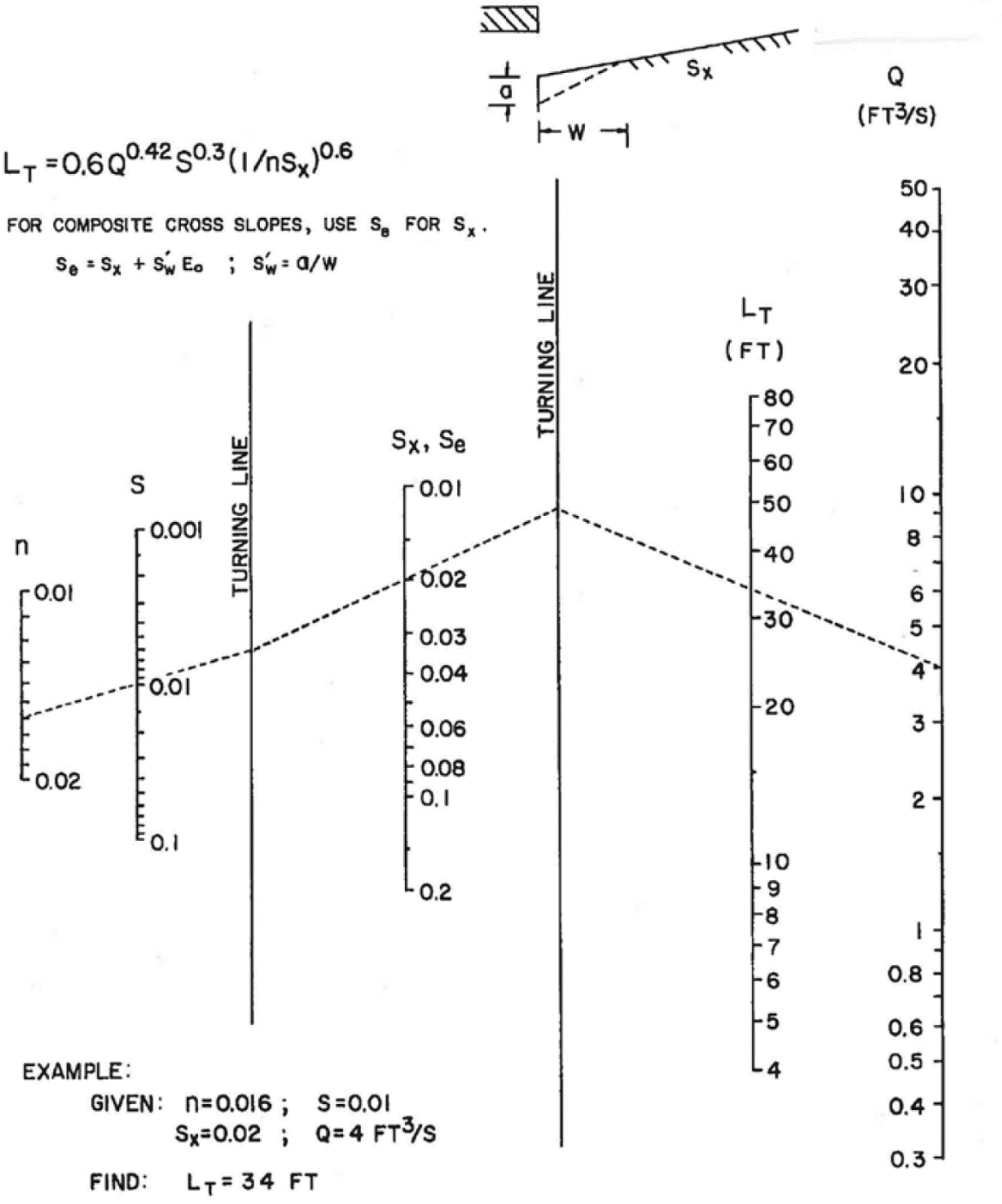


FIGURE D.15 CURB OPENING AND SLOTTED DRAIN INLET LENGTH FOR TOTAL INTERCEPTION – ENGLISH UNITS

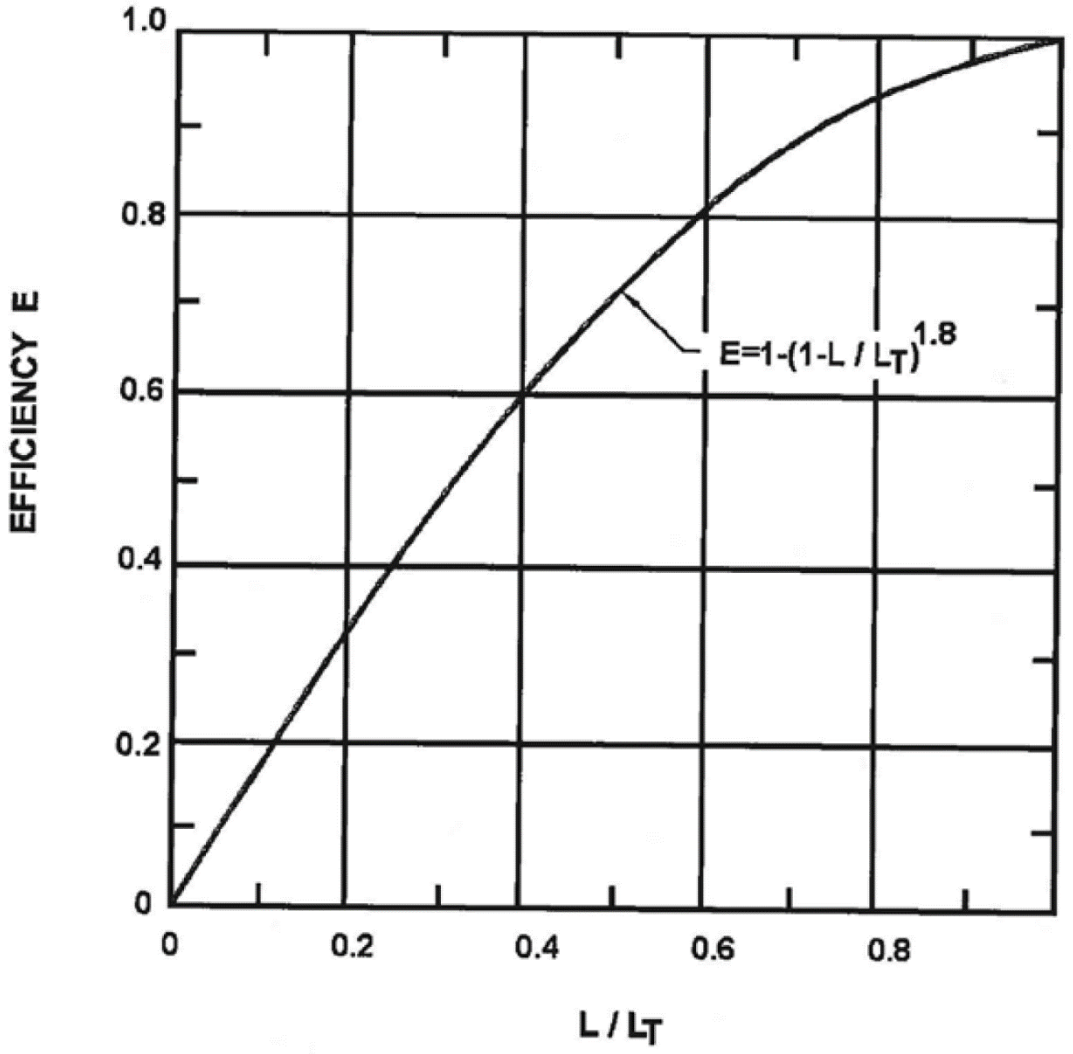


