Adopted June 21, 2021

City of Moore, Oklahoma
Stormwater Management Criteria

MESHEK & ASSOCIATES, LLC

CITY of MOORE • OKLAHOMA • INCORPORATED 1893

Contents

1. STORMWATER POLICY AND STANDARDS

1.1 Goals

1.1.1. To protect the general health, safety, and welfare of the residents of the City of Moore;

1.1.2. To minimize need for rescue and relief efforts associated with flooding which is generally undertaken at the expense of the public;

1.1.3. To encourage best practices in stormwater management, including Low Impact Development (LID) and green stormwater infrastructure (GSI) to reduce pollutant loading within throughout the drainage areas which include site specific measures such as:

1.1.3.1. On-site and regional stormwater detention, including wet ponds

1.1.3.2. Reduction in impervious cover

1.1.3.3. Floodplain preservation

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1.1.3.8. Riparian buffering

1.1.3.9. Preservation of riparian habitat.

1.1.4. To ensure that all development within the City of Moore provides for the proper handling of storm water runoff from a site such that for all studied frequency floods there are no increases in peak downstream discharges or velocities and no increases in water surface elevations which result in additional damages to downstream structures;

1.1.5. To increase the recreational opportunities and to encourage the retention of open space; and

1.1.6. To adopt strong storm water management practices

1.2 Principles

1.2.1. City of Moore Comprehensive Plan

1.2.1.1. Stormwater runoff requires development planning to include the allocation of space for drainage facilities that are compatible with the City of Moore Comprehensive Plan.

1.2.1.2. The drainage system shall be designed as an integral part of the landscape.

1.2.2. Stormwater Drainage System Planning Guidelines

1.2.2.1. Closed conduit storm sewers shall be used in all developments up to a drainage basin size of 40 acres generally. For drainage basins larger than 40 acres, open channels shall be required unless otherwise recommended within a Master Drainage Plan.

1.2.2.2. Drainage channel improvements shall be developed and designed to preserve and protect trees and other worthy botanical and geological features to the maximum extent practicable as recommended in the Comprehensive Plan. Vegetation shall be preserved when feasible. Riparian habitat shall be maintained when feasible, during improvements.

1.2.2.2.1. Wherever channel improvements are required to accommodate storm runoff in a specified manner, the designs shall provide maximum practical utilization of turf, sodding, and natural ground surface protection techniques in order to protect the environment by reducing erosion potential.

1.2.2.3. Water quality control measures shall be incorporated into stormwater management designs, subject to approval of the DIRECTOR. Additionally, impacts on receiving water quality shall be assessed for all flood management projects.

1.2.2.4. Stormwater drainage systems shall be designed to accommodate drainage requirements with adjacent counties and municipalities.

1.2.3. Acknowledgement of Future Urbanization

1.2.3.1. The City of Moore considers preventive drainage measures to be less costly to the taxpayer than retrofit drainage measures over the total life of the project and will have planned and implemented, where possible, those measures during urbanization.

1.2.3.2. The term Regulatory Storm Discharge shall mean the discharge produced by the 1% (100-Year) frequency storm under current levels of urbanization. Generally, buildings will be required to be constructed two feet above the regulatory storm water surface elevation to account for future urbanization.

1.3 Flood Damage Prevention Regulations

1.3.1. Floodplain Ordinance

1.3.1.1. The City's Flood Damage Prevention Ordinance is found in Chapter 8 of the Moore, Oklahoma Land Development Code – FLOOD AND DRAINAGE.

1.3.1.2. All construction affecting a FEMA floodplain or floodway must comply with current NFIP regulations (44CFR, Part 65) as minimum standards. This may include obtaining a Conditional Letter of Map Revision (CLOMR) prior to construction and/or obtaining a Letter of Map Revision (LOMR) following construction.

1.3.2. Drainage Policies

1.3.2.1. Drainage facilities will be designed as prescribed in Section 7, STORMWATER DETENTION, for existing watershed conditions with appropriate freeboard.

1.3.2.2. Finished floor and building appurtenance elevations will be designed to elevations that are 2 feet above the regulatory water surface to prevent flood damage.

1.4 Local Building Code

1.4.1.1. All adopted building codes shall apply to construction and grading within the Moore city limits.

1.5 Regional and Local Planning

1.5.1. Stormwater Management Master Drainage Plans

1.5.1.1. Recommendations in adopted Master Drainage Plans shall be followed and may differ to some extent from standalone technical criteria.

1.5.1.2. In preparation of the Master Drainage Plans and updates to Master Drainage Plans, the policy, standards and criteria set forth in this manual shall be used as a guideline for identifying required facilities. However, the benefit/cost relationship must also be considered when retrofitting drainage facilities, and therefore the DIRECTOR may relax the criteria as deemed necessary to maximize the benefit of the retrofit.

1.5.2. Operation and Maintenance of Storm Drainage Facilities

1.5.2.1. Continual maintenance of storm drainage facilities is required to ensure they will function as designed. Maintenance of detention facilities involves removal of debris and sediment and repair of the embankment and appurtenances. Sediment and debris must also be periodically removed from channels and storm sewers. Trash racks and street inlets must be regularly cleared of debris to maintain discharge capacity. Channel bank erosion, damage to drop structures, crushing of pipe inlets and outlets, and deterioration to the facilities must be repaired to avoid reduced conveyance capability, unsightliness, and ultimate failure.

1.5.2.2. Maintenance access must be provided to all storm drainage facilities for operational and maintenance purposes through completion of the project. After completion, permanent access shall be protected by a dedicated right of way or easement. The right of way or easement shall be shown on final plats or final development plans and shall clearly state that the purpose is for stormwater management facilities.

1.5.2.3. Easements shall be required for all stormwater management facilities not in public rights of way. **Table 1 -** Required Operations, Maintenance Easements & Reserve Areas (Not Within a Public Right of Way) describes the easement requirements for the different types of Drainage Facilities.

Table 1 - Required Operations, Maintenance Easements & Reserve Areas (Not Within a Public Right of Way)	
DRAINAGE FACILITY	**MINIMUM EASEMENT/RESERVE AREA WIDTH**
1. Storm Sewer	Minimum 15 feet. Easement width determined on a case-by-case basis, based on depth, site conditions and pipe size.
2. Storm Sewer Overflow, Where Required	As required to contain surface overflow in an overland drainage easement. Refer to Section 1.6.3.4.
3. Open Channels that FEMA or the City of Moore considers Regulatory	Top width plus 30 feet on each side for maintenance access and riparian buffer. See Appendix A-10 for example riparian buffer.
4. Post-development regulatory floodplain	Area sufficient to contain the regulatory floodplain plus two feet in elevation. See Appendix A-10 for example riparian buffer.
5. Open Space Detention Facilities	For regional detention facilities - as required to access and contain storage volume and associated facilities plus 15 feet of maintenance access around the perimeter. For onsite detention facilities - as required to access and contain storage volume and associated facilities plus 15 feet of maintenance access around the perimeter. Access to the outlet works shall be provided either via a 15-foot top width or other arrangements approved by the DIRECTOR.
6. Parking Lot and Underground Detention Facilities	As required to access and contain storage volume and associated facilities.

1.5.3. Water Quality Control

1.5.3.1. Refer to Stormwater Management Program document (Small Municipal Separate Storm Sewer System (MS4) ODEQ General Permit OKR04), dated January 2016 or latest edition.

1.5.3.2. Stormwater Low Impact Development practices are highly encouraged. See Section 8 - Low Impact Development (LID) Methods for Improving Soil Permeability

1.5.4. Watershed Transfer of Storm Runoff

1.5.4.1. The inter basin transfer of stormwater between the following watersheds is not allowed:

1.5.4.1.1. North Fork River

1.5.4.1.2. Stream A

1.5.4.1.3. Stream B

1.5.4.1.4. Stream C

1.5.4.1.5. Stream D

1.5.4.1.6. Little River

1.5.4.1.7. Stream E

1.5.4.1.8. Kelley Creek

1.5.4.1.9. Northmoor Creek

1.5.4.1.10. Tributary 0 of Canadian River Tributary 1

1.5.4.1.11. Tributary 2 of Canadian River Tributary 1

1.5.5. Acceptance of Existing Stormwater Drainage Systems

1.5.5.1. The City of Moore will consider acceptance of existing stormwater drainage facilities not constructed under these criteria for ownership and maintenance without modification to the system using the following guidelines:

1.5.5.1.1. The system must be capable of conveying the regulatory storm flow rates using the criteria presented in this MANUAL.

1.5.5.1.2. The system must be reasonably maintainable with legal access to all facilities using the standards for access presented in these criteria.

1.5.5.1.3. Facilities submitted as part of previously approved plats, but not building permits, will be considered for acceptance.

1.5.5.1.4. Channels must meet the minimum standards of:

1.5.5.1.4.1. Maximum side slopes of 4:1
1.5.5.1.4.2. Maximum regulatory storm design flow velocity as set forth in these criteria with suitable vegetation and other erosion control facilities.
1.5.5.1.4.3. The regulatory storm flow must be contained within the channel banks.

1.5.5.1.5. Storm sewer systems must meet the minimum standards of:

1.5.5.1.5.1. Manholes at changes in pipe sizes and vertical alignment.
1.5.5.1.5.2. The requirements for manholes at changes in horizontal alignment will be considered on a case-by-case basis.
1.5.5.1.5.3. Manholes or other appropriate maintenance access must not be spaced farther apart than 500 feet.
1.5.5.1.5.4. The sewer must be structurally sound and not subject to imminent failure. The Director may require video of the pipes at his or her discretion.

1.6 Technical Standards and Criteria

1.6.1. Drainage Design and Technical Criteria

1.6.1.1. For projects that include stormwater conveyance systems and/or detention systems, all hydrologic analyses that are submitted for approval by the CITY shall utilize the computational techniques presented in this Manual.

1.6.1.2. HEC-HMS is the preferred computer program for hydrologic analysis. Other models may only be used with the approval of the DIRECTOR.

1.6.1.3. Stormwater conveyance systems and detention ponds shall be sized to pass the regulatory storm. The finished floor elevation of all buildings must be a minimum of 2 feet above the regulatory storm water surface elevations.

1.6.1.4. In order to compare the effects of the project, existing conditions shall be computed and compared to the "post project" conditions at the point of discharge from the project and at stream points downstream due to hydrograph timing changes, as specified by the DIRECTOR to ensure that there is no increase in the discharges.

1.6.2. Storm Runoff Determination

1.6.2.1. Unit Hydrograph method

1.6.2.1.1. The City requires that the timing of peak flows be taken into account by using a hydrograph method for computing storm runoff for the design of stormwater channels, bridges and stormwater detention facilities, as well as storm sewer systems draining more than 20 acres. These systems shall be designed to pass the Regulatory Storm flow rates under the current level of urbanization with at least two feet of freeboard to any building finished floor elevations.

1.6.2.1.2. Unit hydrograph computations are required for all hydrologic studies for stormwater detention ponds and for any drainage basin larger than 20 acres for inlet and pipe design.

1.6.2.2. Rational method

1.6.2.2.1. The Rational Method may be used for individual curb and gutter inlet design and for storm sewer design for drainage basins of 20 acres or less. The Rational Method shall not be used for detention pond design or for bridge design. Multipliers shall be applied for all development when the Rational Method is used as specified in Section 3.6.1.1 of this Manual. A "C" value for residential property shall be used as specified in **Table 7** or computed using actual impervious surfaces, whichever is greater.

1.6.3. Drainage Facility Performance:

1.6.3.1. Existing Floodplains

1.6.3.1.1. Existing floodplain storage will be maintained onsite with a watershed drainage area of 40 acres or greater, based on the City's hydrologic and hydraulic models for existing urbanization, in addition to the storage volumes required to mitigate higher peak flow rates.

1.6.3.2. Stormwater Detention

1.6.3.2.1. For the 50% Annual Chance (2-year), 20% Annual Chance (5-year), 10% Annual Chance (10-year), 4% Annual Chance (25-year), 2% Annual Chance (50-year) and the 1% Annual Chance (100-year) storms, no increase in peak flow rate or shortening of the time to peak is allowed, i.e., the rising side of the post-project hydrograph must match the rising side of the pre-project hydrograph.

1.6.3.2.2. No increases in flow rates are allowed downstream from the facility based on the City's hydrologic and hydraulic models of the creek for the current level of urbanization.

1.6.3.2.3. All embankment structures six feet in height or greater shall meet Oklahoma Water Resources Board (OWRB) high hazard dam requirements for spillway capacity, regardless of downstream development. **Table 2 -** Freeboard Requirements for Stormwater Detention Facilities shows the freeboard requirements for all embankment stormwater detention ponds between 18-inches and 6 feet in height in the City of Moore:

Table 2 - Freeboard Requirements for Stormwater Detention Facilities

Embankment or Excavated Pond	Regulatory water surface elevation depth	Regulatory water surface elevation	0.2% Annual Chance (500-year) water surface elevation
Embankment or Excavated	< 18-inches	Contained within a dedicated stormwater detention easement	No freeboard requirement related to the 0.2% storm
Embankment	18-inches to 6 feet	Contained within the detention facility with one foot of freeboard to the top of the embankment*	Contained within the detention facility with no freeboard to the top of the embankment *
Embankment	> 6 feet	Requirements based on 0.2% water surface elevation	Contained within the detention facility one foot of freeboard to the top of the embankment *
Excavated	> 18-inches	Contained within the detention facility with one foot of freeboard to the top of the surrounding grade*	No freeboard requirement related to the 0.2% storm
*unless more stringent OWRB dam safety requirements control, as outlined in Title 785:25-3-3 of the Oklahoma Administrative Code[1]			

[1] Title 785:25-3-3 of the Oklahoma Administrative Code, http://www.owrb.ok.gov/rules/pdf/current/Ch25.pdf

1.6.3.2.4. It must be shown that the regulatory flow rates can pass safely through the proposed development with two feet of freeboard to any finished floor elevations.

1.6.3.3. Open channels

1.6.3.3.1. Open channels must pass the Regulatory Storm with two feet of freeboard for existing levels of urbanization.

1.6.3.3.2. New open channels with drainage areas larger than 40 acres must be constructed as earth or otherwise pervious channels with non-erosive velocities consistent with channel lining.

1.6.3.3.3. Buildings adjacent to the open channel shall be constructed with finished floors a minimum of two feet above the regulatory storm.

1.6.3.4. Storm sewers and overland relief swales

1.6.3.4.1. Storm sewer systems shall be designed to convey the regulatory storm flow rate with an overland drainage easement assuming the pipe is completely blocked. Buildings on either side of the easement side must have two feet of freeboard above the Regulatory Storm water surface elevations.

1.6.3.5. Bridges and Culverts

1.6.3.5.1. All bridges shall be designed to pass the flow produced by the regulatory storm with two feet of freeboard from the water surface to the low chord of the bridge or the inside top of a culvert or culverts with a clear opening of 20 feet in width or more.

1.6.3.5.2. Culverts shall be designed to pass the flow produced by the Regulatory Storm with two feet of freeboard from the water surface to the inside top for structures under roadways for which backwater from 100% blockage would flood existing upstream properties, regardless of the flow rate.

1.6.3.5.3. Culverts with a clear opening less than 20 feet in width may be designed to pass the flow produced by the Regulatory Storm with maximum headwater to culvert diameter (or rise) ratio of 1.5 with two feet of freeboard below the top of the road. These culverts shall be designed to have overland relief in an overland drainage easement or right-of-way (including street overtopping) assuming 100% blockage of the culvert. Culverts shall be designed such that backwater from the culvert does not violate the requirement that buildings have finished floor elevations at least two feet above the Regulatory Storm water surface elevations or the roadway overtopping elevation, whichever is higher.

1.6.3.5.4. Culverts and embankment protective measures shall be designed to minimize embankment damage during overflow.

1.7 Roadway Drainage Systems

1.7.1. Roadway drainage design will meet the following requirements:

1.7.1.1. The furthest upstream set of inlets will be placed at the location where the combination of street flow and flow in the storm sewer reaches curb depth during a 2% Annual Chance (50-year) storm.

1.7.1.1.1. In no case shall the 10% Annual Chance (10-year) storm exceed curb depth.

1.7.1.1.2. Where street grades transition to a flatter slope, calculations shall be provided to show the flatter slope conforms to street capacity criteria above.

1.7.1.1.3. Where drainage areas greater than 2 acres drain into the street, calculations shall be provided to show the combined flow conforms to street capacity criteria above.

1.7.1.1.4. Where street flow will enter intersections and a storm sewer system is present, inlets shall be place upstream of the intersection to capture flow.

1.7.1.2. In any case the Regulatory Storm flow must be contained within the right of way.

1.7.1.3. Roadside drainage ditches are allowed in A-1, A-2 and RE development areas. They must contain the 10% Annual Chance (10-year) existing storm.

1.7.1.4. Storm sewer lines shall be placed behind the curb if possible or in the center of driving lanes, but never in the wheel path of any street, or along the centerline of arterial streets with an even number of lanes.

1.8 Drainage Improvement Responsibility

1.8.1. Downstream Effects

1.8.1.1. All development, including single lot and infill development that increases the total impervious area above that which previously existed and/or concentrates the flow offsite in a manner different from that which previously existed and is detrimental to adjacent properties shall have mitigating stormwater controls.

1.8.2. Downstream Drainage System Capacity

1.8.2.1. No increased flow from development will be allowed beyond the capacity of the downstream drainage system.

1.8.3. Fee-In-Lieu of Onsite Detention

1.8.3.1. All development, including infill development, may pay a fee-in-lieu of onsite stormwater detention, subject to the discretion of the DIRECTOR depending on its location in the watershed and the potential for adverse impacts downstream. The property owner's engineer must submit his or her recommendation for allowing a fee-in-lieu of onsite detention to be paid, along with all supporting data as outlined in **Section 2.1**.