FIGURE D.16 CURB OPENING AND SLOTTED DRAIN INLET INTERCEPTION EFFICIENCY

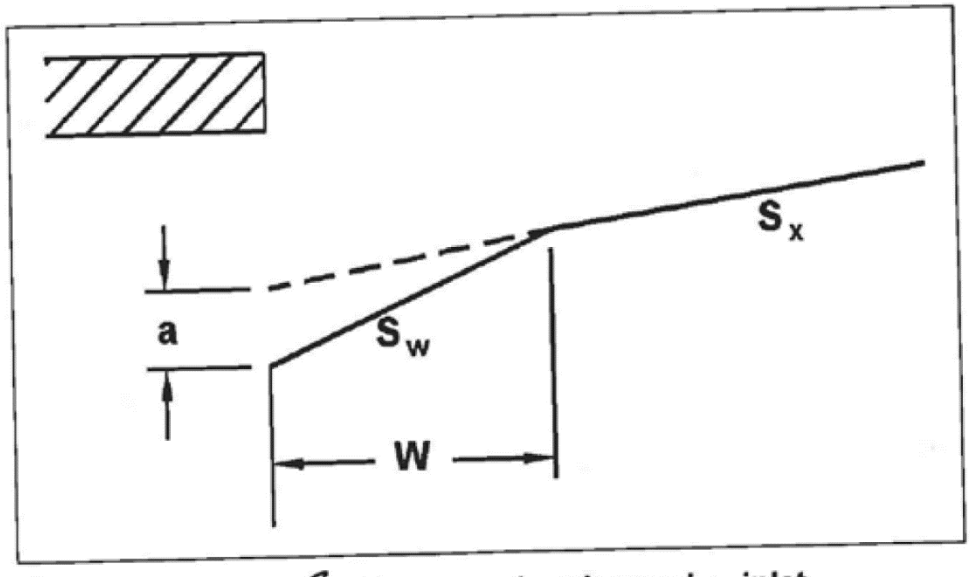
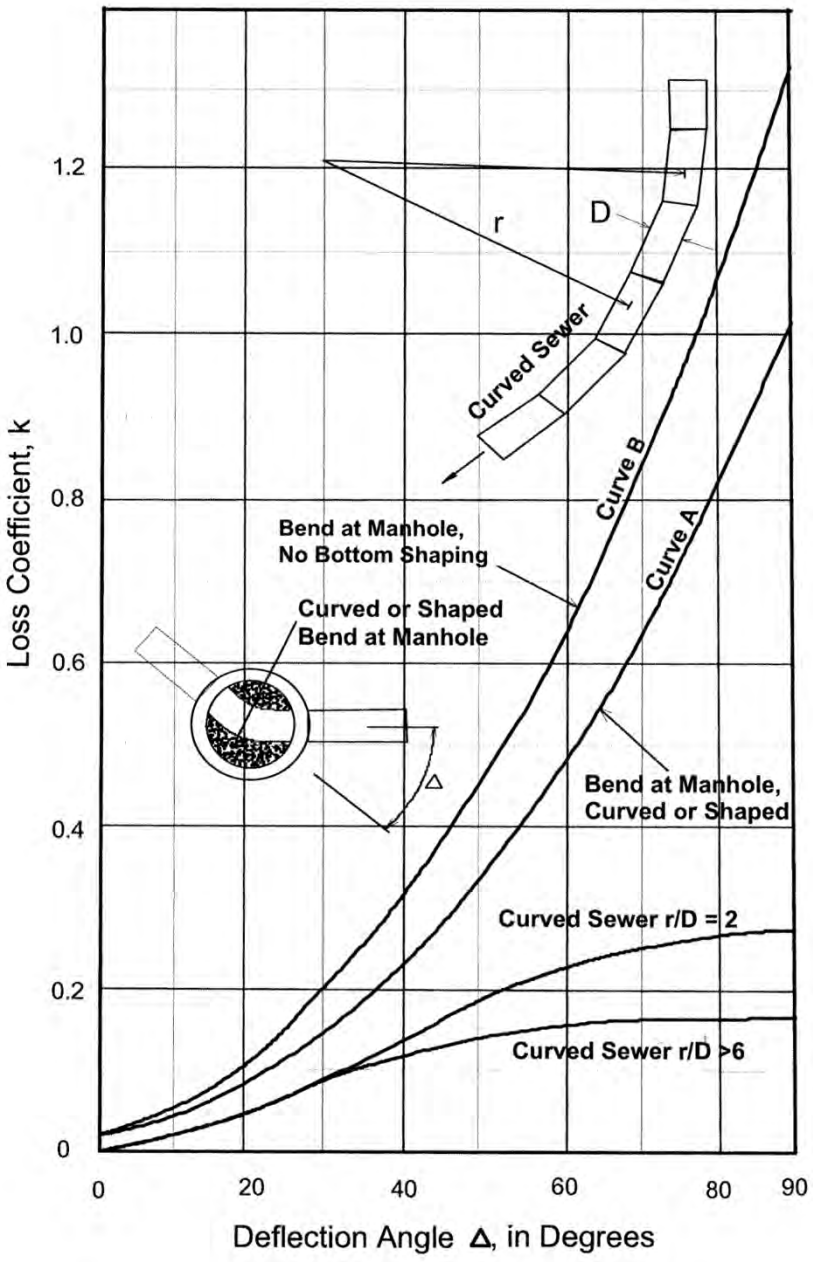
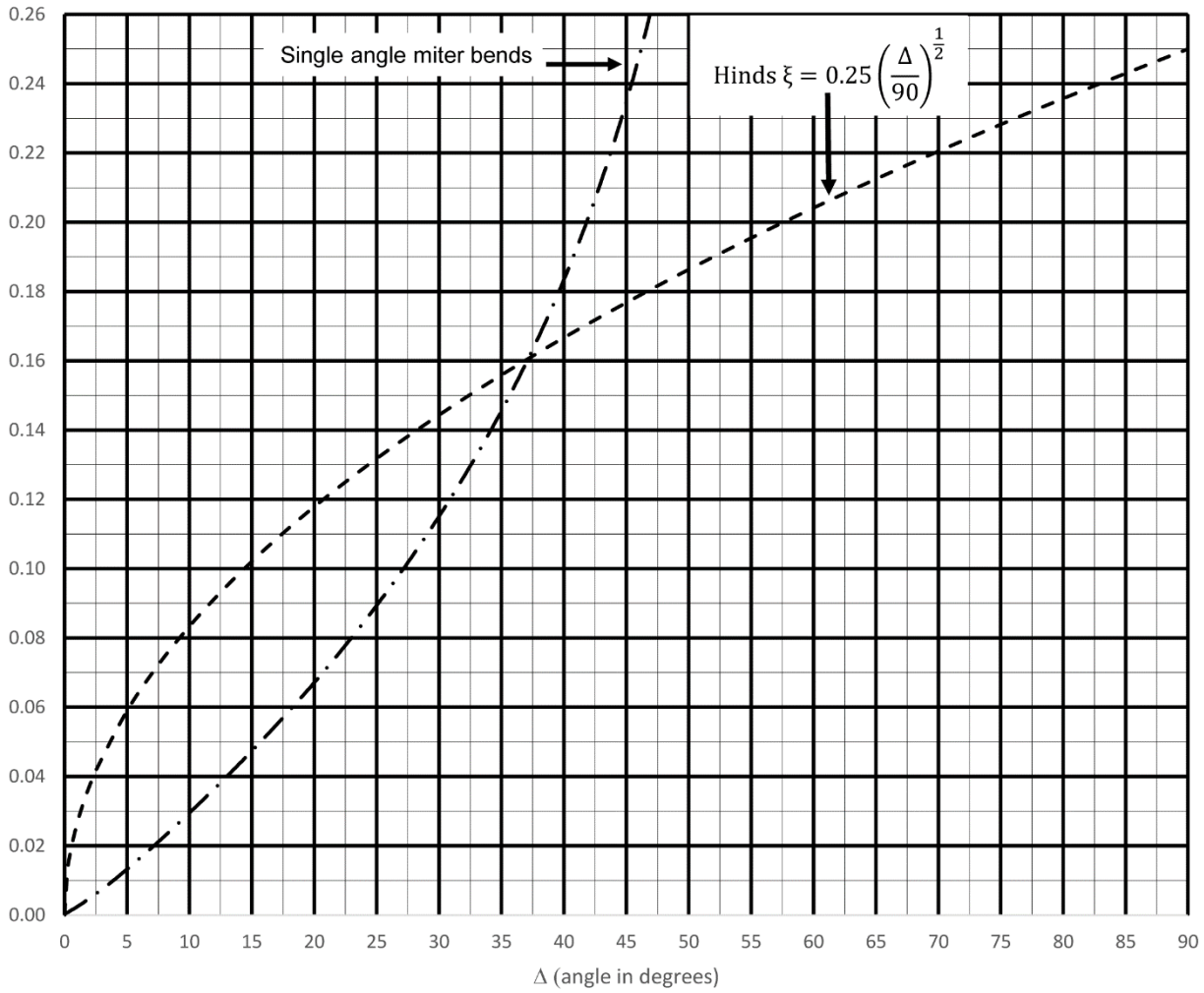