1.8.4. Site Drainage Plans

1.8.4.1. All development projects shall submit drainage plans and a Drainage and Detention Report, described in **Section 2.1.1**. Single lot residential projects may be required to submit drainage plans and/or a Drainage and Detention Report if so required by the DIRECTOR as outlined in **Sections 2.2** and **2.32.2.**

1.9 Lot Drainage

1.9.1. General Requirements

1.9.1.1. Impervious cover cannot exceed the limits given in the City of Moore Zoning Code.

1.9.1.2. Each lot is required to have designated finished floor elevation two feet or more above the Regulatory Storm water surface elevation, and to accept and convey off-site drainage of upstream areas.

1.9.1.3. The development on each lot shall not alter the pre-development course of water flowing onto the lot in such a manner as to restrict drainage from the upstream areas or to cause additional damage to upstream buildings.

1.9.1.4. The development on each lot shall not produce off-site drainage in such a manner as to cause additional damage to downstream buildings.

1.9.1.5. Off-site drainage from each lot shall be diverted into a public storm water conveyance system or, if that is not possible, off-site drainage shall be accomplished in a manner to be approved by the CITY such that no additional damage to downstream buildings occurs. In any case, no more than 2 lots or ½ acre shall be allowed to drain onto an adjacent lot unless it drains into an approved stormwater drainage system component within a drainage easement.

1.9.1.6. Drainage paths or swales shall be constructed directly adjacent to buildings, ensuring drainage around and away from the building. This grading shall have a minimum fall of 6 inches within the first 10 feet.

1.9.1.6.1. Any point along the finished floor perimeter of a building shall be a minimum of 12” above the nearest drainage path invert elevation, unless the requirement in Section 1.9.1.6.2 is a higher elevation than Section 1.9.1.6.1., based on the building pad and finished floor elevations shown on the certified as-built grading plan, signed and sealed by a licensed surveyor, submitted with the Final Plat.

1.9.1.6.2. Any point along foundation walls of a building, the lowest elevation for adjoining grade shall be a minimum of 12” above the nearest drainage path invert elevation. If lot lines, walls, slopes or other physical barriers prohibit 6 inches of fall within 10 feet, the adjoining grade must still be a minimum of 12” above the nearest drainage path invert elevation; consistent with Section R401.3 Drainage from the 2015 International Residential Code.

1.9.1.6.3. Impervious surfaces within 10 feet of the building foundation shall be sloped a minimum of 2 percent away from the building.

1.9.1.7. Crawlspaces shall not be used for mechanical and electrical equipment or storage purposes of any kind except for that area of the crawlspace that is 12” above the flow line of the street drainage system at the point where surface water from the front of the lot drains into said drainage system. If surface water from the front of the lot does not drain to the street, only the area within the crawlspace that is one foot above the lowest adjacent grade may be used.

1.9.1.8. Garage conversions to living space must comply with the requirements is this section using the finished floor elevation of the converted garage.

1.9.1.9. Driveways shall be shaped so that the high point of the driveway is at least 6-inches higher than the adjacent gutter or edge of paving so that water flowing against the curb will not turn into the driveway as it continues downhill.

1.9.1.10. Storm water runoff from residential buildings that is collected in an underground collection system shall be directed to the street drainage system with a pop-up discharge at the property line.

1.9.2. Lot Drainage for Developments Platted in 2015 or Later

1.9.2.1. The portion of the subdivision plans dealing with each individual lot will be reviewed by the City for each building permit to identify and define solutions to drainage problems at each lot.

1.9.3. Lot Drainage in Developments Platted before 2015, Infill Lots and Unplatted Properties

1.9.3.1. A single lot grading and drainage plan shall be submitted showing the flow pattern of stormwater as it enters and the leaves the property.

1.9.3.2. After review of the grading and drainage plan by the Community Development Department, the City may require a more detailed Drainage and Detention Report in accordance with **Section 2.3**.

2. DRAINAGE AND DETENTION REPORT REQUIREMENTS

2.1 Drainage and Detention Report Requirements for New Subdivisions Prior To Filing A Final Plat

2.1.1. Full Drainage and Detention Report Requirements for Residential Subdivisions or Commercial Developments Greater Than One Acre

2.1.1.1. Summary Statement

2.1.1.1.1. The report shall contain a certification sheet as follows:

"I hereby certify that this report (plan) for the drainage design of (Name of Development) was prepared by me (or under my direct supervision) in accordance with the provisions of City of Moore Stormwater Design Criteria Manual for the owners thereof."

________________________________Registered Professional Engineer

State of Oklahoma No.______________

(Affix Seal) Date_______________

CA#_____________________________

2.1.1.2. GENERAL LOCATION AND DESCRIPTION

2.1.1.2.1.1. Location
2.1.1.2.1.2. Name and address of Legal Owner
2.1.1.2.1.3. Vicinity sketch
2.1.1.2.1.4. Legal description of property
2.1.1.2.1.5. Boundary line survey
2.1.1.2.1.6. Township, range, section, 1/4 section
2.1.1.2.1.7. Local streets within and adjacent to the subdivision
2.1.1.2.1.8. Major drainageways and facilities
2.1.1.2.1.9. Names of surrounding developments

2.1.1.2.2. Description of Property

2.1.1.2.2.1. Area in acres
2.1.1.2.2.2. Ground cover (type of trees, shrubs, vegetation)
2.1.1.2.2.3. Major drainageways and SFHA designation
2.1.1.2.2.4. Soil types and Hydrologic Soil Groups

2.1.1.3. DRAINAGE BASINS AND SUB-BASINS

2.1.1.3.1. Major Basin Description

2.1.1.3.1.1. Reference to major drainageway planning studies such as Master Drainage Plans, flood hazard delineation reports, flood insurance rate maps and LOMR's.
2.1.1.3.1.2. Major basin drainage characteristics

2.1.1.3.1.3. Identification of all drainage system components within 50-feet of the property boundary.

2.1.1.3.1.4. Overall drainage area boundary and drainage sub-area boundaries.

2.1.1.3.2. Sub-Basin Description

2.1.1.3.2.1. Historic drainage patterns of the property in question

2.1.1.3.2.2. Off-site drainage flow patterns and impact of development

2.1.1.4. DRAINAGE DESIGN CRITERIA

2.1.1.4.1. Regulations

2.1.1.4.1.1. Discussion of the optional criteria selected or the deviation from the MANUAL, if any.

2.1.1.4.2. Development Criteria Reference and Constraints

2.1.1.4.2.1. Previous drainage studies (i.e., project masterplans) for the site in question that influence or are influenced by the drainage design and how the plan will affect drainage design for the site.

2.1.1.4.2.2. Discussion of the drainage impact of site constraints such as streets, utilities, railways, existing structures, and development of site plan.

2.1.1.4.3. Hydrological Criteria

2.1.1.4.3.1. Design rainfall values used.

2.1.1.4.3.2. Hydrologic analysis for runoff and on-site or regional stormwater detention facilities as required.

2.1.1.4.3.3. Hydrologic analysis for compensatory storage requirements for any alterations of the floodplain.

2.1.1.4.3.4. Runoff calculation method including precipitation loss method and hydrologic soil groups.

2.1.1.4.3.5. Hydrologic analysis for runoff to insure conveyance.

2.1.1.4.3.6. Stormwater detention facility discharge and storage calculation method

2.1.1.4.3.7. Design storm recurrence intervals

2.1.1.4.3.8. Discussion and justification of any criteria or calculation methods used that are not presented in or referenced by the MANUAL.

2.1.1.4.4. Hydraulic Criteria

2.1.1.4.4.1. Routing of off-site drainage flow through the development.

2.1.1.4.4.2. Location of watercourse and the appropriate hydraulic analysis for any alteration of a watercourse.

2.1.1.4.4.3. Hydraulic analysis for runoff to insure conveyance.

2.1.1.4.4.4. Hydraulic analysis for compensatory storage requirements for any alterations of the floodplain.

2.1.1.4.4.5. References for calculation of stormwater detention facility capacity

2.1.1.4.4.6. Detention outlet type

2.1.1.4.4.7. Grade control structure criteria used
2.1.1.4.4.8. Discussion of any drainage facility design criteria used that are not presented in the MANUAL.

2.1.1.4.5. Submittal for Hydrology and Hydraulics

2.1.1.4.5.1. Electronic copies of all hydrologic and hydraulic computer models will be included in the review submittal.

2.1.1.5. DRAINAGE FACILITY DESIGN

2.1.1.5.1. General Discussion of:

2.1.1.5.1.1. Proposed and typical drainage patterns
2.1.1.5.1.2. Compliance with off-site runoff considerations
2.1.1.5.1.3. The content of tables, charts, figures, plates, or drawings presented in the report
2.1.1.5.1.4. Anticipated and proposed drainage patterns

2.1.1.5.2. Specific Discussion of:

2.1.1.5.2.1. Drainage problems encountered and solutions at specific design points
2.1.1.5.2.2. Detention storage and outlet design
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2.1.1.5.2.4. Maintenance access and aspects of the design
2.1.1.5.2.5. Actual maintenance agreement
2.1.1.5.2.6. Easements and/or ROW dedications required

2.1.1.6. CONCLUSIONS

2.1.1.6.1. Compliance with new Standards

2.1.1.6.1.1. Stormwater Management Criteria Manual
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2.1.1.6.1.3. Best Management Practices implemented

2.1.1.6.2. Drainage Concept

2.1.1.6.2.1. Effectiveness of drainage design to control damage from storm runoff.
2.1.1.6.2.2. Influence of proposed development on the Master Drainage Plan recommendation(s).

2.1.1.7. REFERENCES

2.1.1.7.1. Reference all criteria and technical information used

2.1.1.8. REQUIRED REPORT EXHIBITS

2.1.1.8.1. General Location Map

2.1.1.8.2. Floodplain Information

2.1.1.8.3. Drainage Plan

2.1.1.8.4. Map(s) of the proposed development at a scale of 1" = 20' to 1" = 200' on a 22" x 34" drawing shall be included. The plan shall show the following:

2.1.1.8.5. Existing and proposed contours at 1-foot maximum intervals. In terrain where the slope is relatively flat, spot elevations with drainage arrows may be substituted.

2.1.1.8.6. Property lines and easements with purposes noted: Name, address and telephone number of legal owner of property; vicinity sketch.

2.1.1.8.7. Streets, roads and highways adjacent to the property.

2.1.1.8.8. Existing drainage facilities and structures, natural or man-made, including, roadside ditches, drainageways, gutter flow directions, culverts and retaining walls. All pertinent information such as material, size, shape, slope, and location shall also be included.

2.1.1.8.9. Overall drainage area boundary and drainage sub-area boundaries.

2.1.1.8.10. Proposed type of street flow (i.e., vertical or combination curb and gutter), roadside ditch (rehabilitation only), gutter flow directions, and cross pans.

2.1.1.8.11. Proposed storm sewers and open drainageways, including inlets, manholes, culverts, retaining walls, erosion control measures, and other appurtenances.

2.1.1.8.12. Proposed outfall point for runoff from the developed area and facilities to convey flows to the final outfall point without damage to downstream properties.

2.1.1.8.13. Routing and accumulation of flows at various critical points for the minor storm runoff.

2.1.1.8.14. Path(s) chosen for computation of time-of-concentration.

2.1.1.8.15. Details of detention storage facilities and outlet works.

2.1.1.8.16. Location and elevations of all defined floodplains affecting the property.

2.1.1.8.17. Location and elevations of all existing and proposed utilities affected by or affecting the drainage design.

2.1.1.8.18. Routing of off-site drainage flow through the development.

2.1.1.8.19. Construction sequence bar graph.

2.1.1.8.20. Maintenance requirements and schedule.

2.1.1.9. DRAINAGE REVIEW DRAWING SUBMITTAL REQUIREMENTS PRIOR TO FILING A FINAL PLAT

2.1.1.9.1. Drawings must be prepared using an accepted engineering scale of not smaller than 1” = 40’.

2.1.1.9.2. All property lines, final street names, Lot and Block shall be shown.

2.1.1.9.3. Details shown for all Drainage Facilities and Structures

2.1.1.9.4. Contours with 1-foot elevations

2.1.1.9.5. Drainage Flow Arrows for each lot extending around impervious areas.

2.1.1.9.6. Required building pad elevation (may be up to 1’ below minimum finished flood elevation or at or above the FEMA BFE if applicable)

2.1.1.9.7. Building setbacks and easements shown and labeled

2.1.1.9.8. Floodways, Regulatory Floodplain clearly delineated with BFE shown for lots

2.1.1.10. EROSION CONTROL PLAN (SPPP)

2.1.1.11. APPENDICES

2.1.1.11.1. Hydrologic Computations

2.1.1.11.2. Land use assumptions regarding adjacent properties

2.1.1.11.3. Path(s) chosen for computation of time-of-concentration, including lengths types and slopes of each type of flow (grass, concrete, etc.).

2.1.1.11.4. Stormwater runoff at specific design points onsite and offsite.

2.1.1.11.5. Historic and fully developed runoff computations at specific design points

2.1.1.11.6. Hydrographs at critical design points if applicable

2.1.1.11.7. Hydraulic Computations

2.1.1.11.7.1. Culvert capacities
2.1.1.11.7.2. Storm sewer capacity
2.1.1.11.7.3. Street capacity
2.1.1.11.7.4. Storm inlet capacity including inlet control rating at connection to storm sewer
2.1.1.11.7.5. Open channel design
2.1.1.11.7.6. Check and/or channel drop design
2.1.1.11.7.7. Detention area/volume capacity and outlet capacity calculations

2.1.1.11.8. All appropriate FEMA (Federal Emergency Management Agency) submittal data to achieve a Conditional Letter of Map Revision (LOMR) or a Letter of Map Revision (LOMR).

2.1.1.11.8.1. Digital copies of all computer models.

2.2 New Construction Review Requirements in Subdivisions Platted In 2015 Or Later:

2.2.1. A building pad elevation certificate is required prior to issuance of building permit if no certified building pad elevations are on file (must be no more than 1’ below the minimum finished floor elevation or at or above the BFE in or adjacent to a designated floodplain).

2.2.2. City Staff will review all other information from the drainage plan submitted with the engineering review of the neighborhood. The City will review current topography to determine if more than ½ acres drain onto the lot that could potentially cause drainage problems. If questions or issues arise, the City will work with the builder on an individual basis.

2.3 Unplatted Residential and Infill Development Platted Prior To 2015

2.3.1. The following requirements are to be used in the planning, design, and construction of new homes, additions to existing homes, outbuildings, swimming pools, and other significant activities that could change the drainage patterns and characteristics of property that could impact neighboring properties:

2.3.1.1. Flow coming from off-site onto the property cannot be blocked. Flow from off-site must be conveyed so that it does not cause damage to neighbors. Any additional flow originating on the property must be collected and conveyed to the street, if possible, or other approved drainage conveyance facility.

2.3.1.2. All new houses must have roof drainage directed to the street or other approved conveyance (exceptions will be made on a case-by-case basis).

2.3.1.3. All home builders must prepare and submit an elevation certificate demonstrating that the building pad is no more than 1 foot below the established minimum finished floor elevation. Existing topographic maps should be used, if available.

2.3.1.4. All existing drainage pipes and drainage features must be shown on the house plans for infill lots.

2.3.1.5. Existing and proposed flow conditions must be shown on the house plans for infill lots.

2.3.1.6. Accessory structures must have the same drainage documentation as houses if they require a permit.

2.3.1.7. Every new and infill construction residential building permit and land disturbance permit must have an erosion control plan. The plan must be approved prior to installation. Construction, including any fill on the site, cannot be started until the approved plan is in place and inspected by the City. The erosion control measures must continue to be functional and provide the required level of protection throughout the duration of construction.

2.3.2. The UNPLATTED RESIDENTIAL AND PRE-2015 INFILL DEVELOPMENT REPORT shall contain all of the applicable information listed:

2.3.2.1. A certified survey shall be provided at an acceptable engineering scale not smaller than 1:40 that shows building pad elevations, existing contours at the property with a 1-foot maximum interval, and drainage arrows. In terrain where the slope is relatively flat, spot elevations may be substituted. A sketch of the proposed drainage around the building will be provided with the required building pad elevation (may be up to 1’ below minimum finished flood elevation or at or above the FEMA BFE if applicable)~~.~~

2.3.2.2. A drawing shall be provided showing the erosion control plan.

2.3.2.3. Subdivision name, Address and/or Lot/Blk, property lines, setbacks and easements with purposes shall be noted on the drawing.

2.3.2.4. Street names, roads and highways adjacent to the property shall be shown.

2.3.2.5. Location and BFE’s of all defined floodplains affecting the property, pre platting and post construction, shall be shown.

3. RAINFALL AND RUNOFF

3.1 Introduction

3.1.1. Unit hydrograph computations are required for all hydrologic studies for stormwater detention ponds and for any drainage basin larger than 20 acres for inlet and pipe design.. HEC-HMS is the preferred computer program for performing these computations. HEC-HMS is a hydrologic simulation model developed by the US Army Corps of Engineers Hydrologic Engineering Center in Davis, California. Other models may be used with the approval of the DIRECTOR.

3.1.2. The City will allow the use of HydroCAD[2] software for hydrology if it extends far enough downstream to evaluate impacts and includes floodplain storage.

3.2 HEC-HMS - Stream Network Modeling

3.2.1. For use with the HEC-HMS program, a river basin is subdivided into an interconnected system of stream network components using topographic maps and other geographic information. Basin components are developed by the following steps:

3.2.2. The study area watershed boundary is delineated first. This can be done using a topographic map and, in an urban area, supplemented by investigating the storm sewer drainage system.

3.2.3. The watershed is then sub divided into a number of sub-basins as required to accurately model the runoff. Each sub-basin is intended to represent an area of the watershed which, on average, has the same hydrologic properties. These properties shall be described by the NRCS unit hydrograph method (**Section 3.5**). Precipitation loss rates are also described for each sub-basin.

3.2.4. Routing reaches are then determined to convey the hydrographs to downstream points. Routing reaches can be contained within the channel, a combination of channel and overbank flow, all overbank flow, storm sewer flow, or a routing through a reservoir. These routing reaches shall be described by the Kinematic Wave, Storage-Discharge (Modified Puls), or Lag method, depending on the type of routing reach (**Section 3.4.4**.)

3.2.5. Diversions may be required if water leaves a portion of the system. For example, water may leave the overland flow portion of the system at a particular point and enter the storm sewer system where it is then routed downstream by a different method than the overland flow. The storm sewer flow should be diverted from the network and returned at the appropriate point.

3.2.6. Precipitation data is entered into the HEC-HMS model as described in **Section 3.3**.

3.3 Rainfall

3.3.1. Evaluations of Collection System Design, Hydraulic Structure Design, Detention Design, or Floodplain Analysis

3.3.1.1. The design storm for all projects requiring a hydrological analysis shall be the 1% Annual Chance (100-year), 24-hour annual duration storm.

3.3.1.2. In order to determine the effects of the project on more frequent and less frequent flooding, and for water quality considerations, the 50% Annual Chance (2-year), 20% Annual Chance (5-year), 10% Annual Chance (10-year), 2% Annual Chance (50-year), 1% Annual Chance (100-year) and 0.2% Annual Chance (500-year) storms shall be evaluated.

3.3.1.3. In order to compare the effects of the project, existing conditions shall be computed and compared to the "post project" conditions at the point of discharge from the project and at points downstream as specified by the DIRECTOR to ensure that there is no increase in flooding. This applies to all frequency storms that are studied.

3.3.2. Rainfall Depth-Duration Relationship - National Oceanic and Atmospheric Administration (NOAA)

3.3.2.1. NOAA Atlas 14, Point Precipitation-Frequency Atlas of the United States, Volume 8, Version 2.0, provides annual maximum series estimates of rainfall depths for 50% Annual Chance (2-year) through 0.2% Annual Chance (500-year) storms with durations of 5-minutes to 24-hours for the City of Moore and are presented in **Table 3**. These rainfall depth-duration values are for Latitude: 35.3348°, Longitude: -97.4897° (near I35 and SE 4th St.) and shall be used in all HEC-HMS models to calculate existing and future development discharges for frequency storms. The latest rainfall adopted by NOAA can be obtained on the NOAA Atlas 14 website. (https://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html)

Table 3 - Rainfall Depth-Duration Estimates, in Inches

Moore, Oklahoma (Latitude: 35.3348°, Longitude: -97.4897°)							
Duration	Annual Exceedance Probability						
	50%	20%	10%	4%	2%	1%	0.2%
5-min	0.458	0.596	0.705	0.857	0.976	1.10	1.41
10-min	0.670	0.872	1.03	1.25	1.43	1.61	2.06
15-min	0.817	1.06	1.26	1.53	1.74	1.97	2.51
30-min	1.20	1.56	1.85	2.25	2.57	2.90	3.71
60-min	1.58	2.07	2.48	3.05	3.51	3.99	5.22
2-hr	1.96	2.59	3.10	3.85	4.45	5.09	6.72
3-hr	2.19	2.90	3.49	4.36	5.08	5.84	7.83
6-hr	2.60	3.42	4.13	5.19	6.09	7.06	9.64
12-hr	3.04	3.92	4.72	5.93	6.98	8.13	11.2
24-hr	3.48	4.47	5.37	6.75	7.94	9.25	12.8

[2] HydroCAD Stormwater Modeling

3.3.3. Storm Area

3.3.3.1. A storm area size equal to the entire watershed shall be evaluated for area reduction used in the stream network model. The values in **Table 3** represent point rainfall depths and are applicable up to a watershed size of up to 10 square miles. For watershed areas less than 10 square miles, the HMS storm area should be left blank. For watershed areas greater than 10 square miles, the HMS storm area should be set to the largest drainage sub-basin size. Drainage sub-basins should be generally comparable in size.

3.3.4. Storm Duration

3.3.4.1. All hydrologic studies shall use a partial duration of 24 hours.

3.3.5. Computational Time Interval

3.3.5.1. The computational time interval for all hydrologic studies shall be set to a time that is not more than 0.29 times the lag time for the smallest sub-basin. This is a limitation set by the HEC-HMS program to allow the hydrologic model to accurately capture the true shape of the computed hydrographs. A maximum one-minute time interval is required to capture peak flow rates.

3.4 Approved Hydrology Methods

3.4.1. The City of Moore requires that the timing of peak flows be taken into account by using a hydrograph method for computing storm runoff. Unit hydrograph computations will require the use of a HEC-HMS computer model (other models may be used with the approval of the DIRECTOR to simulate the stormwater runoff of the watershed.

3.4.2. The NRCS unit hydrograph method is the approved method for developing hydrographs to compute storm runoff.

3.4.3. The Rational Method may be used to compute peak frequency discharges for inlet and storm sewer design for drainage basins less than 20 acres using appropriate multipliers, found in Section 3.6.1.1 of this Manual. The Modified Rational Method may not be used to design stormwater detention facilities.

3.4.4. There are three approved routing methods to convey hydrographs to downstream points:

3.4.4.1. The Kinematic Wave method may be used in channel and storm sewer routings.

3.4.4.2. The Storage-Discharge (Modified Puls) routing shall be used for channel/overbank and reservoir routings.

3.4.4.3. The Lag method may be used for storm sewer flow only.

3.5 NRCS Unit Hydrograph Method

3.5.1. Introduction

3.5.1.1. The NRCS methodology combines the effect that specific soils and soil cover (i.e., vegetation) have on the runoff from a storm into one parameter called the Soil-

Cover Complex number (CN). For a specific type of land use, soil type, and cover condition in a watershed, a CN value can be determined. Utilizing the total rainfall value and the CN value, the storm runoff volume is then calculated. Lastly, a NRCS unit hydrograph is determined through combination of the runoff volume and lag time for each basin; typically, now performed by hydrologic software.

3.5.1.2. When using the NRCS unit hydrograph method for a basin, the NRCS basin lag time shall be used in conjunction with the CN value to determine runoff.

3.5.2. Curve Number Determination

Table 4 - Runoff Curve Numbers - NRCS Method[3]

Cover Type and Hydrologic Condition	Percent Impervious	A	B	C	D
		Curve Numbers for Hydrologic Soil Group			
Fully Dev. Urban Areas (Vegetation Established)					
Open space (lawns, parks, golf courses, cemeteries, etc.):					
Poor condition (grass cover < 50%)	0	68	79	86	89
Fair condition (grass cover 50% to 75%)	0	49	69	79	84
Good condition (grass cover > 75%)	0	39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc.	100	98	98	98	98
Streets and roads:					
Paved with curbs and storm sewers	100	98	98	98	98
Paved with open ditches	80	83	89	92	93
Gravel	100	98	98	98	98
Dirt	80	72	82	87	89
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing Urban Areas (No Vegetation)					
Newly graded areas - no vegetation	0	77	86	91	94

Note 1: Average runoff condition, and Ia = 0.2S. (AMC II)

Note 2: The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. CN's for other combinations of conditions may be computed using figure 2-3 or 2-4.

3.5.2.1. The hydraulic soil group and surface conditions of a watershed are generally classified separately. A weighted combination of a hydrologic soil group and surface conditions is referred to as a Soil-Cover Complex. A complex can be assigned a runoff potential value or Curve Number (CN). The CN for each watershed in the hydrologic analysis can be derived by first determining the classifications of the soil groups, and then choosing CN's from **Table 4**, weighting the representative combination of surface conditions and known hydrologic soil group.

3.5.2.2. The Natural Resources Conservation Service provides soil survey data for Cleveland County, including the hydrologic soil group classification. These data can be obtained online at https://websoilsurvey.sc.egov.usda.gov/App/WebSoilSurvey.aspx.

3.5.3. Drainage Basin Characteristics

3.5.3.1. The following are required for the NRCS Unit Hydrograph Method:

3.5.3.1.1. Drainage area of the basin

3.5.3.1.2. Longest flow path length

3.5.3.1.3. Characteristics of individual flow paths that make up the longest flow path (e.g., overland, grassed channel, gutter),

3.5.3.1.4. Slope of individual flow paths

3.5.3.1.5. Land use types and areas throughout the basin (e.g., agricultural, residential, business)

3.5.3.2. Basin Lag Time (T_{lag})

3.5.3.2.1. A dimensionless unit hydrograph has been developed by the NRCS based upon the evaluation of a large number of natural unit hydrographs from various watersheds.[4] To determine the NRCS unit hydrograph Basin Lag Time (T_{lag}) for a specific basin, first calculate the Time of Concentration (T_c) for the basin by summing the overland flow times for the various surfaces using the procedures outlined in **Section 3.5.3.2.2**. The Basin Lag Time (T_{lag}) is then calculated using the following equation:

$$T_{lag} = 0.6\ T_c$$

Where: T_{lag} = basin lag time (hours)

T_c = time of concentration (hours)

3.5.3.2.2. The Time of Concentration (Tc) for the basin is made up of two time components, according to the following equation:

[3] SCS TR-55 Table 2-2a – Runoff curve numbers for urban areas

[4] Part 630 Hydrology, National Engineering Handbook, Chapter 16 Hydrographs, Mar 2007

T_c = $t_I + t_T$

Where: T_c = time of concentration (minutes)

t_I = initial, inlet, or overland flow time (minutes)

t_T = travel time in the ditch, channel, gutter, storm sewer, etc. (minutes)

3.5.3.2.3. General Notes on Time of Concentration (Tc) calculations

3.5.3.2.3.1. For urban areas, the time of concentration consists of an overland flow time (t_I) plus the time of travel (t_T) in the storm sewer, shallow channelized flow, paved gutter, roadside drainage ditch, or drainage channel.

3.5.3.2.3.2. For non-urban areas, the time of concentration consists of an overland flow time (t_I) plus the time of travel in a combined form, such as a small swale, channel, or drainageway.

3.5.3.2.3.3. Overland flow, or sheet flow, is the runoff traveling in a thin uniform sheet spread across the ground surface that has not concentrated or collected in drainage flowlines yet. Overland flow time t_I varies with surface slope, surface cover and distance of surface flow. Overland Flow shall be calculated by the Velocity Method[5], described in Section 630.1502 (b) of Chapter 15 of Part 630 Hydrology of the NRCS National Engineering Handbook. The following formula (eq. 15-8 in Part 630.1502 (b), ibid, page 15-6) will be used to compute overland flow:

$$T_t = \frac{0.007(nl)^{0.8}}{(P_2)^{0.5}S^{0.4}}$$

where:

Tt = travel time, h
n = Manning's roughness coefficient (table 15–1 from)
l= sheet flow length, ft
P_2 = 2-year, 24-hour rainfall, in
S = slope of land surface, ft/ft

The length of overland flow will be limited by eq. 15-9 from Part 630.1502 ((b), ibid, page 15-7):

$$l = \frac{100\sqrt{S}}{n}$$

n = Manning's roughness coefficient
l = limiting length of flow, ft
S = slope, ft/ft

[5] https://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=27002.wba, Chapter 15 of Part 630 Hydrology, NRCS National Engineering Handbook, May 2010.

Table 5 - Manning's roughness coefficients for sheet flow (flow depth generally < or = 0.1 ft)

Surface description	n
Smooth surface (concrete, asphalt, gravel, or bare soil	0.011
Fallow (no residue)	0.05
Cultivated Soils:	
Residue cover < or = 20%	0.06
Residue cover > 20%	0.17
Grass:	
Short-grass prairie	0.15
Dense grasses	0.24
Bermudagrass	0.41
Range (natural)	0.13
Woods:	
Light underbrush	0.4
Dense underbrush	0.8
Note: Taken from Table 15-1, Section 630.1502 (b), Chapter 15 of Part 630 Hydrology, NRCS National Engineering Handbook, May 2010.	

3.5.3.2.3.4. At the downstream end of the overland travel reach sheet flow becomes shallow concentrated flow collecting is swales, small rills, and gullies. Therefore, below the overland travel reach the "Grassed Waterway" or "Paved Area & Shallow Gutter Flow" lines or equations in **Figure 1** shall be used.

3.5.3.2.3.5. The latter portion (t_T) of the time of concentration can be estimated from the hydraulic properties of the storm sewer, gutter, swale, ditch, or drainageway, or may be calculated using the "Paved Area (Sheet Flow) & Shallow Gutter Flow" line in **Figure 1**.

3.5.4. Routing of Hydrographs

3.5.4.1. All hydrologic studies involving routing of sub-basin hydrographs shall use one of the following methods in **Table 6**:

Table 6 - Hydrologic Stream Routing Methods

Routing Method	Flow Condition
Kinematic Wave	Flow completely contained in the channel,
	Flow contained in storm sewer
Storage-Discharge (Modified Puls)	Overbank flow
	Reservoir routing
Lag	Flow contained in full storm sewer

3.5.4.2. Kinematic Wave Method

3.5.4.2.1. The Kinematic Wave routing method is appropriate when the flow is contained in a channel or storm sewer where flood wave attenuation is not significant. In those cases, Manning's equation can be simplified to say that the flow rate at any given time is equal to the time rate of change of the cross-sectional area of flow plus the rate of change in flow with distance. The Kinematic Wave routing method is therefore defined in the HEC-HMS model by the reach length, roughness, shape, width or diameter, and side slope of a typical cross section in the routing reach.

3.5.4.3. Storage-Discharge (Modified Puls)

3.5.4.3.1. When flood flows exceed the channel carrying capacity, water flows into the overbank areas and, depending on the characteristics of the overbanks, can be slowed greatly, and often ponding will occur. The Storage Discharge (Modified Puls) routing method accounts for the significant effects that overbank flow has on the attenuation and translation of a flood wave.

3.5.4.3.2. The storage-discharge relationship for a routing reach can be defined by calculating the storage volume (acre-feet) in the reach for each discharge that passes through the reach from below the lowest flow to beyond the highest flow that will be studied, ensuring storage discharge volume values are interpolated and not extrapolated. This can be accomplished by hydraulic analysis of the reach for a range of discharges with HEC-RAS. Care should be taken to include the non-conveyance portions of all the cross sections, such as ineffective flow areas, to correctly account for storage.

3.5.4.3.3. In addition to the storage-discharge relationship for a routing reach, the number of routing "subreaches" must also be determined for use in the HEC-HMS model. The number of subreaches is determined by comparing the hydrograph travel time to the computational interval:

No. of Subreaches = $K/\Delta t$

Where: K = Average Travel Time, in minutes

Δt = Computational Time Interval, in minutes

3.5.4.3.4. Notes:

3.5.4.3.4.1. For example, if the average travel time between the upper and lower ends of a routing reach is 10 minutes and the computational time interval is 2 minutes, the number of subreaches would be 5.

3.5.4.3.4.2. The K value is reported as the "Trvl Tme Avg" variable in the HEC-RAS summary table for the reach. The computational time interval, Δt, is defined in the Control Specifications section of the HEC-HMS model.

3.5.4.4. Reservoir Routing

3.5.4.4.1. Reservoir routing using the Storage-Discharge (Modified Puls) method is accomplished by defining the elevation-area-capacity relationship of the reservoir and by defining the outflow rating curve.

3.5.4.4.2. The outflow rating curve shall take into account all of the available outflow structures (low flow pipes, notched weirs, overflow spillways, etc.). The built-in outflow structures routines in HEC-HMS may be used or individual rating curves may be calculated and added together as appropriate. Care must be taken to properly account for tailwater conditions downstream of the outlets.

3.5.4.5. Lag Method

3.5.4.5.1. The Lag method for routing hydrographs may be used to translate flows in storm sewers that are flowing full from an upstream point to a downstream point. The lag method does not attenuate the peak flow; it merely translates it by the given number of minutes. The lag time can be estimated by assuming 5 feet per second velocity of the flood wave through the storm sewer system.

3.6 Rational Method

3.6.1. Rational Formula

3.6.1.1. The Rational Method is allowed for basins smaller than 20 acres, and is based on the formula:

$$Q = C_f CiA$$

Where: Q = Peak discharge, cubic feet per second

C_f = Frequency factor for less frequent storms

= 1.0 for ≥10% Annual Chance (≤10-year)

= 1.1 for 4% Annual Chance (25-year)

= 1.2 for 2% Annual Chance (50-year)

= 1.25 for 1% Annual Chance (100-year)

C = Runoff coefficient, dimensionless (see **Table 7**)

i = Rainfall intensity for a duration equal to the time of concentration, inches/hour

A = Watershed area, acres

Note: If the product of C_f and C is greater than 1.0, use 1.0 for C x C_f.

3.6.2. Runoff Coefficient

3.6.2.1. Runoff Coefficients for different land use or surface characteristics are found in **Table 7.**

3.6.2.1.1. **Table 7** gives a range of Runoff Coefficients. The higher value shall be used unless justification can be given for use of a lower value.

3.6.2.1.2. If the sub-basin is not homogeneous in its land use type, a composite runoff coefficient should be calculated by averaging the areas of different runoff coefficients.