FIGURE D.17 DEPRESSED CURB OPENING INLET



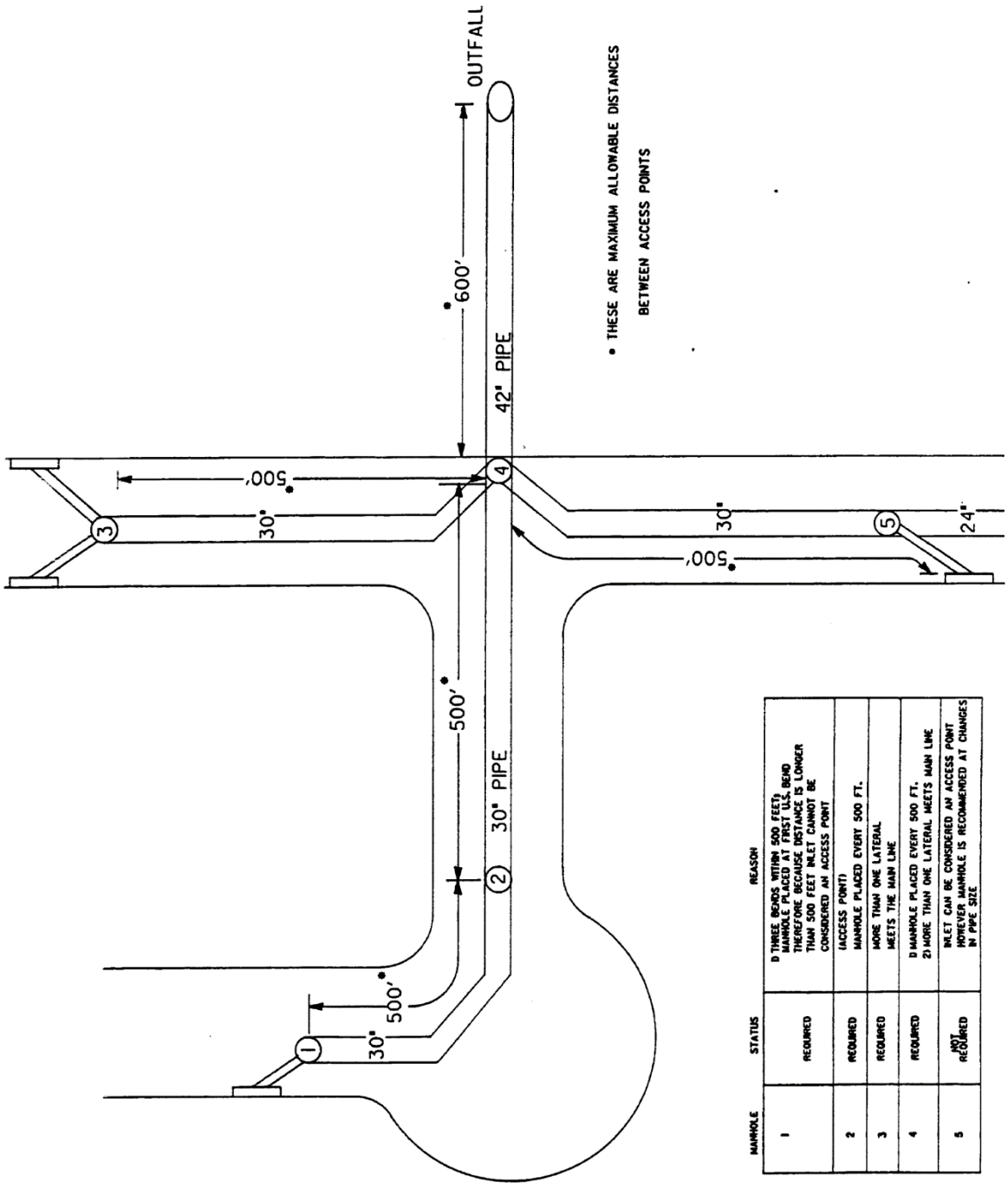
Headloss Coefficient for Cases 2 and 7
Source: Urban Storm Drainage Criteria Manual
Volume 1
Urban Drainage and Flood Control District
Denver, Colorado
June 2001, Revised April 2008

FIGURE D.18 HEADLOSS COEFFICIENTS FOR CASES 2 AND 7



Headloss Coefficient for Case 5
Source: Design of Small Canal Structures
U.S. Bureau of Reclamation
Denver, Colorado
1978