Table 7 - Runoff Coefficients and Percent Imperviousness for the Rational Method

LAND USE OR SURFACE CHARACTERISTIC	PERCENT IMPERVIOUSNESS	RUNOFF COEFFICIENTS*
BUSINESS:		
Commercial areas	70 to 95	0.70 - 0.90
Neighborhood areas	60 to 80	0.50 - 0.70
RESIDENTIAL:		
Single Family	40 to 60	Use percent impervious for runoff coefficient **or** calculate composite runoff coefficient (0.40 minimum), whichever is greater.
Multi-unit (detached)	45 to 55	
Multi-unit (attached)	65 to 75	
½-acre lot or larger	20 to 40	
Apartments	65 to 75	
INDUSTRIAL:		
Light uses	70 to 80	0.50 – 0.80
Heavy uses	80 to 90	0.60 - 0.90
PARKS, CEMETERIES	4 to 8	0.40 - 0.60
PLAYGROUNDS	10 to 20	0.40 - 0.50
SCHOOLS	40 to 60	0.40 - 0.60
RAILROAD YARDS	35 to 45	0.40 - 0.60
UNDEVELOPED AREAS		
Cultivated	30 to 70	0.40 - 0.60
Pasture	20 to 60	0.40 - 0.50
Woodland	5 to 40	0.40 - 0.50
Offsite flow (land use not defined)	35 to 55	0.40 - 0.90
STREETS:		
Paved	90 to 100	0.70 - 0.95
DRIVES AND WALKS	90 to 100	0.75 - 0.90
ROOFS	85 to 95	0.75 - 0.95

***** The higher value shall be used unless justification can be given for use of a lower value.

3.6.1. Runoff Coefficients for Street Drainage

3.6.1.1. For drainage areas of 20 acres or less, the Rational Method with appropriate multipliers may be used for inlet and storm sewer design only. A runoff coefficient, C, of 0.9 shall be used for all areas adjacent to an arterial street to accommodate future commercial use, unless modified by the use of Low Impact Development (See Section 8).

3.6.1.2. For drainage areas larger than 20 acres the NRCS Unit Hydrograph Method shall be used for inlet and storm sewer design. A curve number, CN, of 95 shall be used adjacent to an arterial street to accommodate anticipated commercial use.

3.6.1.3. The DIRECTOR may allow for special circumstances regarding the actual development assumptions.

3.6.2. Rainfall Intensity

3.6.2.1. The rainfall intensity is the average rainfall rate in inches per hour for the period of maximum rainfall of a given frequency having a duration equal to the time of concentration. The following equations have been modified from the ODOT Roadway Drainage Manual recommendation for Zone 5 to more closely match the very short times of concentration used in inlet design and parking lot stormwater detention design. These formulae shall be used in the Moore area to calculate the average rainfall intensity:

20% Annual Chance (5-Year) Rainfall Frequency $I = \frac{64.0}{(t_c+12.0)^{0.79}}$

1% Annual Chance (100-year) Rainfall Frequency $I = \frac{108.0}{(t_c+15.0)^{0.77}}$

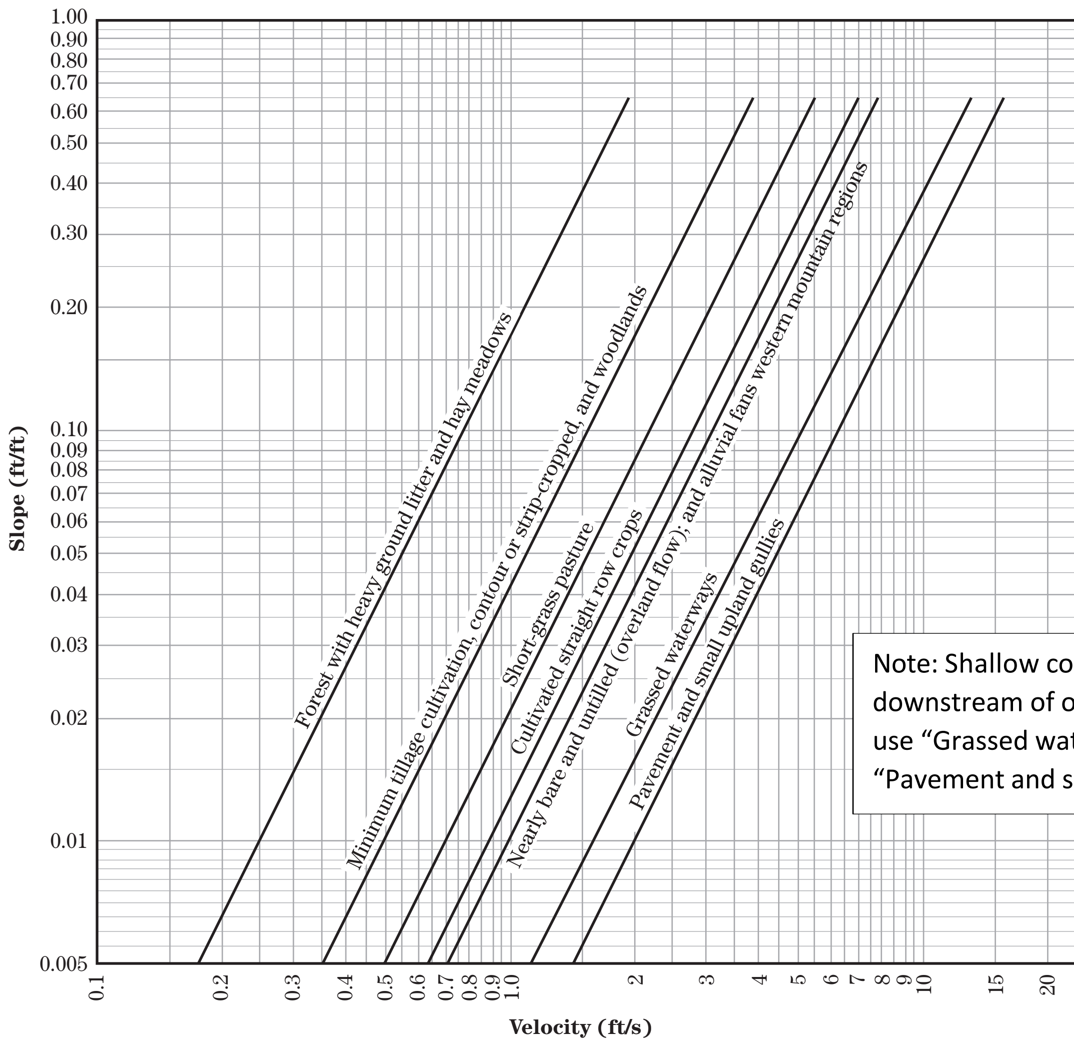
0.2% Annual Chance (500-Year Rainfall Frequency $I = \frac{130.0}{(t_c+15.0)^{0.75}}$

Where: I = Average rainfall intensity, inches/hour

t_b = Rainfall duration (T_c, time of concentration), minutes

Figure 1 - Velocity Versus Slope for Shallow Concentrated Flow[6]

Figure 15–4 Velocity versus slope for shallow concentrated flow



Note: Shallow concentrated flow, downstream of overland flow, must use “Grassed waterways” or “Pavement and small upland gullies”.

Table 15–3 Equations and assumptions developed from figure 15–4

Flow type	**Depth (ft)**	**Manning’s *n***	**Velocity equation (ft/s)**
Pavement and small upland gullies	0.2	0.025	$V = 20.328(s)^{0.5}$
Grassed waterways	0.4	0.050	$V = 16.135(s)^{0.5}$
Nearly bare and untilled (overland flow); and alluvial fans in western mountain regions	0.2	0.051	$V = 9.965(s)^{0.5}$
Cultivated straight row crops	0.2	0.058	$V = 8.762(s)^{0.5}$
Short-grass pasture	0.2	0.073	$V = 6.962(s)^{0.5}$
Minimum tillage cultivation, contour or strip-cropped, and woodlands	0.2	0.101	$V = 5.032(s)^{0.5}$
Forest with heavy ground litter and hay meadows	0.2	0.202	$V = 2.516(s)^{0.5}$

[6] USDA NRCS National Engineering Handbook, Part 630 Hydrology, Chapter 15, May 2010

4. HYDRAULICS OF OPEN CHANNELS

4.1 Introduction

4.1.1. Hydraulic analyses that are submitted for approval by the CITY shall utilize the criteria presented in this Chapter.

4.1.2. Hydraulic analyses will be required for all floodplain studies that include the design or evaluation of bridges, culverts, hydraulic structures, natural open channels and improved open channels.

4.1.3. All hydraulic analyses will require the use of a HEC-RAS[7] computer model (other models may be used with the approval of the DIRECTOR to simulate the flow of water through the study reach.

4.1.4. The model will describe the channel and overbanks for existing and proposed conditions, including all bridges, culverts, and other hydraulic structures. The model will extend far enough upstream and downstream of the project area to sufficiently review negative effects of the project to prevent adverse impacts.

4.1.5. Channels will be designed to carry the 1% Annual Chance (100-year) storm for existing levels of urbanization with a minimum of two feet of freeboard above any superelevation requirements for subcritical flow. For supercritical flow, see Section 4.2.1.4.

4.2 Open Channel Hydraulics

4.2.1. Types of Flow in Open Channels

4.2.1.1. Uniform Flow

4.2.1.1.1. Uniform flow depth (normal depth) occurs only in channel reaches with uniform cross section, roughness and slope (balanced gravitational and resistance forces).

4.2.1.1.2. The depth of flow is the same at every section of the channel reach.

4.2.1.1.3. The water surface is parallel to the channel bottom.

4.2.1.1.4. The energy grade line (EGL) and the bottom slopes are equal.

4.2.1.1.5. Manning's Equation can be used to compute normal depth.

$$Q = \left(\frac{1.49}{n}\right) A R^{\frac{2}{3}} S^{\frac{1}{2}}$$

Where	Q	=	Discharge (cfs)
	n	=	Roughness coefficient[8]
	A	=	Flow Area (ft^2)

[7] https://www.hec.usace.army.mil/software/hec-ras/[8] Chow, V.T., Open Channel Hydraulics, McGraw-Hill Book Company, 1959, Table 5-6, "Values of the Roughness Coefficient n"
[8] Chow, V.T., Open Channel Hydraulics, McGraw-Hill Book Company, 1959, Table 5-6, "Values of the Roughness Coefficient n"

R	=	Hydraulic radius (ft) = A/P
P	=	Wetted perimeter, (ft)
S	=	Energy grade line (EGL) slope (ft/ft)

4.2.1.2. Gradually Varied Flow

4.2.1.2.1. The most common flow regime in stormwater drainage analyses is subcritical flow. Subcritical gradually varied flow typically occurs in the backwater of bridge openings, culverts, storm sewer inlets and channel constrictions. Under these conditions, subcritical gradually varied flow depth will be greater than normal depth in the channel reach.

4.2.1.2.1.1. Occurs due the backwater or headwater created by culverts, bridges, hydraulic structures, or the natural variations in cross sectional configuration (constrictions, bends, changes in roughness, slope, etc.).

4.2.1.2.1.2. Gradually varied flow is indicated by small changes in velocity and depth along the channel.

4.2.1.2.1.3. The flow depth will be greater than normal depth in the channel reach.

4.2.1.2.1.4. The water surface profile must be computed using subcritical downstream to upstream computational techniques (for open channels, performed using HEC-RAS).

4.2.1.3. Critical Flow

4.2.1.3.1. The design of any kind of channel in the critical flow regime is not permitted in the City of Moore.

4.2.1.3.1.1. To determine if the critical flow regime exists in any channel reach, the Froude Number, which is a ratio of the flow’s inertial to weight forces, shall be calculated.

4.2.1.3.1.2. See **Table 10** for design guidelines for the Froude Number in natural channels and improved channels.

4.2.1.3.1.3. If the Froude Number approaches the respective limit for a channel type, measures must be taken to lower the Froude Number. Drop structures are recommended to flatten the slope of the channel.

4.2.1.3.2. The definition of the Froude Number (F) as follows:

$$F = \frac{V}{(gD)^{0.5}}$$

Where	F	=	Froude Number
	V	=	Velocity (fps)
	g	=	Acceleration of gravity (32.2 ft./sec.2)
	D	=	Hydraulic Depth (ft) = A/T
	A	=	Channel flow area (ft2)
	T	=	Top width of flow area (ft)

4.2.1.4. Supercritical Flow

4.2.1.4.1. Supercritical flow (Froude Number exceeds 1.0) may be allowed in concrete lined channels at the discretion of the DIRECTOR under the following conditions:

4.2.1.4.1.1. The Froude Number must be 1.2 or greater to provide stable flow.
4.2.1.4.1.2. Stream velocities are less than or equal to 18 feet per second.
4.2.1.4.1.3. Freeboard will be calculated as follows:

$$2.0 + 0.025Vd^{1/3}$$

Where V = Velocity (ft/sec)
d = depth (ft)

4.2.1.5. Rapidly Varied Flow (Hydraulic Jumps)

4.2.1.5.1. Rapidly varied flow (RVF) is characterized by abrupt changes in the water surface elevation for a given flow. The change in elevation may become so abrupt that the flow profile is virtually broken. RVF is generally caused by hydraulic structures, or sudden changes in slope and geometry. RFV reaches act as a transition area for changes between the two flow regimes, subcritical flow and supercritical flow, or for induction and dissipation of turbulence.

4.2.1.5.2. For this criteria section, the RFV transition from a supercritical flow to a subcritical flow, results in a state of high turbulence and hydraulic jump.

4.2.1.5.2.1. When a hydraulic jump occurs, a great deal of turbulence and erosive forces are generated as the flowing water loses velocity and energy.
4.2.1.5.2.2. It is important to know where and to what extent the jump occurs so that adequate channel protection (e.g., flexible armor or concrete lining) may be provided in that reach. This applies to natural channels as well as to improved channels.
4.2.1.5.2.3. For hard-lined facilities such as pipes or concrete channels, the forces and the change in energy can affect the structural stability or the hydraulic capacity.
4.2.1.5.2.4. For natural and grass lined channels, the erosive forces at the outlet of culverts must be controlled otherwise serious damages will result.

4.2.1.5.3. For improved channel hydraulics, a hydraulic jump in an RVF reach may occur at weir structures, at energy dissipators, at grade control structures (i.e., check drops), inside of or at the outlet of storm sewers or concrete box culverts or at the outlet of an emergency spillway for a detention pond.

4.2.1.5.4. In natural channels hydraulic jumps may occur where there are greatly varying channel configurations or slope and it is therefore necessary to calculate the location and extent of the hydraulic jump so that adequate channel protection may be provided in that reach.

4.2.1.5.5. The calculation of the location and extent of the hydraulic jump can be accomplished using the hydraulic modeling program HEC-RAS for open channels, and Storm CAD for closed storm sewers. Additional or interpolated cross sections are acceptable and likely necessary to accurately model a hydraulic jump in an open channel.

4.2.2. Channel Design Requirements

4.2.2.1. Table 10 lists the design requirements for channels.

4.2.2.2. A letter from the US Army Corps of Engineers is required for all channel projects in jurisdictional streams, informing of the 404 Permit status. This shall be submitted with the design data for the channel.

4.2.3. Riparian/Erosion Buffers and Drainage Easements

4.2.3.1. Any development that occurs along a stream identified in the City's adopted Comprehensive Plan[9], Figure 4.10, for "Riparian Buffer Protection and Restoration", a 50-foot riparian buffer measured from top of bank is required. Any development adjacent to all other streams identified in the City's adopted Comprehensive Plan[10], Figure 4.10 as "Streams", a 25-foot erosion buffer measured from top of bank is required.

4.2.3.2. The 50-foot riparian buffer can be reduced to not less than 25 feet when a reduction in nitrogen of at least seventy-five (75) percent and a reduction in phosphorous of at least fifty-eight (58) percent is achieved through the use of an engineered process that is certified by a licensed Professional Engineer. A development plan using the reduced buffer width shall also document protection against flooding and bank erosion that would be anticipated during the Regulatory Storm flood event. To determine the applicable reduction in the base width of the buffer, **Table 8** below may be utilized to determine pollution removal for a particular structural control. The listed structural controls must be constructed in accordance with the specifications for said control contained in the Wichita/Sedgwick County Stormwater Manual, Volume 2, Section 3.2, Primary TSS Treatment Facilities.[11]

[9] Envision Moore Plan 2040 Comprehensive Plan, Adopted May 15, 2017.
[10] Envision Moore Plan 2040 Comprehensive Plan, Adopted May 15, 2017.
[11] https://www.wichita.gov/PWU/StandardsStormwater/Volume%202-05,%20Chapter%203.pdf

Table 8 - Design Pollutant Removal Efficiencies for Stormwater Controls			
Structural Control	Total Suspended Solids (%)	Total Phosphorous (%)	Total Nitrogen (%)
Stormwater Wet Pond, Wet Extended Detention Pond, Micro-pool Extended Detention Pond, Multiple-Pond System	80	55	30
Extended Dry Detention Pond	60	35	25
Enhanced Swales	90	50	50
Grass Channel	50	25	20
Infiltration Trench	90	60	60
Soakage Trench	90	60	60
Vegetative Filter Strips	50	20	20
Surface Sand Filters	80	50	30

4.2.3.3. The 25-foot erosion stream buffer may be reduced to not less than 15 foot when a plan to prevent streambank erosion is submitted by a licensed professional engineer and accepted by the City of Moore. The following materials may be utilized for streambank stabilization:

4.2.3.3.1. Stabilization Mats

4.2.3.3.2. Rip rap

4.2.3.3.3. Gabion baskets

4.2.3.3.4. Other naturalized stabilization techniques

4.2.3.4. With any riparian or erosion buffer, the following will apply:

4.2.3.4.1. The buffer(s) will not be made a part of a residential or commercial lot but will be placed in either a HOA common area or a drainage right of way dedicated to the City.

4.2.3.4.2. No utility easements will be allowed within the buffer(s).

4.2.3.4.3. A pedestrian easement overlay is required for riparian buffers.

4.2.4. Other Hydraulic Structures

4.2.4.1. Weir Flow

4.2.4.1.1. Weirs are hydraulic structures, commonly used for spillway outlets in detention ponds, at check dams in improved channels, or at overtopping roadways and earthen levees acting as a broad crested weir. The general form of the equation for horizontal crested weirs is:

$$Q \quad = \quad CLH^{\frac{3}{2}}$$

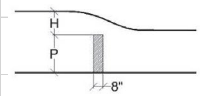
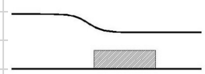
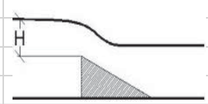
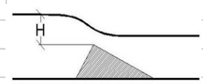
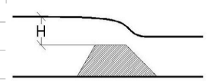
Where Q = discharge (cfs)

C = weir coefficient (see **Table 9)**

H = total energy head (ft)

See the Handbook of Hydraulics, Brater and King[12], for detailed procedures.

Table 9 - Weir Coefficients

WEIR SHAPE	WEIR COEFF.	COMMENTS	SCHEMATIC
Sbarp Crested			
Projection ratio (H/P = 0.4)	3.4	H>l .0	
Projection ratio (H/P = 2.0)	4.0	H> l .0	
Broad Crested			
With sharp upstream conner	2.6	Minimum value is	
With rounded upstream conner	3.0	Critical Depth	
Triangular with vertical upstream slope			
1:1 downstream slope	3.8	H>0.7	
4:1 downstream slope	3.2	H>0.7	
10:1 downstream slope	2.9	H>0.7	
Triangular with 1:1 upstream slope			
1:1 doWI15tream slope	4.1	H> l	
3:1 doWI15tream slope	3.5	H> l	
Trapezoidal section			
1:1 U/S slope, 2:1 D/S slope	3.4	H> l	
2:1 U/S slope, 2:1 D/S slope	3.4	H> l	
Road Crossings			
Gravel	3.0	H> l	
Paved	3.1	H> l	

4.2.4.1.2. Another common weir is the v-notch weir. The general form of the equation is as follows (Reference- 28).

$$Q \quad = \quad 2.5 \tan\left(\frac{\Theta}{2}\right) H^{\frac{5}{2}}$$

Where Θ = angle of the notch at the apex (degrees)

4.2.4.1.3. When designing or evaluating weir flow the effects of submergence must be considered. Also, there are factors to account for contraction and shape of the weir. See the Handbook of Hydraulics, Brater and King[13], for detailed procedures.

[12] Handbook of Hydraulics, King, H.W., and Brater, E.F., Fifth Edition, McGraw Hill Book Company, N.Y., N.Y., 1996.

[13] Handbook of Hydraulics, King, H.W., and Brater, E.F., Fifth Edition, McGraw Hill Book Company, N.Y., N.Y., 1996.

4.2.4.2. Energy Dissipators

4.2.4.2.1. Energy dissipators are used to keep design velocities within acceptable limits for the type of channel by utilizing blocks, sills, or other roughness elements to impose exaggerated resistance to the flow. Typically, the energy dissipator will induce a controlled hydraulic jump to achieve the design downstream channel velocity. Refer to ***HEC 14 - Hydraulic Design of Stilling Basins and Energy Dissipators***[14] for design guidelines.

4.2.4.3. Channel Drops

4.2.4.3.1. The most common use of channel drops is to control the longitudinal slope of channels to keep design velocities within acceptable limits. Refer to ***HEC 14 - Hydraulic Design of Stilling Basins and Energy Dissipators*** for design guidelines.

4.2.4.4. Culvert/Channel Transition Lengths

4.2.4.4.1. A channel discharging into a culvert shall have a transition length equal to the difference between the top width of the channel at the elevation of the regulatory storm and the width of the culvert.

4.2.4.4.2. A culvert discharging into a channel shall have a transition length equal to four times the difference between the width of the culvert and the top width of the channel at the elevation of the regulatory storm.

[14] US Department of Transportation, FHWA, Hydraulic Engineering Circular No. 14, Third Edition (or latest), Hydraulic Design of Energy Dissipators for Culverts and Channels, Publication No. FHWA-NHI-06-086, July 2006 (or latest)

Table 10 - Channel Design Requirements

Channel Type	Natural Channel	Grass Lined Channels	Concrete Lined Channels (where allowed)	Rock Lined Channels	Other Channel Types
Design Flow with 2' Feet of Freeboard	Regulatory Storm	Regulatory Storm	Regulatory Storm	Regulatory Storm	Regulatory Storm
Manning's "n" Values	0.025 to 0.10	0.03 to 0.05 unless not maintained	0.013 to 0.025	0.02 to 0.033	At the discretion of the DIRECTOR with all accompanying design data
Overbank "n" Values	0.03 to 0.20	N/A	N/A	N/A	
Froude Number	Less than 0.9*	< 0.8	<0.8 or >1.2	< 0.8	
Channel Velocity	Obtain from hydraulic computations	6 ft/sec unless sandy soil where max is 5 ft/sec	< 18 ft/sec.	Varies with type	
Minimum Freeboard in Channel (in excess of superelevation)	N/A	2 ft above 1% Annual Chance (100-year) Storm Ex Urb. WSEL	2 ft above Regulatory Storm WSEL for Froude No. <0.8, or $2.0 + 0.025Vd^{1/3}$ above Regulatory Storm WSEL (V=Velocity, d=depth) for Froude No. > 1.2	2 ft above the Regulatory Storm WSEL	
Minimum Freeboard for buildings (above of superelevation)	2 ft above the Regulatory WSEL	2 ft above the Regulatory WSEL	2 ft above the Regulatory WSEL	2 ft above the Regulatory WSEL	
Longitudinal Slope	Natural slope**	As required to maintain velocity and Froude Number; may need grade control structures	As required to maintain velocity and Froude Number; may need grade control structures	As required to maintain velocity and Froude Number; may need grade control structures	
Radius of Curvature	Natural curvature	3 X Top Width and > 100'	2 X Top Width and > 100'	2 X Top Width and > 100'	
Minimum Bottom Width	N/A	4' (width of a trickle channel)	8' for 2:1 side slopes or flatter, 10 feet for vertical side slopes	As required for vegetation and footings for walls.	
Maximum Side Slope	N/A	4:1 or flatter	2:1 unless walls are designed as retaining walls	Steeper than 3:1	
Super Elevation* **	N/A	h = $\frac{V^2Tw}{gr_c}$	h = $\frac{V^2Tw}{gr_c}$	h = $\frac{V^2Tw}{gr_c}$	
Trickle Channel Required	N/A	If channel slope < 1.0%	N/A	N/A	
Riparian Buffers	Easement to Floodplain Boundary	30' Each side – may be reduced to 20' with LID methods	N/A	30' Each side – may be reduced to 20' with LID methods	
Sodding, Seeding and Mulching	N/A	ODOT Stand. Spec. for HWY Constr., Latest Ed., Sec. 230, Sodding and Sprigging	N/A	N/A	
Excluded				Grouted Riprap and Gabion Baskets	

*** Natural channel reaches with a Froude Number (a measure of turbulence) greater than 0.9 for the 1% Annual Chance (100-year) design flow shall be protected from erosion.**

**** Grade control structures like check dams may be required to decrease the flowline slope and to control erosion**

***** h=height of super elevation, V=channel velocity, Tw = top width of design flow, g = 32.2 ft/sec2, r_c = centerline radius of curvature**

5. HYDRAULICS OF CULVERTS, BRIDGES AND OTHER HYDRAULIC STRUCTURES

5.1 Definitions

5.1.1. Bridge

5.1.1.1. A bridge is defined as a hydraulic structure that is constructed with abutments and superstructures which are typically concrete, steel or other materials. Bridges are generally constructed with earth or rock inverts. Since the superstructures are not an integral part of the abutments and could therefore potentially move, the hydraulic criteria for bridges are different than for culverts.

5.1.1.2. A culvert or culverts with a clear opening of 20 feet in width or more are considered a bridge for the purposes of freeboard.

5.1.2. Culvert

5.1.2.1. A culvert is defined in this Section as a closed conduit for the passage of water under an embankment, such as a road, railroad or trail, where flow generally enters by and exits to an open channel, generally at a similar elevation. The geometry of the culvert inlet plays a major role in determining the required size or capacity of the culvert.

5.2 Bridge Hydraulic Design

5.2.1. Bridge Design Standards

5.2.1.1. The following design standards shall apply except as modified by this MANUAL.

5.2.1.1.1. ODOT Bridge Standards & Specifications[15]

5.2.1.1.2. Oklahoma Department of Transportation 2019 Standard Specifications for Highway Construction, (or latest edition)[16]

5.2.1.1.3. FHWA publication: “Hydraulic Design of Safe Bridges”[17], latest edition.

5.2.1.1.4. FHWA publication: “Hydraulic Scour at Bridges”[18], latest edition.

5.2.2. FEMA Requirements

5.2.2.1. All bridges shall follow the applicable FEMA and DIRECTOR’s submission and review requirements as addressed in 44 CFR 60.3 and 44 CFR 65.12, as well as City of Moore, Oklahoma, Land Development Code, Chapter 8, Article A. – Flood Damage Prevention Regulations.

[15] https://www.odot.org/bridge/standards.htm

[16] https://www.odot.org/c_manuals/specbook/2019%20-FULL-SPEC-Web-Version.pdf

[17] US Department of Transportation, Federal Highway Administration, Publication No. FHWA-HIF-12-018, Hydraulic Design Series Number 7

[18] US Department of Transportation, Federal Highway Administration, Publication No. FHWA-HIF-12-003.

5.2.3. Zero Rise

5.2.3.1. There shall be no increase in flooding (zero rise in water surface elevation) for the design discharge and the existing conditions 1% Annual Chance (100-year) discharge upstream or downstream of the bridge for FEMA Zone AE streams.

5.2.4. Bridge Hydraulic Design Program

5.2.4.1. HEC-RAS is the preferred hydraulic design program for bridges.

5.2.5. Design Discharge

5.2.5.1. The design discharge for all bridge structures shall be the Regulatory flood. There will be two feet of freeboard from the resultant water surface elevations to adjacent building finished floors.

5.2.6. Freeboard

5.2.6.1. Freeboard is defined as the vertical clearance of the lowest horizontal structural member of the bridge superstructure or inside top of a culvert as defined in **Section 5.1.1** above the water surface elevation of the design frequency flood. The minimum freeboard shall be two feet for the Regulatory Storm as defined in **Section 1.6.3.5**.

5.2.7. Backwater

5.2.7.1. Backwater is defined as the rise in the flood water surface during the existing 1% Annual Chance (100-year) storm due to the restrictions created by the construction of the bridge. The maximum backwater shall be 1 foot as required by the CITY floodplain regulations for Zone A and additional City regulatory streams and shall not encroach on abutting properties.

5.2.8. Velocity

5.2.8.1. The maximum channel velocity through the bridge opening is limited by the design guidelines for the type of channel and protection provided (see **Table 10**) through the bridge.

5.2.9. Hydraulic Analysis

5.2.9.1. The hydraulic design calculations for all bridges must be prepared and certified by a licensed Oklahoma Professional Engineer using the hydraulic modeling program HEC-RAS (or other program approved by the DIRECTOR). Tailwater elevation must be provided as part of the analysis.

5.2.10. Inlet and Outlet Configurations and Scour Control

5.2.10.1. The design of all bridges shall include adequate wing walls of sufficient length to prevent abutment erosion and to provide slope stabilization from the embankment to the channel. Erosion protection on the inlet and outlet transition slopes shall be provided to protect from the erosive forces of eddy current. See Reference 4 in Section 5.2.1.1. for information on bridge scour considerations.

5.3 Culvert Hydraulic Design

5.3.1. Culvert Design Standards

5.3.1.1. The following Oklahoma Department of Transportation Standards shall apply except as modified by this MANUAL.

5.3.1.1.1. Roadway Drainage Design Manual, Chapter 9, Culverts[19]

5.3.1.1.2. Oklahoma Department of Transportation 2019 Standard Specifications for Highway Construction, (or latest edition)[20]

5.3.1.1.3. FHWA publication: Hydraulic Design of Highway Culverts[21]

5.3.2. FEMA Requirements

5.3.2.1. All culverts shall follow the applicable FEMA and DIRECTOR's submission and review requirements as addressed in City of Moore, Oklahoma, Land Development Code, Chapter 8, Article A. – Flood Damage Prevention Regulations, and 44 CFR 60.3 and 44 CFR 65.12.

5.3.3. Zero Rise

5.3.3.1. There shall be no increase in flooding (zero rise in water surface elevation) for the design discharge and the existing conditions 1% Annual Chance discharge upstream or downstream of the culvert for FEMA Zone AE streams.

5.3.4. Culvert Hydraulic Design Program

5.3.4.1. HEC-RAS is the preferred hydraulic design program for culverts in streams to be modelled for water surface elevations and compensatory storage.

5.3.4.2. HY-8 may be used for single culvert sizing if the culvert is not in a modelled stream.

5.3.4.3. Other design programs may be approved by the DIRECTOR.

5.3.5. Design Discharge

5.3.5.1. The design discharge for all culverts shall be the Regulatory Storm discharge under existing levels of urbanization. There will be two feet of freeboard above the resultant water surface elevations and adjacent building finished floor elevations.

5.3.6. Freeboard

5.3.6.1. Freeboard is defined as the vertical clearance between the inside top of the culvert above the water surface elevation of the design frequency flood. For culverts

[19] https://www.ok.gov/odot/documents/Chapter%209%20Culverts.pdf
[20] https://www.odot.org/c_manuals/specbook/2019%20-FULL-SPEC-Web-Version.pdf
[21] US Department of Transportation, Federal Highway Administration, Publication No. FHWA-HIF-12-026, April 2012, Hydraulic Design Series Number 5

defined as bridges (**Section 5.1.1)**, the minimum freeboard shall be two feet from resultant finished floor elevations and adjacent buildings.

5.3.6.2. For culverts defined in **Section 5.1.2**, culverts, the minimum freeboard shall be two feet from the outside edge of shoulder to the water surface for the Regulatory Storm.

5.3.7. Headwater

5.3.7.1. For culverts defined in **Section 5.1.2**, for the Regulatory Storm discharge, the maximum headwater to culvert diameter (or rise) ratio shall be 1.5

5.3.8. Backwater

5.3.8.1. The maximum backwater shall be 1 foot as required by the CITY floodplain regulations for Zone A and additional City regulatory streams and shall not encroach on abutting properties.

5.3.9. Velocity

5.3.9.1. The minimum velocity in the culvert shall be 3 feet per second for any studied flow rate to assure a self-cleaning condition. The maximum velocity in the culvert shall be 20 feet per second. The velocity at the outlet of the culvert will require channel protection or an energy dissipator according to the design guidelines applicable for the downstream channel type (see **Table 10**).

5.3.10. Hydraulic Analysis

5.3.10.1. The hydraulic design calculations for all culverts must be prepared and certified by a licensed Oklahoma Professional Engineer using the hydraulic modeling program HEC-RAS (or other program approved by the DIRECTOR). Tailwater elevation must be provided as part of the analysis. The hydraulic data presented in **Table 11** shall be used in the design and evaluation of culverts.

Table 11 - Hydraulic Data for Culvert End Losses [22]

Type of Structure and Design of Entrance	Coefficient Ke
• Pipe, Concrete	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, sq. cut end	0.5
Headwall or headwall and wingwalls	

[22] US Department of Transportation, Federal Highway Administration, Publication No. FHWA-HIF-12-026, April 2012, Hydraulic Design Series Number 5

Table 11 - Hydraulic Data for Culvert End Losses [22]	
Type of Structure and Design of Entrance	Coefficient Ke
Socket end of pipe (groove-end	0.2
Square edge	0.5
Rounded (radius = D/12	0.2
Mitered to conform to fill slope (1)	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7^0 or 45^0 bevels	0.2
Side- or slope-tapered inlet	0.2
• Pipe. Or Pipe-Arch. Corrugated Metal (2)	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
*End-Section conforming to fill slope (1)	0.5
Beveled edges, 33.7^0 or 45^0 bevels	0.2
Side- or slope-tapered inlet	0.2
• Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of D/12 or B/12 or beveled	
edges on 3 sides	0.2
Wingwalls at 30^0 to 75^0 to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of D/12 or beveled top edge	0.2
Wingwall at 10^0 to 25^0 to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2
(1) Commonly available from manufacturers	
(2) CGMPs are not allowed for new construction. Provided for evaluation of existing CGMPs only	

5.3.11. Inlet and Outlet Configurations

5.3.11.1. Culverts are to be designed with erosion protection at the inlet and outlet areas. The City of Moore Standard Pipe Headwalls[23] shall be used for culvert entrance and outlet protection. For larger culverts, headwalls and wing walls of similar design are required. Other culvert protection methods, such as flared end sections, may be used with the approval of the DIRECTOR. The headwalls or end section are to be located a sufficient distance from the edge of the shoulder or back of walk to allow for a maximum slope of 3H:1V to the back of the structure.

5.3.12. Construction Materials

5.3.12.1. All culverts within the CITY shall be constructed of reinforced concrete or corrugated smooth interior polypropylene. Reinforced Concrete Box (RCB) culverts or Reinforced Concrete Pipe (RCP) culverts are acceptable. Only culverts sizes 18" inches to 60" inches may be constructed of corrugated smooth interior polypropylene in accordance with City of Moore specifications.

5.3.13. Shapes

5.3.13.1. Numerous cross-sectional shapes are acceptable including circular, rectangular, elliptical, pipe-arch, and arch.

5.3.14. Driveway Crossings

5.3.14.1. Driveway culverts shall be sized to pass the 10% Annual Chance (10-year) storm discharge under existing levels of urbanization. The minimum size culvert shall be an 18" RCP (or equivalent). The effects of the Regulatory Storm discharge on any driveway culvert shall not flood any existing buildings.