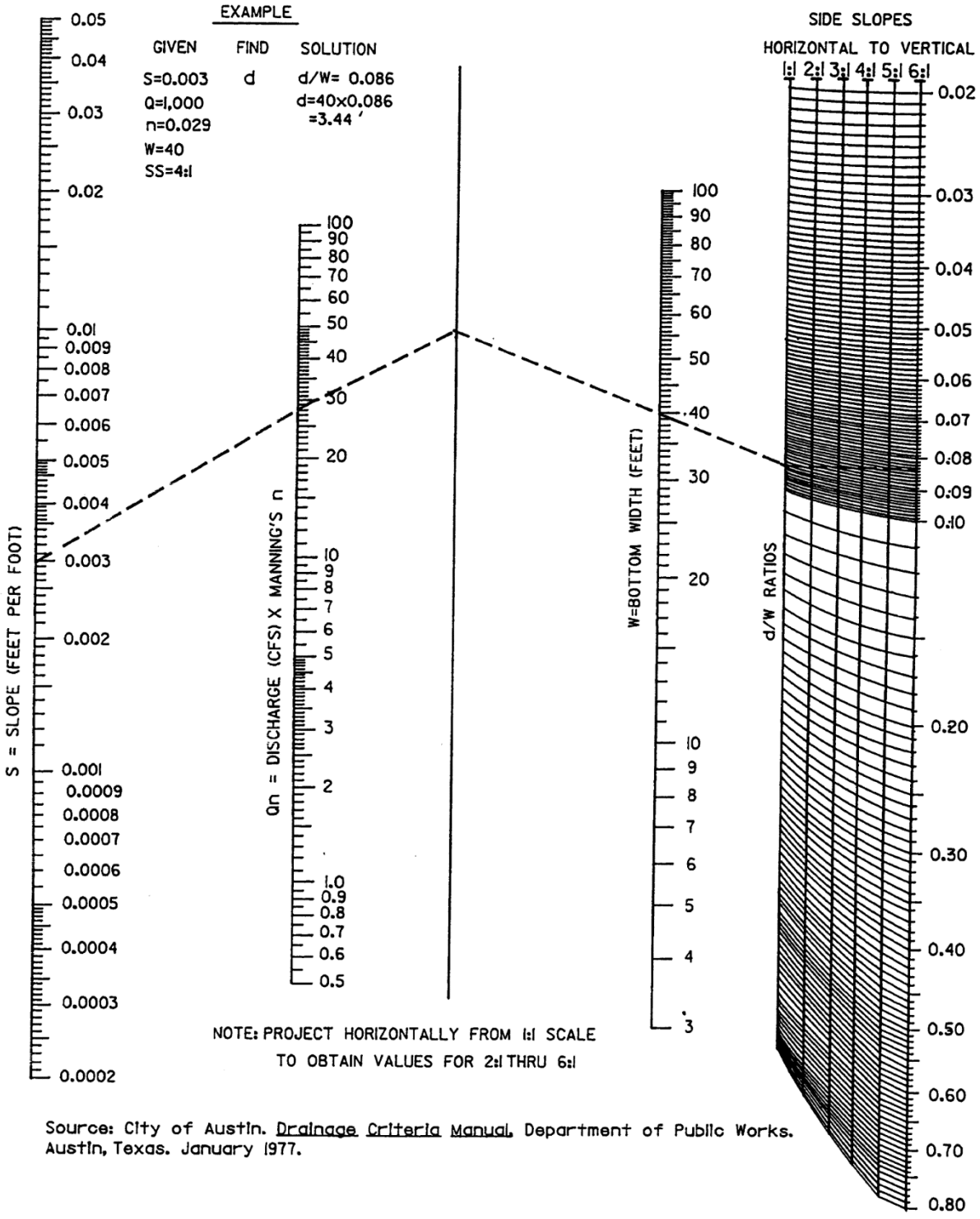
FIGURE D.19 HEADLOSS COEFFICIENT FOR CASE 5



MANHOLE	STATUS	REASON
1	REQUIRED	1) THREE BENDS WITHIN 500 FEET) MANHOLE PLACED AT FIRST U.S. BEND THEREFORE BECAUSE DISTANCE IS LONGER THAN 500 FEET INLET CANNOT BE CONSIDERED AN ACCESS POINT
2	REQUIRED	(ACCESS POINT) MANHOLE PLACED EVERY 500 FT.
3	REQUIRED	MORE THAN ONE LATERAL MEETS THE MAIN LINE
4	REQUIRED	1) MANHOLE PLACED EVERY 500 FT. 2) MORE THAN ONE LATERAL MEETS MAIN LINE
5	NOT REQUIRED	INLET CAN BE CONSIDERED AN ACCESS POINT HOWEVER MANHOLE IS RECOMMENDED AT CHANGES IN PIPE SIZE

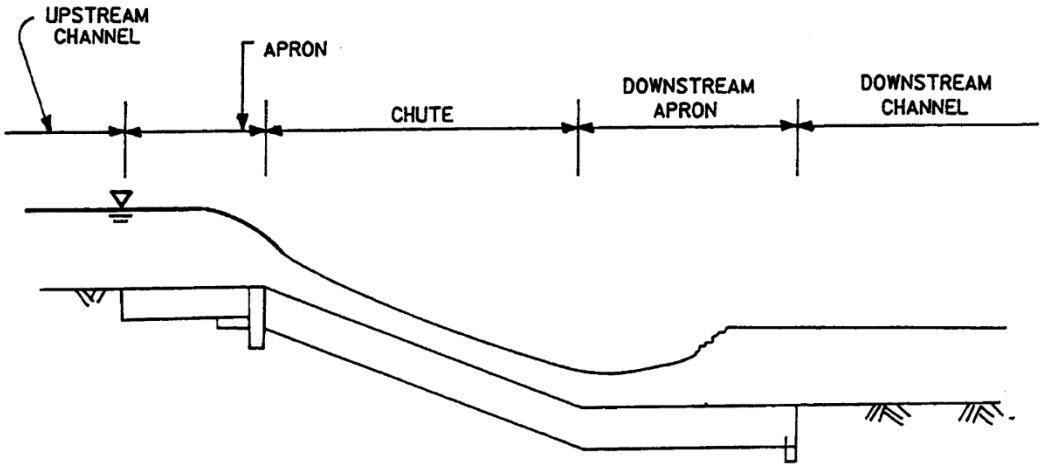
Source: City of Austin, Watershed Management Division

FIGURE D.20 MAXIMUM DISTANCES FOR MANHOLE LOCATIONS

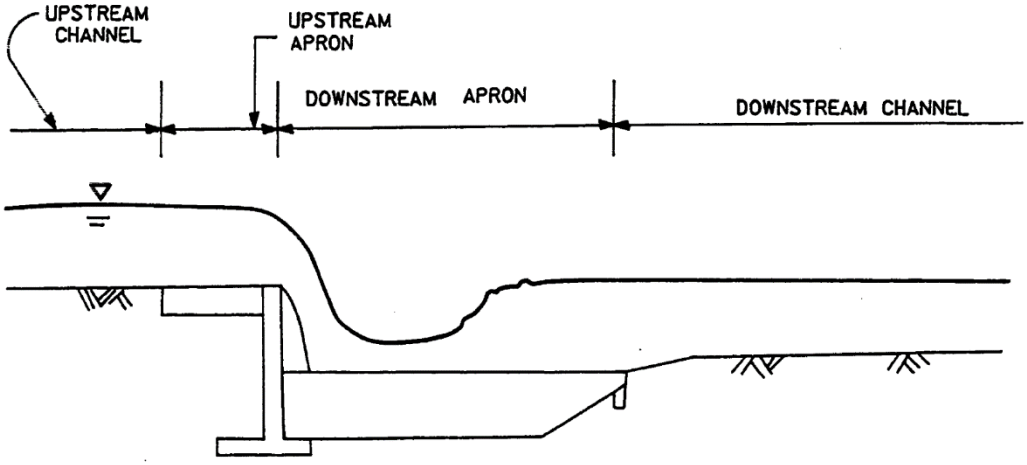


Source: City of Austin. Drainage Criteria Manual. Department of Public Works. Austin, Texas. January 1977.

FIGURE D.21 UNIFORM FLOW FOR TRAPEZOIDAL CHANNELS



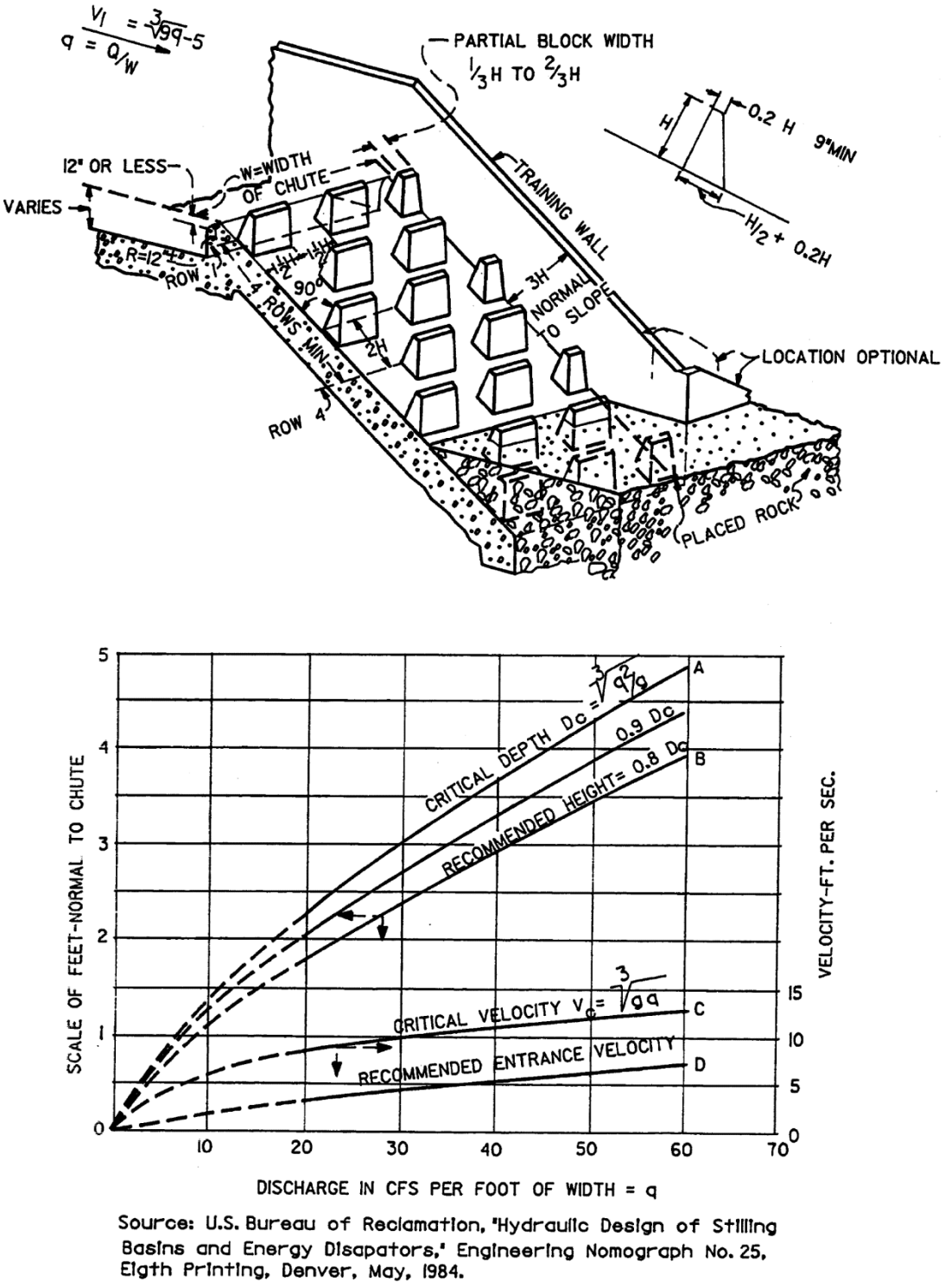
A. SLOPING CHANNEL DROP



B. VERTICAL CHANNEL DROP

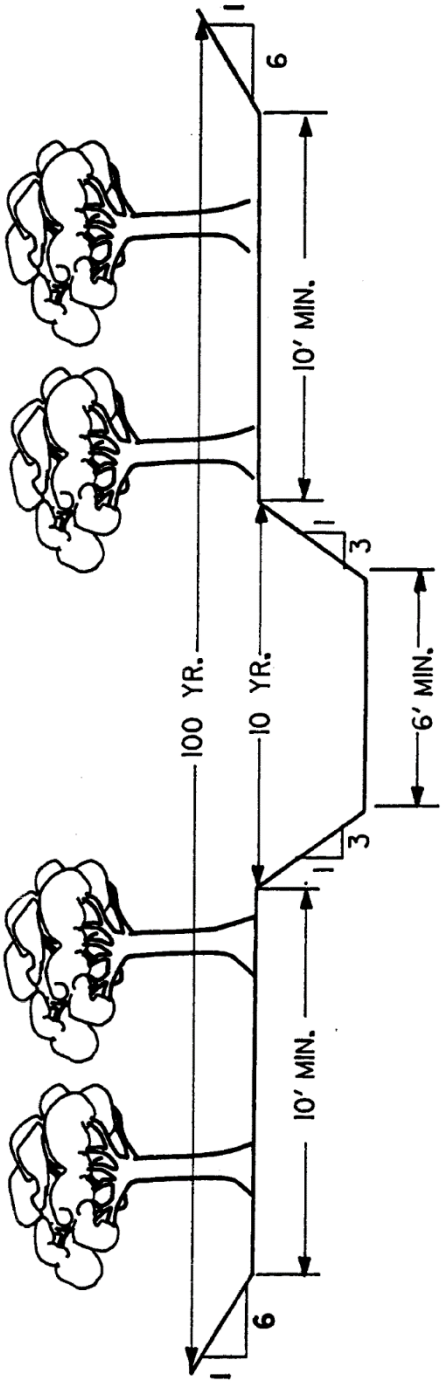
Source: U.S. Bureau of Reclamation, 'Hydraulic Design of Stilling Basins and Energy Disapators,' Engineering Nomograph No. 25, Eigth Printing, Denver, May, 1984.