[23] https://www.cityofmoore.com/departments/planning-development/public-improvements

6. HYDRAULICS OF STREET DRAINAGE, STORM SEWER INLETS AND PIPE DESIGN

6.1 Street Drainage Policy

6.1.1. Introduction

6.1.1.1. The design of public streets within Moore is performed under the direct control of the DIRECTOR. When the drainage in the street exceeds allowable limits established in **Section 1.7**, a storm sewer system or an open channel is required to convey the excess flows. Street drainage shall be designed to pass the Regulatory Storm discharge within the right of way.

6.1.1.2. Inlets and storm sewers shall be designed to pass Regulatory Storm discharge using the criteria established in **Section 1.7,** within the street right of way.

6.1.1.3. For both residential and non-residential streets, the first inlets shall be located based on the criteria stated in **Section 1.7.1.1**.

6.1.2. Drainage Areas to Inlets in Commercial Developments

6.1.2.1. For commercial areas draining to roadway inlets, drainage areas will be contained within an offset limit of 300 feet from the roadway centerline. The 300 feet offset shall be used for calculation of peak runoff flows to roadway inlets in commercial areas.

6.1.2.2. For commercial areas draining to roadways with drainage areas beyond the 300 feet offset limit are required to drain into an internal storm drainage system that will connect to the roadway storm drainage system.

6.2 Street Drainage Design

6.2.1. Drainage Areas to Inlets

6.2.1.1. Runoff will not be allowed to cross more than two residential lots before entering a public storm drainage system (including streets) as defined in Section 10, GLOSSARY.

6.2.1.2. Concentrated overland flow will not be allowed drain directly to the street. It must enter the right of way in a public system.

6.2.2. Cross Slope

6.2.2.1. For new construction of non-residential streets, the cross slope shall be 1/4 inches per foot, with 3/8 inches per foot required on outside lanes.

6.2.2.2. For new construction residential streets, the cross slope shall be 3/8 inches per foot.

6.2.3. Location of Storm Sewers

6.2.3.1. The preferred location for a storm sewer within a street ROW is behind the curb.

6.2.3.2. Where this is not possible, storm sewers shall be placed in a location that is not within the wheel paths of the pavement.

6.2.4. Driving Lane Inundation

6.2.4.1. For residential streets, the depth of street flow is limited to curb depth or 0.5 feet for all storms up to a 2% Annual Chance (50-year) storm discharge.

6.2.4.2. For arterial and collector streets, the depth of street flow is limited to inundation of the outside lane (typically 0.38’) for all storms up to a 2% Annual Chance (50-year) storm discharge.

6.2.5. Ponding in Sump Locations

6.2.5.1. The depth of ponding permitted in residential streets is limited to 0.5’ for all storms up to a 2% Annual Chance (50-year) storm discharge.

6.2.5.2. The depth of ponding permitted in arterial or collector streets is limited to the outside lane of traffic (typically 0.38’) for all storms up to a 2% Annual Chance (50-year) storm discharge.

6.2.6. Cross Flow

6.2.6.1. Cross flow is allowed at residential intersections only, provided the total flow immediately downstream of the intersection does not exceed the street capacity. Valley Gutters are required for cross flow.

6.3 Hydraulic Evaluation - Curb and Gutter Sections

6.3.1. Allowable Gutter Capacity - Curb and Gutter Sections

6.3.1.1. The flow capacity of a street section with curb and gutter on a continuous grade is calculated using the modified Manning's formula. A uniform section is required for new streets. The composite section is shown for analysis of existing street capacities only.

$$Q = \left(\frac{K_u}{n}\right)\left({S_x}^{\frac{5}{3}}\right)\left({S_l}^{\frac{1}{2}}\right)\left(T^{\frac{8}{3}}\right)$$

or in terms of T

$$T = \left(\frac{Q*n}{K_u \times {S_X}^{\frac{5}{3}} \times {S_l}^{\frac{1}{2}}}\right)^{3/8}$$

Where Q = flow rate (ft/s)

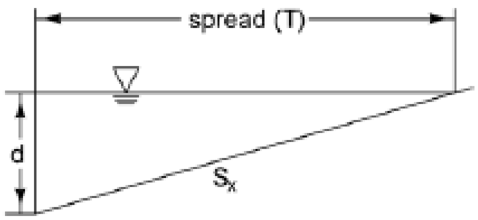
K_u = 0.56 (unitless)

n = Manning's coefficient (.016 asphalt, .013 concrete)

T = width of flow (spread) (ft)

S_x = cross slope (ft/ft)

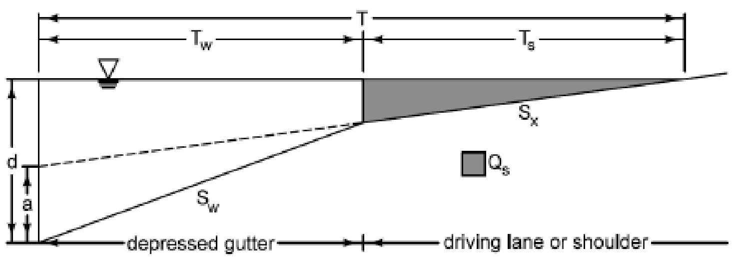
S_l = longitudinal slope (ft/ft)



6.3.1.2. In a uniform section with a Manning's n value of 0.016, a cross slope of 3/8"/ft, and a given flow rate the depth of flow in the gutter, Y_T, is calculated as:

$$Y_T = \left(\frac{Q}{1120 \times Sx^{\frac{1}{2}}}\right)^{\frac{3}{8}}$$

Given $Y_T = S_x T$



6.3.1.3. A composite (depressed) section is more complicated than the simple triangle section and requires different calculations.

6.3.1.4. If a composite section is proposed, see Hydraulic Engineering Circular No. 22 3rd Edition[24] for specific procedures.

NOTE: If a composite gutter section is selected for design, the calculations can be simplified by assuming a uniform cross slope of Sx. A uniform cross slope produces a wider spread than a composite section for the same value of Q. Assuming a uniform cross slope results in more conservative (closer) inlet spacing.

6.3.2. Allowable Inlet Types for Streets and Outside of the Roadways

6.3.2.1. Cast Iron Curb Opening Inlets with Grates on Continuous Grade

6.3.2.2. Cast Iron Curb Opening Inlets with Grates in a Sump Recessed 6" Metal Frame Inlet w/Access Manhole Back of Curb, 4' and 8' length (Residential only)

6.3.2.3. Cast Iron Curb Opening Inlets with Grates in a Sump Recessed 10" Metal Frame Inlet w/Access Manhole Back of Curb, 4' and 8' length (Non-residential only)

6.3.2.4. Standard Drop Inlets (15-inch through 48-inch pipes)

6.3.2.5. Standard Three Way Drop Inlet

6.3.2.6. ODOT Standard Median Drain with Type 1 Grate only

6.3.2.7. Standard drawings, including hydraulic performance charts for clogging, are accessible at https://www.cityofmoore.com/departments/planning-development/public-improvements and https://www.odot.org/roadway/roadway2009/IndexStandards2009.htm.

[24] US Department of Transportation, Federal Highway Administration, Publication No. FHWA-NHI-10-009, September 2009, Revised August 2013

6.3.3. Required Locations of Curb Inlets

6.3.3.1. Curb inlets are required when the allowable depth of flow in the gutter is exceeded. Inlets are also required at the following locations:

6.3.3.1.1. At all low points in the gutter grade.

6.3.3.1.2. On side streets at intersections where runoff would flow onto an arterial street or highway.

6.3.3.1.3. Upgrade from bridges to prevent runoff from flowing onto the bridge deck.

6.3.4. Additional Inlet Requirements

6.3.4.1. Inlets at intersections shall be located in such a manner that no part of the inlet will encroach upon the curb return.

6.3.4.2. Inlets on a continuous grade in the interior of a block should be placed upstream of a nearby driveway.

6.3.4.3. The design drawings shall include the flowline and top of curb elevations on all inlets.

6.3.5. Use of Concrete Inlets

6.3.5.1. Concrete curb opening inlets are not allowed in the City of Moore.

6.3.5.2. Recessed 6” metal frame inlets or cast iron curb inlets are to be used in all new residential neighborhoods.

6.3.5.3. Recessed 10” metal frame inlets or cast iron curb inlets are to be used in all new commercial construction/industrial developments and along arterial and collector streets.

6.3.6. Spacing Between Curb Inlets

6.3.6.1. The spacing between curb inlets shall be such that depth of flow or width of spread requirements is not violated.

6.3.7. Interception and Bypass

6.3.7.1. Inlets on continuous grades may bypass no more than 30 percent of the flow.

6.3.7.2. Depth of flow and width of spread requirements must not be violated unless otherwise approved by the DIRECTOR.

6.3.7.3. The bypassed flow will be added to the design flow for the next downstream inlet.

6.3.8. Clogging Factors

6.3.8.1. Hydraulic design equations and charts presented in this MANUAL were developed with the assumption that all openings are clear, i.e., no portion of the curb or grate opening is clogged with leaves, sticks, cans, mud, or other urban litter.

6.3.8.2. The following clogging factors are required to reduce the theoretical interception given by the hydraulic design charts. A clogging factor of 0.8 is interpreted to mean that the inlet capacity obtained from the equations or charts is multiplied by 0.8 to obtain the allowable capacity, i.e., the allowable capacity of the inlet is 80% of the theoretical capacity.

6.3.8.3. The method by which these clogging factors are incorporated with the hydraulic design charts is detailed in Section 6.4.

6.3.9. Inlets in Sump Condition

6.3.9.1. When inlets are placed in a sump, an emergency overland drainage easement shall be provided in accordance with Policy 305.4.3 based on 100% clogging of the sump inlet.

6.3.10. Gutter Longitudinal Slope

6.3.10.1. Gutter longitudinal slopes shall be no less than 0.5%.

6.4 Storm Sewer Inlet Design Technical Criteria

6.4.1. General

6.4.1.1. The current state-of-the-practice for designing non-recessed storm inlets is presented in the FHWA publication Urban Drainage Design Manual Third Edition[25]. The methods are also described in the 2014 ODOT Roadway Drainage Manual[26], Chapter 10.

6.4.1.2. The guidelines for design of recessed metal curb openings without grates are presented in this chapter and are based on the Kansas Director of Transportation (KDOT) publication K-TRAN Research Project KU-98-3, Hydraulic Performance of Set-Back Curb Inlets and included in the Kansas City Metropolitan Chapter, American Public Works Association, Standard Specifications & Design Criteria [27].

6.4.1.3. All metal hoods shall be stamped "Dump No Waste, Drains to River".

6.4.2. Trench Inlets

6.4.2.1. Trench inlets are allowable in locations where significant roadway drainage has not been intercepted into a storm sewer system. The crown of the street shall be

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[25] https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf
[26] https://www.ok.gov/odot/documents/Chapter%2010%20Stormwater%20Drainage.pdf
[27] Kansas City Metropolitan Chapter, American Public Works Association, Standard Specifications & Design Criteria, Section 5604, Storm Drainage, Systems & Facilities, February 16, 2011 http://kcmetro.apwa.net/content/chapters/kcmetro.apwa.net/file/Specifications/APWA5600.pdf

eliminated over a transition of 10 feet on both sides of the inlet. Vane grates will be positioned to receive flow from upstream on a sloped section and alternately reversed in a sump. Details for the inlet frame and grate are found in the ODOT Storm Sewer Inlet Frames (Curb Inlets) detail. [28]

6.4.3. Storm Sewer Inlet Grates

6.4.3.1. Grated inlets without a curb opening are not permitted within City of Moore streets.

6.4.3.2. The vane grate (in combination with a curb opening) is the only grate approved by the City of Moore within the street ROW.

6.4.4. Curb Opening Inlets

6.4.4.1. Recessed 6” and 10” Metal Frame Inlet w/Access Manhole Back of Curb, 4’ and 8’ length.

6.4.4.1.1. The diagram of the 6-inch and 10-inch inlet can be found at https://www.cityofmoore.com/departments/planning-development/public-improvements. The length of the opening is either 4 feet or 8-feet.

6.4.4.1.2. For the 10-inch opening, the upstream transition is 10 feet and the downstream transition is 5 feet. In a sump condition, both transitions are 5 feet.

6.4.4.1.3. For the 6-inch opening, the upstream transition is 6 feet and the downstream transition is 3 feet. In a sump condition, both transitions are 3 feet.

6.4.4.1.4. Captured flow on Recessed Metal Frame Curb Openings is calculated in the following manner:

6.4.4.1.4.1. Calculate the flow rate for 100% efficiency as follows:

$$Q_o = (a + bL_o)(S_o)^x$$

Where

Q_o = Largest flow that is captured completely
a = -0.35 (for $^3/_8$”/ft or 3% cross slope, 6-inch curb opening), or
a = -0.4 (for ¼”/ft or 2% cross slope, 6-inch curb opening)
a = 1.25 (for $^3/_8$”/ft or 3% cross slope, 10-inch curb opening), or
a = 1.0 (for ¼”/ft or 2% cross slope, 10-inch curb opening)
b = 0.2 (for $^3/_8$”/ft or 3% cross slope, 6-inch curb opening), or
b = 0.1 (for ¼”/ft or 2% cross slope, 6-inch curb opening)
b = 0.25 (for $^3/_8$”/ft or 3% cross slope, 10-inch curb opening), or
b = 0 (for ¼”/ft or 2% cross slope, 10-inch curb opening)
x = -0.78 (for $^3/_8$”/ft or 3% cross slope, 6-inch curb opening), or
x = -0.7 (for ¼”/ft or 2% cross slope, 6-inch curb opening)

[28] https://www.odot.org/roadway/roadway2009/R-40.pdf

x	=	-0.5 (for either cross slope, 10-inch curb opening)
L_o	=	Length of opening in feet (4 feet or 8 feet)
S_o	=	Street grade in percent

6.4.4.1.4.2. If Q_t is equal to or less than Q_o, $Q_t = Q_o$

Where Q_t = Total approach flow

6.4.4.1.4.3. If Q_t is greater than Q_o, Q_c is calculated as follows:

Q_a = $(c + dL_o)(S_o)^X$

Q_c = $Q_O + (Q_a - Q_o)\left[1 - exp\left\{-(\frac{Q_t - Q_o}{Q_a - Q_o})\right\}\right]$

Where Q_a = The upper limit constant on the captured discharge

c	=	3.9 (for $^3/_8$"/ft or 3% cross slope, 6-inch opening), or
c	=	3.5 (for ¼"/ft or 2% cross slope, 6-inch opening)
c	=	2.9 (for $^3/_8$"/ft or 3% cross slope, 10-inch opening), or
c	=	3.2 (for ¼"/ft or 2% cross slope, 10-inch opening)
d	=	1.65 (for $^3/_8$"/ft or 3% cross slope, 6-inch opening), or
d	=	0.8 (for ¼"/ft or 2% cross slope, 6-inch opening)
d	=	1.8 (for $^3/_8$"/ft or 3% cross slope, 10-inch opening), or
d	=	1.7 (for ¼"/ft or 2% cross slope, 10-inch opening)

Q_c = Total captured flow

6.4.4.1.4.4. The bypassed flow (Q_b) is that flow greater than Q_c, or

Q_b = Q_t - Qc

6.4.4.1.4.5. The clogging factor for Recessed 6-Inch and 10-inch Height Metal Frame Curb Opening with Access Manhole Back of Curb on a grade is 1.0 and does not affect this calculation.

6.4.4.2. Recessed Metal Frame Curb Opening with Access Manhole Back of Curb in a Sump[29]

6.4.4.2.1. The flow in the curb opening is weir flow, and is calculated as:

Q = $3.1 \times L \times d^{1.5}$

Where L = Length of Curb Opening

d = Depth of flow

[29] Kansas City Metropolitan Chapter, American Public Works Association, Standard Specifications & Design Criteria, Section 5604, Storm Drainage, Systems & Facilities, February 16, 2011 http://kcmetro.apwa.net/content/chapters/kcmetro.apwa.net/file/Specifications/APWA5600.pdf

6.4.4.2.2. At a depth approximately equal to the height of opening, the flow changes to orifice control, calculated as:

Q = $0.65 \times A \times (2 \times g \times d)^{0.5}$

Where A = Area of Opening

g = 32.2 ft/sec2

d = Depth of Flow above Centroid of Area

6.4.4.2.3. In the transition zone between weir flow and orifice flow, 1.0-1.4 times the opening height, the smaller of the flow calculations will control.

6.4.4.2.4. The clogging factor for a Recessed 6-Inch and 10-inch Height Metal Frame Curb Opening with Access Manhole Back of Curb in a sump is 0.8. The capacity of the inlet is equal to the capacity computed above multiplied by 0.8.

6.4.4.3. Cast Iron Curb Opening Inlets with Grates on Continuous Grade

6.4.4.3.1. See ODOT Roadway Drainage Manual[30], Chapter 10, Section 10.12, or FHWA HEC 22[31], Chapter 4, Section 4.4.4.

6.4.4.4. Cast Iron Curb Opening Inlets with Grates in a Sump

6.4.4.4.1. See ODOT Roadway Drainage Manual Chapter 10, Section 10.12, or FHWA HEC 22, Chapter 4, Section 4.4.5.

6.4.4.5. Drop Inlets

6.4.4.5.1. Drop inlets are only permitted within street rights-of-way with unpaved medians or within local drainage easements or reserves outside of the street ROW.

6.4.4.5.2. Drop inlets will typically operate in a sump condition, since the opening is perpendicular to the direction of flow and the inlet (and local grading) blocks the flow. The inlet will operate as a weir to a depth of flow just above the top of the opening. Above this depth, the inlet operation will transition to orifice control, with the capacity calculated based on a head computed above the centerline of the opening.

6.4.4.5.3. The capacities for drop inlets are calculated using a weir coefficient of 3.1, an orifice coefficient of 0.6, and a clogging factor of 0.8. The inlet operates as a weir to depths equal to the opening height and as an orifice at depths greater than 1.4 times the opening height. At depths between 1.0 and 1.4 times the opening height, flow is in a transition stage.