FIGURE D.22 SLOPING AND VERTICAL CHANNEL DROPS



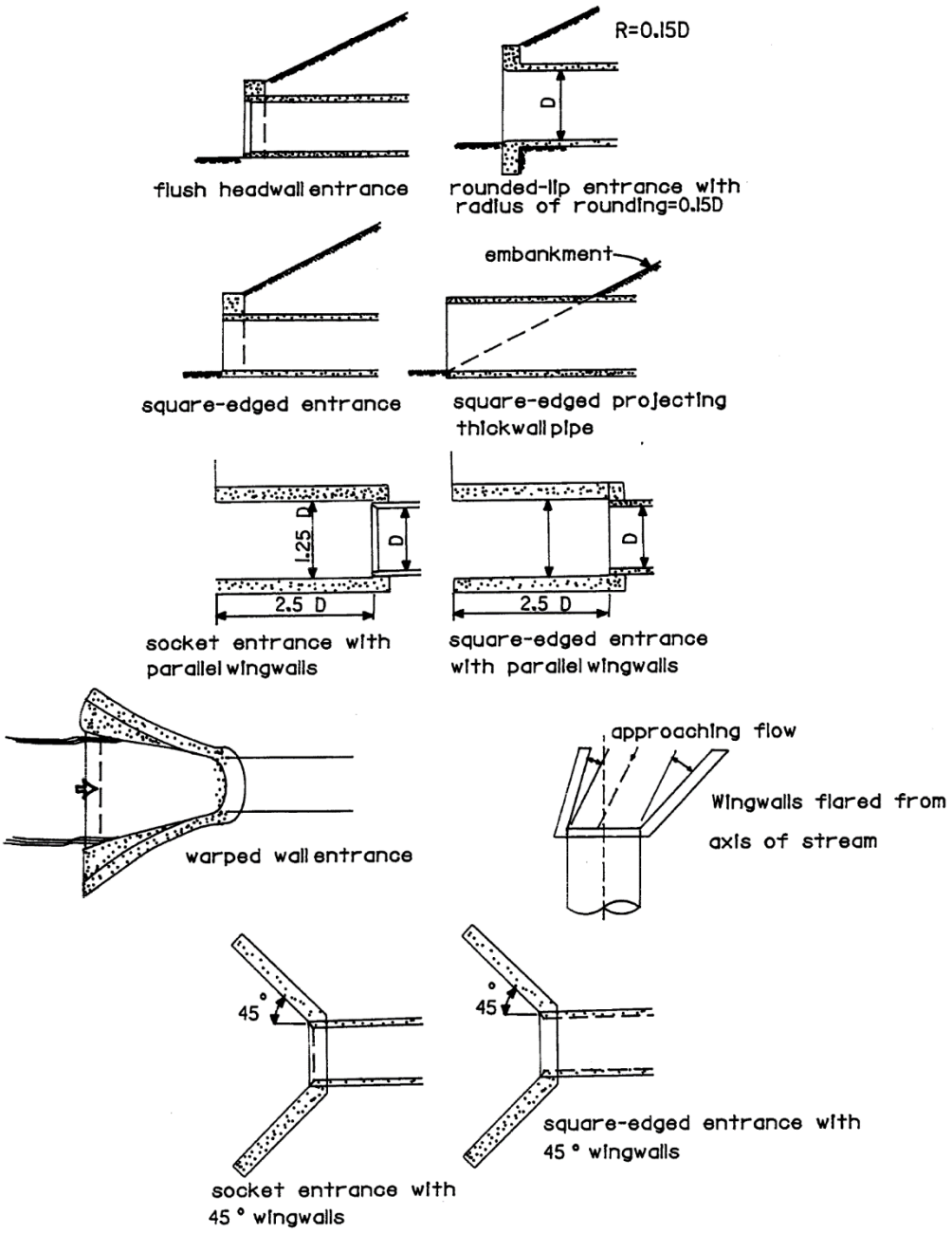
Source: U.S. Bureau of Reclamation, "Hydraulic Design of Stilling Basins and Energy Disapators," Engineering Nomograph No. 25, Eigth Printing, Denver, May, 1984.

FIGURE D.23 BAFFLED APRON AND ITS DESIGN CURVE



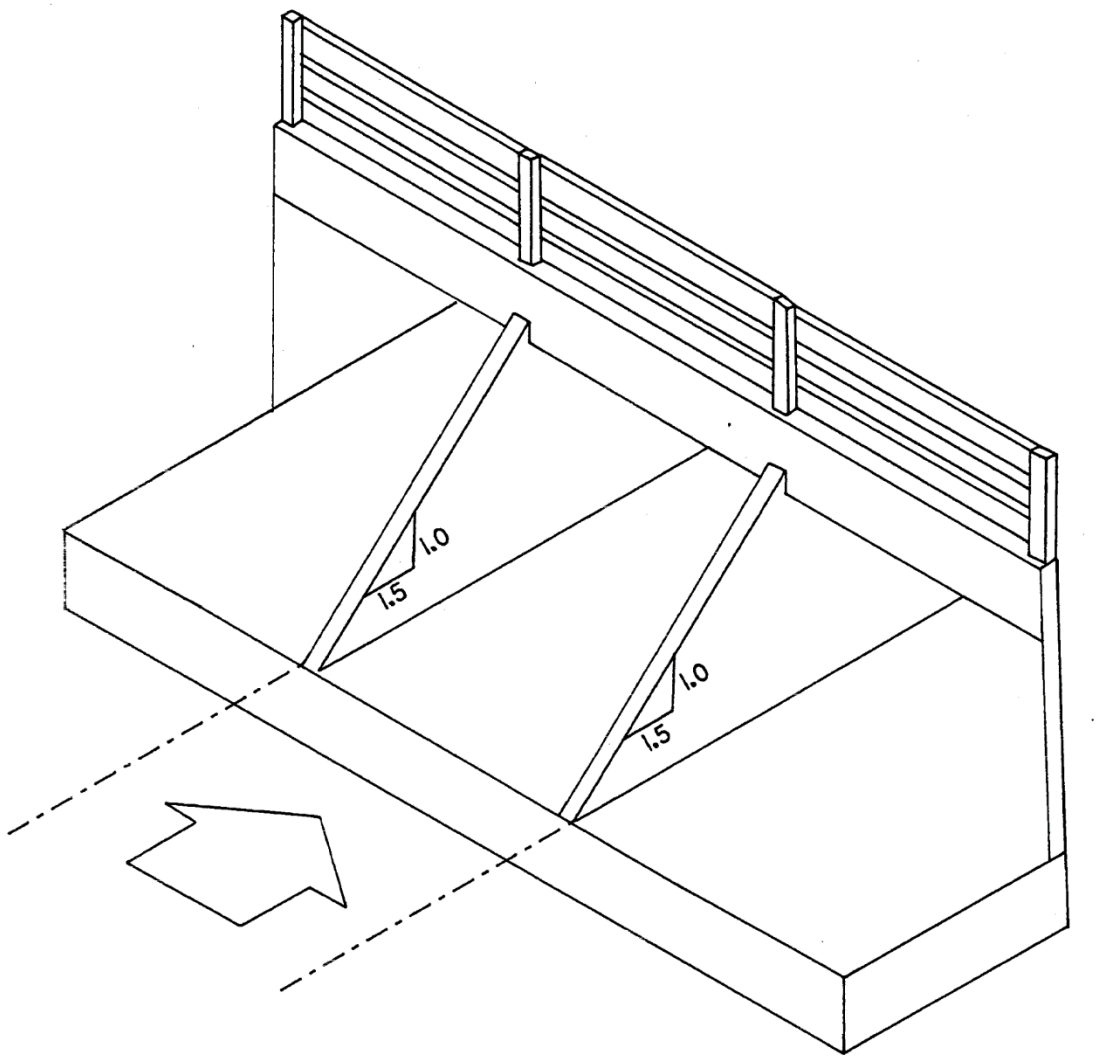
Source: City of Austin, Department of Environmental Protection

FIGURE D.24 CONCEPTUAL DESIGN OF ALTERNATIVE CHANNEL



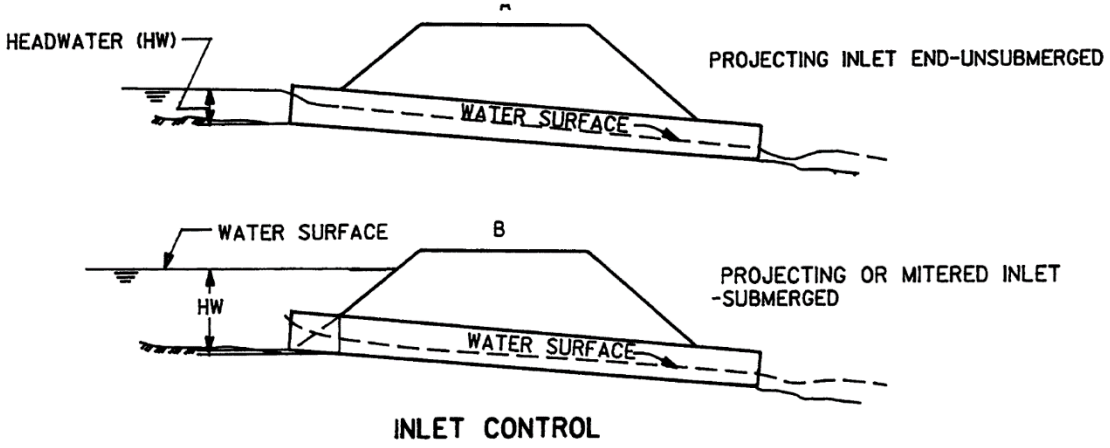
Source: City of Austin, Watershed Management Division