Q = $3.1Ld^{1.5}$

[30] https://www.ok.gov/odot/documents/Chapter%2010%20Stormwater%20Drainage.pdf

[31] https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf

or d = $\left(\frac{Q}{3.1L}\right)^{\frac{2}{3}}$

Where L = Length of Opening

d = Depth of Flow

6.4.4.5.4. At a depth approximately equal to the height of opening, the flow changes to orifice control, calculated as:

$$Q = 0.6A(2gd)^{0.5}$$

$$\text{or} \quad d = \left(\frac{Q}{0.6A(2g)^{0.5}}\right)^{2}$$

Where A = Area of Opening

g = 32.2 ft/sec2

d = Depth of Flow Centroid of Area

6.4.4.5.5. In the transition zone between weir flow and orifice flow, 1.0-1.4 times the opening height, the smaller of the flow calculations (higher of the depth calculations) will control.

6.4.4.6. Multiple Inlets

6.4.4.6.1. Multiple inlets occur when more than one inlet (of the same type) is used in a continuous series, resulting in greater flow interception capacity. To calculate the capacity of multiple inlets, the most upstream inlet is first evaluated using procedures described above to determine the amount of flow intercepted. Each subsequent inlet is assumed to have efficiency equal to the first inlet and will pick up a proportional amount of the remaining flow.

6.5 Allowable Capacity - Roadside Ditch Sections

6.5.1. Roadside ditch sections shall carry the 10% Annual Chance (10-year) storm discharge under existing levels of urbanization. The effects of the Regulatory Storm discharge on any driveway culvert shall not flood any existing buildings.

6.5.2. Driveway culverts shall be designed to pass the Regulatory Storm discharge within the right of way without encroaching on the roadway. Side slopes must be 1 foot vertical to 2 feet horizontal for depths up to 30-inches. For deeper ditches, the side slopes must be 1 foot vertical to 3 feet horizontal.

6.6 Storm Sewer Pipe Design

6.6.1. General

6.6.1.1. A storm sewer system is required when other parts of the drainage system no longer have capacity for additional runoff without exceeding design criteria.

6.6.2. Design Criteria

6.6.2.1. Design Storm Frequency and Bypass

6.6.2.1.1. To maximize storm sewer efficiency no more than 30% of the approaching flow is allowed to bypass an inlet on grade.

6.6.2.2. Vertical Alignment

6.6.2.2.1. Cover

6.6.2.2.1.1. The sewer grade shall maintain the minimum cover necessary to withstand AASHTO HS-20 loading on the pipe or the pipe manufacturer's recommendation, whichever is greater.

6.6.2.2.1.2. The minimum cover depends upon the pipe size, type and class, and soil bedding condition, but shall not be less than 1 foot from the top of pipe to the finished grade at any point.

6.6.2.2.1.3. If the pipe encroaches into the street sub-grade, approval from DIRECTOR is required.

6.6.2.2.2. Manholes

6.6.2.2.2.1. Manholes will be required whenever there is a change in size, alignment, or slope and where there is a junction of two or more sewers.

6.6.2.2.2.2. The maximum spacing between manholes for various pipe sizes shall be in accordance with **Table 12.**

6.6.2.2.2.3. For large storm sewers (i.e.: cross sectional area greater than 25 square feet), manholes may be placed at a maximum distance of 500 feet.

Table 12 - Storm Sewer Alignment and Size Criteria

A.	MANHOLE SPACING:		
	PIPE SIZE	MAXIMUM SPACING	MINIMUM MANHOLE
		FOR MANHOLES	SIZE
	15" to 24"	300 ft	4 ft
	27" to 36"	400 ft	5 ft
	42"	400 ft	6 ft
	48"	500 ft	6 ft
	54" to 66"	500 ft	8 ft
	> 66"	500 ft	Junction Structure
B.	MINIMUM PIPE DIAMETER:		
	TYPE	MINIMUM EQUIVALENT PIPE DIAMETER	MINIMUM CROSS-SECTIONAL AREA
	Main Trunk	18 in	1.77 sq. ft.
	Lateral from inlet [1]	18 in	1.77 sq. ft.

[1] Minimum size of lateral shall also be based upon a water surface inside the inlet with a minimum distance of 1' below the grate or throat.

6.6.2.2.3. Clearance between utility lines

6.6.2.2.3.1. For new construction, the minimum clearance between storm sewer and water mains or storm sewer and sanitary sewer, either above or below, shall be 24 inches.

6.6.2.2.3.2. When a 24” clearance between existing water mains cannot be obtained, ductile iron pipe (with proper bedding) or concrete encasement of the water line will be required.

6.6.2.2.3.3. When a 24” clearance between existing sanitary sewer cannot be obtained, the sanitary sewer shall have an impervious encasement or be constructed of structural sewer pipe (i.e.: ductile iron pipe) for a minimum of 10-feet on each side of the storm sewer crossing.

6.6.2.2.4. Siphons

6.6.2.2.4.1. Siphons or inverted siphons are not allowed in the storm sewer system.

6.6.2.3. Horizontal Alignment

6.6.2.3.1. Radius Pipe is not allowed.

6.6.2.3.2. A minimum horizontal clearance of 5 feet is required between sanitary sewer or water utilities and the storm sewer.

6.6.2.4. Pipe Size

6.6.2.4.1. The minimum storm sewer diameter allowed in a closed system is 18 inches.

6.6.2.4.2. The minimum pipe size for an open culvert is 18 inches.

6.6.2.4.3. If a lateral pipe extends outside of the street ROW or easements, then manholes shall be included on the lateral within the street ROW.

6.6.2.5. Storm Sewer Capacity

6.6.2.5.1. Storm sewers may be surcharged when approved by the DIRECTOR, and when adequate joint treatment and/or depth is specified to prevent joint separation.

6.6.2.6. Storm Sewer Velocities

6.6.2.6.1. The velocity shall be based on the Manning's n-values presented in **Table 13.**

6.6.2.6.2. The maximum full flow velocity shall be less than 20 fps.

6.6.2.6.3. To avoid excessive accumulations of sediment, the minimum velocity in a pipe based on half-full flow shall be 2.5 fps.

Table 13 - Manning's N-Values for Pipe Materials

MATERIAL	N-VALUE
(A) - CONCRETE	
Pre-Cast	0.013
Cast-in-Place	
Steel forms	0.013
Wood forms	0.015
(B) - PLASTIC	
Corrugated Polyethylene	0.012
Corrugated Polyethylene (smooth interior)	0.011
Polyvinyl chloride (smooth interior)	0.012

6.6.2.7. Energy and Hydraulic Grade Restrictions

6.6.2.7.1. The energy grade line (EGL) for the design flow shall be no more than one foot above the final grade at manholes, inlets, or other junctions.

6.6.2.7.2. The HGL shall not exceed the grate or weir elevations of inlet or manholes unless approved by the DIRECTOR. In some conditions, use of a bolt-down manhole cover may be allowed.

6.6.2.7.3. The hydraulic grade line (HGL) and the energy grade line (EGL) shall be calculated by accounting for friction, expansion, contraction, bend, manhole, and junction losses, as described in Section 6.6.3.3.

6.6.2.8. Storm Sewer Outlets

6.6.2.8.1. All storm sewer outlets into open channels shall be constructed with a slope wall or prefabricated culvert end section.

6.6.2.8.2. Erosion control shall be provided at the outlet.

6.6.3. Hydraulic Evaluation

6.6.3.1. Evaluation of Head Losses in Storm Sewer Systems

6.6.3.1.1. The methodology in FHWA HEC-22[32] is the standard for all Hydraulic Grade Line (HGL) calculations and is included herein as a reference. The ODOT Roadway Drainage Manual[33] (November 2014) details and streamlines the methods for evaluating the hydraulic grade line in storm sewer systems as "ODOT Practice". Section 10.15 of the ODOT Roadway Drainage Manual – HYDRAULIC GRADE LINE CALCULATION – provides a step-by-step approach to

[32] FHWA HEC 22

[33] ODOT Roadway Drainage Design Manual, November 2014, https://www.ok.gov/odot/Doing_Business/Pre-Construction_Design/Roadway_Design/Support_Units/Oklahoma_Roadway_Drainage_Manual.html

verifying that the HGL does not rise above the ground surface, potentially surcharging inlets, causing joint displacement and lifting manhole covers.

6.6.3.1.2. Computation of the hydraulic grade line of a storm drainage run will not be necessary where these three conditions exist:

6.6.3.1.2.1. the slope and the pipe sizes are chosen so that the slope is equal to or greater than the friction slope,
6.6.3.1.2.2. the top surfaces of successive pipes are aligned at changes in size (rather than flow lines being aligned), and
6.6.3.1.2.3. the surface of the water at the point of discharge does not rise above the top of the outlet.

6.6.3.1.3. The pipe will not operate under pressure in these cases, and the slope of the water surface under capacity discharge will approximately parallel the slope of the pipe invert.

6.6.3.2. Tailwater

6.6.3.2.1. For most design applications where the flow is subcritical, the tailwater will either be above the crown of the outlet or can be between the crown and critical depth. To determine the EGL, begin with the tailwater elevation or the average of the critical depth and height of culvert (i.e.(d_c + D)/2), whichever is higher, add the velocity head for full flow and proceed upstream to adding appropriate losses (e.g., exit, friction, junction, bend, entrance).

6.6.3.3. Energy Losses Required to be Included in the HGL and EGL Evaluation

6.6.3.3.1. Exit Losses.

6.6.3.3.1.1. Note that if discharging into a pond, the loss will be one velocity head. If discharging into a stream at an angle flatter than 90o, the loss is based on the difference in velocity head.

$$H_o = C_o\left[\left(\frac{V^2}{2g}\right) - \left(\frac{{V_d}^2}{2g}\right)\right]$$

Where:

V = average outlet velocity (fps)
V_d = channel velocity downstream of outlet (fps)
C_o = exit loss coefficient = 1.0 (unitless)

6.6.3.3.2. Pipe Friction Losses.

6.6.3.3.2.1. The friction loss, assuming full flow, is dependent on the type of pipe, the length of travel (L) and the area of flow (A). The friction slope is the energy gradient in ft/ft for that run. The friction loss is simply the energy gradient multiplied by the length of the run in feet. Energy losses from pipe friction may be determined by rewriting Manning's equation in terms of the friction slope:

$$S_f = \left[\frac{Q_n}{1.486AR^{2/3}}\right]^2$$

Where:

Q_n = discharges (ft^3/s)
R = hydraulic radius
A – flow area

6.6.3.3.2.2. The head losses due to friction may be determined by the formula:

$$H_f = S_f L$$

6.6.3.3.3. Bend Losses.

6.6.3.3.3.1. The bend loss coefficient for storm drainage system design is minor but can be evaluated using the formula:

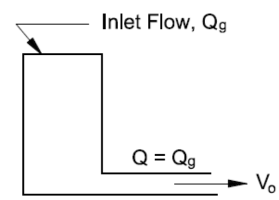
$$h_b = 0.0033(\Delta)\left(\frac{V^2}{2g}\right)$$

Where:

Δ = angle of curvature (degrees)

6.6.3.3.4. Manhole/Inlet Losses (ODOT Manhole Loss Method[34]) for Open Channel or Full Barrel Flow

6.6.3.3.4.1. Terminal Manhole Losses



A manhole is called terminal manhole when it has only outflow pipe. This manhole receives the runoff water directly through gutter inlets. The head loss Ht through a terminal manhole is computed as:

$$H_t = 1.0\, x \left(\frac{V^2}{2g}\right)$$

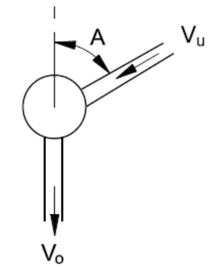
Where:

V = the flow velocity in the outlet pipe (ft/sec)

[34] ODOT Roadway Drainage Design Manual, November 2014, https://www.ok.gov/odot/Doing_Business/Pre-Construction_Design/Roadway_Design/Support_Units/Oklahoma_Roadway_Drainage_Manual.html

6.6.3.3.4.2. Bend Losses

The head loss due to a bend is:



$$H_b = K\ x \left(\frac{V^2}{2g}\right)$$

Where:

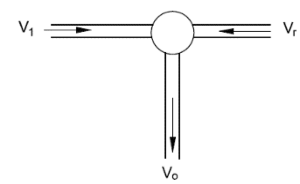
H_b = Head loss due to bend (ft)

V_u = Flow velocity in upstream inline pipe (fps)

K = A dimensionless coefficient as given in **Figure 2** (Figure 10.15-D in the ODOT Roadway Drainage Manual)

6.6.3.3.4.3. Opposite Lateral Losses (not including upstream in line or inlet flow)

The head loss due to opposite laterals is:



$$H_t = 1.0\ x \left(\frac{V^2}{2g}\right)$$

Where:

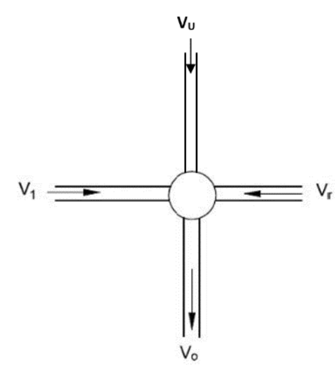
V = the flow velocity in the outlet pipe (ft/sec)

6.6.3.3.4.4. Junction Losses (with or without inlet flow)

The head losses due to the convergence of several storm sewer pipes in a manhole are:

$$H_j = \left(\frac{{V_o}^2}{2g}\right) - \left({}^{Q_u}/_{Q_o}\right)\left(\frac{V^2}{2g}\right) - (1 - K_1)\left({}^{Q_1}/_{Q_o}\right)\left(\frac{{V_1}^2}{2g}\right) - (1 - K_r)\left({}^{Q_r}/_{Q_o}\right)\left(\frac{{V_r}^2}{2g}\right)$$

Where:



Hj = Head loss in the manhole, ft
Vo = Flow velocity in outflow pipe, fps
Vu = Flow velocity in upstream inflow pipe, fps
Vr = Flow velocity in right lateral pipe, fps
V1 = Flow velocity in left lateral pipe, fps
K1 = Bend coefficient, left lateral – see **Figure 2**[35]
Kr = Bend coefficient, right lateral – see **Figure 2**

[35]Figure 10.15-D, ODOT Roadway Drainage Design Manual, November 2014, https://www.ok.gov/odot/Doing_Business/Pre-Construction_Design/Roadway_Design/Support_Units/Oklahoma_Roadway_Drainage_Manual.html

The total head losses in the manhole/Junction box for open channel flow are:

$$H = H_t + H_b + H_1 + H_j$$

Where H = Total head losses, ft

H_t = Terminal Manhole Loss

H_b = Bend Losses

H_1 = Opposite Lateral Losses

H_j = Manhole/Junction Losses

Figure 2 - Bend Loss (Open Channel Flow Design)

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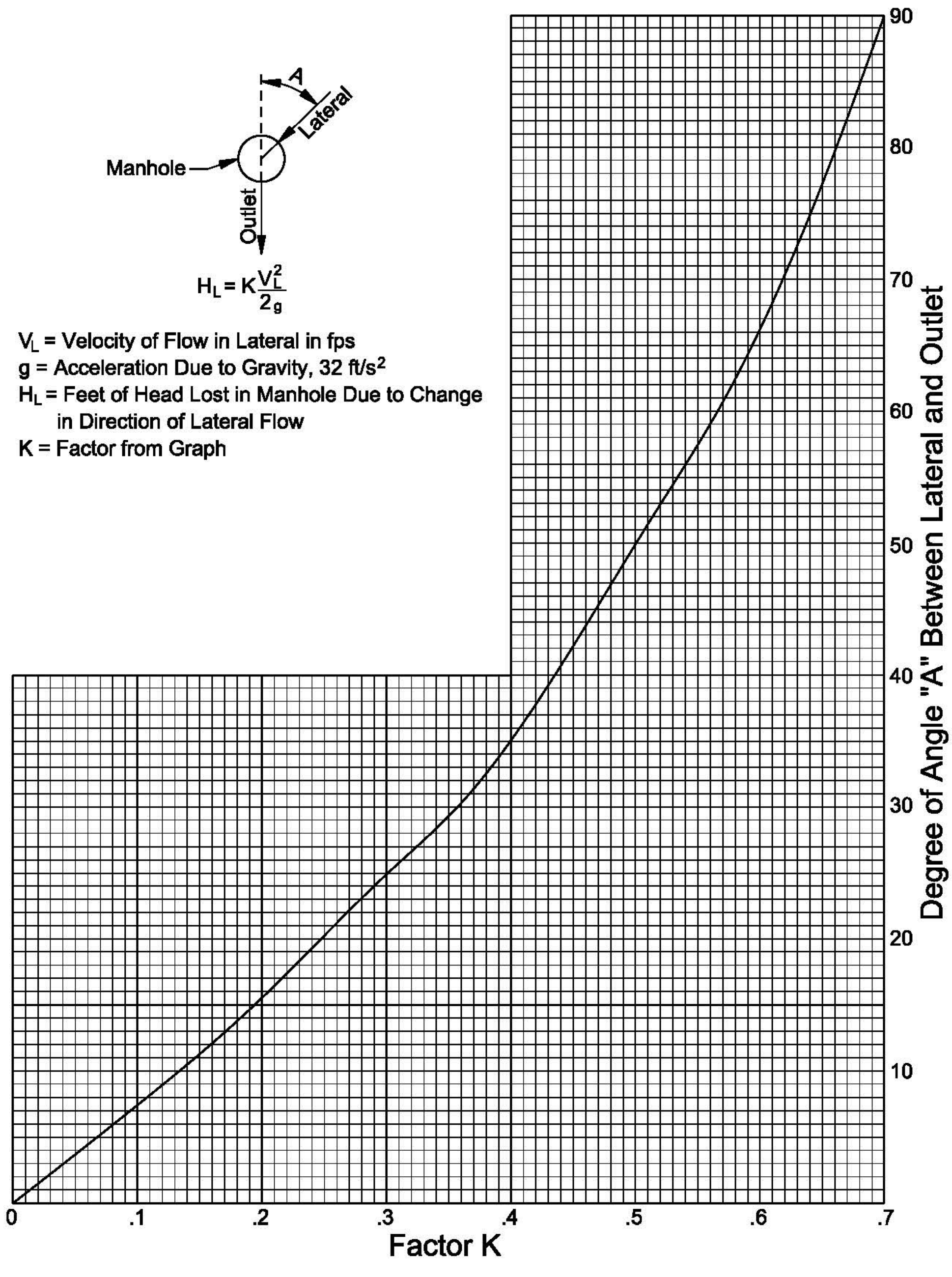


Figure 10.15-D — BEND LOSS – OPEN CHANNEL FLOW DESIGN

Stormwater Drainage 10.15-9

7. STORMWATER DETENTION

7.1 Introduction

7.1.1. The criteria presented in this section shall be used in the design and evaluation of all stormwater storage/detention facilities for the City of Moore. The review of all planning submittals will be based on the criteria presented in this Chapter. Technical Criteria applying to stormwater detention is found in Section 1.6.3.2.