FIGURE D.25 HEADWALL ENTRANCE TYPES

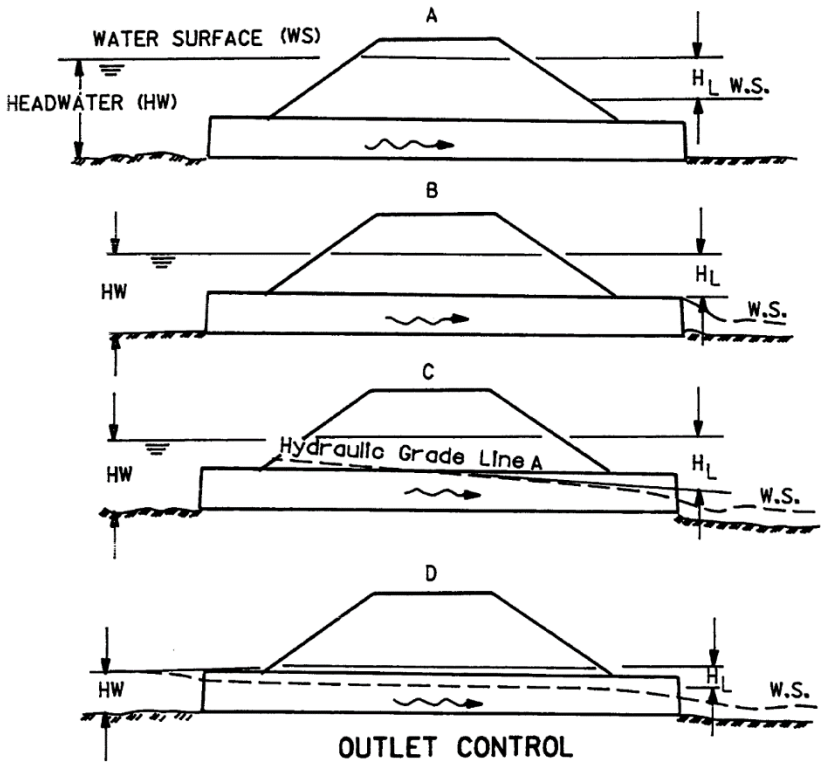


Source: City of Austin, Watershed Management Division

FIGURE D.26 CONCEPTUAL DESIGN OF DEBRIS FINS

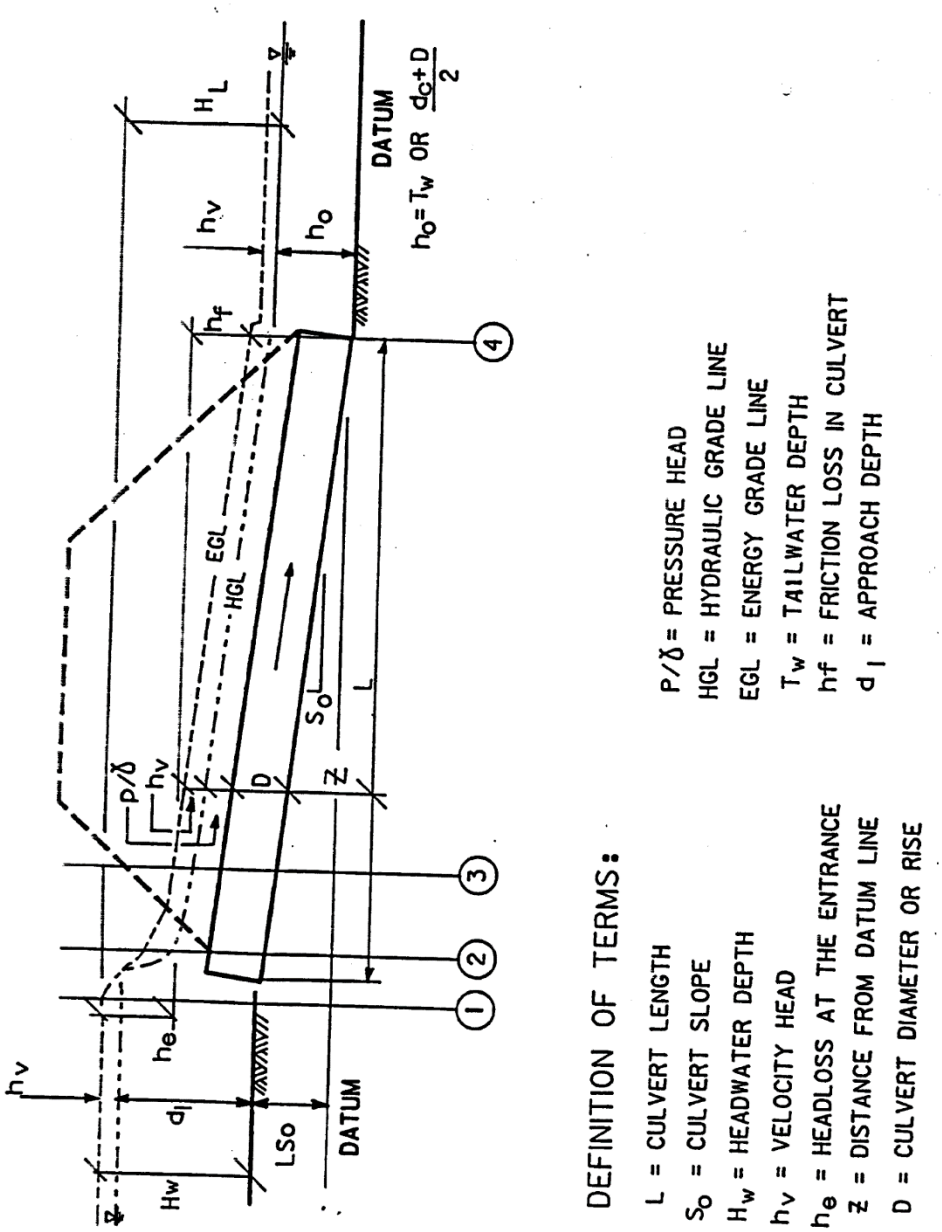


INLET CONTROL IS ONE OF THE TWO MAJOR TYPES OF CULVERT FLOW. CONDITION A WITH AN UNSUBMERGED CULVERT INLET IS PREFERRED TO THE SUBMERGED END. SLOPE, ROUGHNESS AND LENGTH OF CULVERT BARREL ARE NOT A CONSIDERATION.



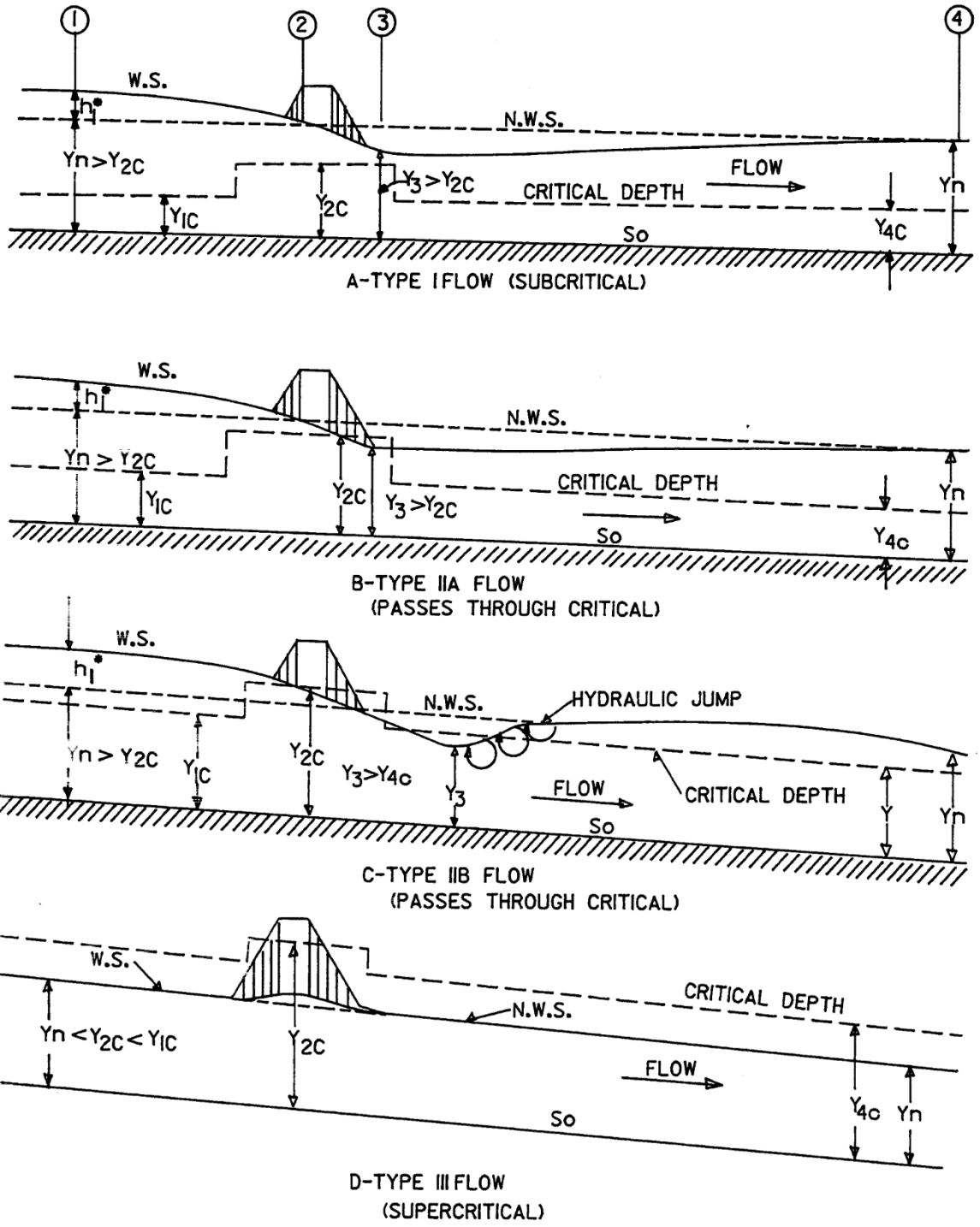
OUTLET CONTROL INVOLVES THESE FACTORS: CROSS-SECTIONAL AREA OF BARREL, INLET "GEOMETRY", PONDING, SLOPE, ROUGHNESS, TAILWATER, AND LENGTH OF CULVERT BARREL.
Source: Boulder County, Storm Drainage Criteria Manual

FIGURE D.27 INLET AND OUTLET CONDITIONS FOR CULVERTS



Source: Boulder County, Storm Drainage Criteria Manual

FIGURE D.28 HYDRAULICS OF A CULVERT UNDER OUTLET CONTROL CONDITION



Source: Federal Highway Administration, "Hydraulics of Bridge Waterways," Hydraulic Design Series No. 1, Second Edition, September, 1973.

FIGURE D.29 TYPES OF FLOW FOR BRIDGE DESIGN