7.1.2. An example of the stormwater quality benefits of dry stormwater detention is shown in Appendix A, Page A-5. Wet stormwater detention facilities are also allowable. Examples of the stormwater quality benefits of wet stormwater detention is shown in Appendix A, Page A-6, Retention Pond.

7.1.3. The main purpose of a detention facility is to store the stormwater runoff associated with increased watershed imperviousness due to development and to discharge this runoff at a rate similar to the runoff rate from the watershed without development. The City of Moore defines two types of detention: on-site and regional:

7.1.4. On-site detention is defined as a privately owned and generally privately maintained open space, parking lot, or underground facility which serves the development.

7.1.5. Regional detention is publicly owned and maintained and generally is part of a planned open space park system or greenbelt area in accordance with the Comprehensive Plan, serving a larger portion of the watershed.

7.1.6. Any dam that has a height of 25 feet or more from the natural streambed and/or 50 acre-feet or more of storage capacity, is under the jurisdiction of the Oklahoma Water Resources Board (OWRB). The OWRB also classifies dams as high-hazard, significant-hazard, and low-hazard, depending on the downstream populations and infrastructure. The hazards are based on first, potential for loss of life from a breach and secondly from the level of economic damage that will occur downstream from a breach.

7.1.7. All detention facilities shall be designed to meet OWRB spillway requirements for a High Hazard Dam for the size category under which they fall. Any lower hazard rating is raised to High Hazard status once a residence, business or roadway is constructed downstream within the dam breach inundation zone for the dam. The minimum spillway design shall pass the 0.2% Annual Chance (500-year) storm discharge with one foot of freeboard to the top of the dam for a dam 6 feet or higher.

7.2 General Design Criteria

7.2.1. General Design Criteria for all Stormwater Detention Facilities

7.2.1.1. The primary function of a detention facility is to reduce stormwater runoff from a development to the rate of runoff prior to the development.

7.2.1.2. All detention facilities shall be designed using the unit hydrograph method to determine the effects of the storage on the peak Annual Chance discharges for the 50% Annual Chance (2-year), 20% Annual Chance (5-year), 10% Annual Chance (10-year), 2% Annual Chance (50-year), and 1% Annual Chance (100-year) storm events.

7.2.1.3. HEC-HMS is the preferred hydrologic modeling programs, but other programs may be used with approval from the DIRECTOR.

7.2.1.4. Multi-stage outlet works are both acceptable and generally required to meet the requirements for no increase for any or the storm frequencies mention above in B. They are also encouraged because of the water quality benefits.

7.2.1.5. Outlet works should be designed with as large an opening as possible to still maintain the design objectives.

7.2.1.6. Pump systems to evacuate the detention facility during Flood events are not allowed.

7.2.1.7. Grass side slopes should be no steeper than 4:1.

7.2.1.8. For regional detention ponds the top of the embankment shall be 15 feet wide at a minimum.

7.2.1.9. For on-site detention ponds the top of the embankment shall be 15 feet wide at a minimum. Access to the outlet works shall be provided 7either via a 15' top width or other arrangements approved by the DIRECTOR.

7.2.1.10. Detention facilities may be either wet or dry depending upon multiple-use and water quality considerations.

7.2.1.11. Safety of the detention pond and outlet works shall be addressed in design. This includes embankment stability and the consequences of embankment failure.

7.2.1.12. Detention facilities shall be environmentally sound and compatible with the neighborhood and, where feasible, multi-use should be included.

7.2.2. General Design Submittals Required by the CITY for Review and Approval.

7.2.2.1. Inflow and outflow hydrographs.

7.2.2.2. A comparison of the pre-project and with-project peak discharges at the point(s) of discharge from the development and at points downstream as required by the DIRECTOR.

7.2.3. Elevation-Storage-Discharge relationships.

7.2.3.1. Discharge rating curves for each component of the outflow structure.

7.2.3.2. Tailwater rating curves at the outlet. Tailwater shall be considered when designing the outlet structure.

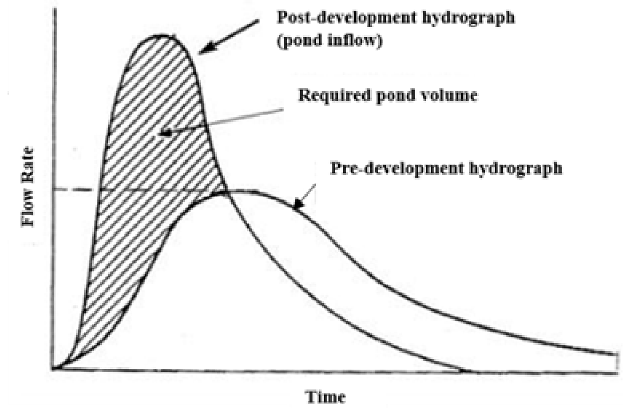
7.2.4. Erosion protection measures at the outlets and spillway.

7.2.4.1. Embankment design in accordance with OWRB guidelines, including slope protection in case of overtopping, slope stability, and maintenance access.

7.2.4.2. Trash rack design.

7.3 On-Site Stormwater Detention Requirements

7.3.1. The effectiveness of an on-site detention facility in controlling the peak discharges at downstream points is highly dependent on the location of the facility in the overall drainage basin. Studies have shown that on-site detention may actually increase downstream peak discharges by delaying the hydrograph from the development and causing it to combine with downstream hydrographs into a larger flood peak. For this reason, all on-site detention designs shall consider the effects on peak discharges downstream as required by the DIRECTOR.



7.3.2. On-site detention facilities are recognized by the CITY to be effective in controlling the peak discharges from the development immediately downstream of the development, and as a valuable part of a Low Impact Development.

7.3.3. The stormwater detention facility must contain the volume of the post-development hydrograph up to the point on the falling limb where the post-development flowrate equals the pre-development flowrate. The rising side of the post-development hydrograph after routing through the stormwater detention facility must not occur sooner than the rising side of the pre-development hydrograph.

7.3.4. On-site detention facilities shall be designed so that there is no increase in the peak discharge created by the development for the 50% Annual Chance (2-year), 20% Annual Chance (5-year), 10% Annual Chance (10-year), 2% Annual Chance (50-year), and 1% Annual Chance (100-year) flood events. This applies at the point(s) of discharge from the development as well as at points downstream of the development that may be impacted, at the required discretion of the DIRECTOR.

7.3.5. The Oklahoma Water Resources Board may have jurisdiction over the design of onsite detention facilities, depending on the dam height and/or storage volume. OWRB High Hazard dam guidelines shall be followed for all embankment facilities.

7.3.6. The erosive effects of the increased runoff volume from the on-site detention facility shall be mitigated by armoring the stream bank downstream.

7.4 Regional Stormwater Detention

7.4.1. General Requirements

7.4.1.1. Regional detention is part of the CITY's comprehensive plan to mitigate increased peak discharges due to upstream urbanization. Regional detention facilities are generally owned and maintained by the CITY and are part of the Master

Drainage Planning for each major watershed within the CITY that may also include channelization, improved bridges and culverts, and non-structural measures. Meant to serve a large portion of the watershed, regional detention facilities regulate the inflow to provide peak discharge reductions downstream and work in combination with the other features of the Master Drainage Plan. When designing regional detention, the following factors must be considered:

7.4.1.1.1. Regional detention facilities shall discharge into a 1% Annual Chance (100-year) conveyance system (improved channel, storm sewer system, or natural channel with adequate overbank conveyance and regulation).

7.4.1.1.2. The Oklahoma Water Resources Board may have jurisdiction over the design of regional detention facilities, depending on the dam height and/or storage volume. OWRB High Hazard dam guidelines shall be followed for all embankment facilities.

7.4.1.1.3. Regional detention facilities shall be designed so that there is no increase in the peak Annual Chance discharge for the 50% Annual Chance (2-year), 20% Annual Chance (5-year), 10% Annual Chance (10-year), 2% Annual Chance (50-year), and Regulatory Storm events, and the Regulatory Storm discharge will produce an elevation at least two feet below the finished floor elevation of any building.

7.4.1.1.4. Water quality considerations of the urban runoff leaving the on-site detention facility shall be considered and implemented when feasible in accordance with the Comprehensive Plan.

7.5 Design Standards for Open Space Detention

7.5.1. Oklahoma Water Resources Board

7.5.1.1. All detention facilities shall be designed to meet OWRB requirements or the requirements set forth in this Chapter, whichever is stricter in terms of dam safety.

7.5.2. Grading

7.5.2.1. Grass slopes on earthen embankments shall be 4:1 unless otherwise permitted by the DIRECTOR. Rip rap covered embankments shall not be steeper than 2:1. The minimum bottom slope in grassed detention facilities shall be 2.0 percent measured perpendicular to the trickle channel.

7.5.3. Freeboard

7.5.3.1. The embankment elevation shall provide two feet of freeboard as described in **Table 2 -** Freeboard Requirements for Stormwater Detention Facilities.

7.5.4. Trickle Channel

7.5.4.1. All grassed bottom detention ponds shall include a concrete trickle channel or equivalent performing materials design. Longitudinal slopes shall be no less than 0.5%.

7.5.5. Outlet Configuration

7.5.5.1. The outlet shall be designed to provide discharges from the pond that are equal to or less than pre-development Annual Chance discharges for the 50% Annual Chance (2-year), 20% Annual Chance (5-year), 10% Annual Chance (10-year), 2% Annual Chance (50-year), and 1% Annual Chance (100-year) storm events. Orifice or slotted weir configurations should be as large as possible to meet the design requirements. An emergency spillway shall be provided to pass the difference between the 0.2% Annual Chance (500-year) storm discharge and the 0.1% Annual Chance (100-year) storm discharge with the required freeboard. The crest elevation of the spillway shall be at or above the 1% Annual Chance (100-year) flood elevation in the facility.

7.5.6. Embankment Protection

7.5.6.1. Whenever a detention pond uses an embankment to contain water, the embankment, spillway crest, and spillway apron shall be protected from catastrophic failure due to overtopping.

7.5.7. Vegetation

7.5.7.1. All open space detention areas shall be planted with permanent Bermuda sod, native dry-land grasses, or other native plants that are compatible with the multi-use plan and irrigated as required until established.

7.5.8. Maintenance Access

7.5.8.1. All open space detention areas shall be bounded by an easement or dedicated ROW for the purposes of obtaining access from a public ROW and for maintenance activities. Maintenance access ramps shall be provided and constructed with a drivable slope no steeper than 10%. Porous type driving surface material is preferred.

7.6 Design Standards for Parking Lot Detention

7.6.1. Depth

7.6.1.1. The maximum allowable design depth of the ponding is 18-inches for the 1% Annual Chance (100-year) flood and 9-inches for the 20% Annual Chance (5-year) flood. In areas where the probability of significant vehicular damage is high the 1% Annual Chance (100-year) depth shall be limited to 12-inches.

7.6.2. Outlet Configuration

7.6.2.1. The minimum pipe size for the outlet is 18" diameter where a drop inlet is used to discharge to a storm sewer or drainage way. Where a weir or a small dimension outlet through a curb is used, the size and shape are dependent on the discharge/storage requirements.

7.6.3. Performance

7.6.3.1. HEC-HMS (or other approved hydrologic modelling software) may be used to design the parking lot detention so that the 20% Annual Chance (5-year) flood depth is no greater than 9-inches and the 1% Annual Chance (100-year) flood depth is no greater than 18-inches. The outlet shall be designed so that there is no increase in off-site peak discharge.

7.6.3.2. To assure that the detention facility performs as designed, maintenance access shall be provided. The outlet shall be designed to minimize unauthorized modifications which effect function. Any repaving of the parking lot shall be evaluated for impact on volume and release rates and are subject to approval by the DIRECTOR prior to issuance.

7.6.4. Flood Hazard Warning

7.6.4.1. All parking lot detention areas shall have a minimum of two signs posted identifying the detention pond area. The signs shall have a minimum area of 1.5 square feet and contain the following message:

"WARNING"
"This area is a stormwater detention pond and
is subject to periodic flooding to a depth of “x”
during a 1% Annual Chance (100-year) flood."

7.6.4.2. Any suitable materials and geometry of the sign are permissible, subject to approval by the DIRECTOR.

7.7 Required Design Standards for Underground Detention

7.7.1. Example

7.7.1.1. See **Appendix A, Page A-6** for an example of underground stormwater detention.

7.7.2. Materials

7.7.2.1. Underground detention shall be constructed using reinforced concrete pipe, reinforced concrete box culvert, concrete vaults, or other material as approved by the DIRECTOR. The material thickness, cover, bedding, and backfill shall be designed to withstand HS-20 loading.

7.7.3. Configuration

7.7.3.1. Pipe (storage) segments shall be sufficient in number, area, and length to provide the required minimum storage volume for the 1% Annual Chance (100-year) design. As an option, the 10% Annual Chance (10-year) design can be stored in the pipe segments and the difference for the 1% Annual Chance (100-year) stored above the pipe in an open space detention or in a parking lot detention. The minimum diameter of the pipe segments shall be 36 inches.

7.7.3.2. The pipe segments shall be placed side by side and connected at both ends by a manifold system or an approved alternative. The pipe segments shall be continuously sloped at a minimum of 0.25% to the outlet. Maintenance access points shall be provided on each line segment and near the outlet works to facilitate system cleanout.

7.7.3.3. Permanent buildings or structures shall not be placed above the underground detention.

7.7.4. Inlet and Outlet Works

7.7.4.1. The outlet from the detention shall consist of a short (maximum 25 ft. lengths) of RCP with an 18" minimum diameter. A two-pipe outlet may be required to control all design frequencies. The invert of the lowest outlet pipe shall be set at the lowest point in the detention pipes. The outlet pipe(s) shall discharge into a standard manhole or into a drainage way with erosion protection provided. If an orifice plate is required to control the release rates, the plate(s) shall be hinged to open into the detention pipes to facilitate back flushing of the outlet pipe(s).

7.7.4.2. Inlet to the detention pipes can be by way of surface inlets and/or by a local private storm sewer system.

7.7.5. Maintenance Access

7.7.5.1. Access easements to the detention site shall be provided. To facilitate cleaning of the pipe segments, 30-inch diameter maintenance access ports shall be placed according to the **Table 14**:

Table 14 - Maintenance Access Requirements

DETENTION PIPE SIZE (INCHES)	MAXIMUM SPACING (FT)	MINIMUM FREQUENCY
36 to 54	150	Every pipe segment
60 to 66	200	Every other pipe segment
> 66	200	One at each end of the battery of pipes

Cleanouts or inspection ports are not acceptable access points for maintenance and inspection nor are any other alternatives which do not allow for full entry into the system.

8. LOW IMPACT DEVELOPMENT (LID) METHODS FOR IMPROVING SOIL PERMEABILITY

8.1 Recommended LID Techniques to be Allowed by Right

8.1.1. The City allows the following LID techniques to be used in any Zoning District:

8.1.1.1. Bioswales

8.1.1.2. Filter Strips

8.1.1.3. Pervious Paving for non-public improvements only.

8.1.1.4. Wet Ponds

8.1.2. These are discussed below. Further information and resources on these and other LID techniques can be found at http://www.bwdh2o.org/wp-content/uploads/2012/03/Low_Impact_Development_Manual-2010.pdf.

8.1.2.1. Bioswale (also called Biofiltration Swale) – See **Appendix Page A-11**.

8.1.2.1.1. These are shallow landscape features that store, filter, and either retain or detain stormwater. These systems utilize biological and physical processes to remove pollutants from stormwater and mimic predevelopment hydrology. They are typically designed to treat small catchment areas that have high percentages of impervious surface.

8.1.2.1.2. Biofiltration features can be cells, swales, or planters. Bioretention cells are geometrically variable, but typically oval, and the surface is level. A swale has a channel-like flow path and may utilize grade control structures to increase surface storage and infiltration while reducing the potential for erosion. A planter is an engineered structure, typically installed in ultra-urban areas where open landscapes are not available or practical.

8.1.2.1.3. Several conditions must be considered:

8.1.2.1.3.1. A separation distance of 2 feet is required between the bottom of the bioretention practice and the elevation of the seasonally high water table (saturated soil) or top of bedrock (i.e., there must be a minimum of 2 feet of undisturbed soil beneath the infiltration practice and the seasonally high water table or top of bedrock).

8.1.2.1.3.2. An underdrain with ¾" clean gravel will be placed at the bottom of the trench,

8.1.2.1.3.3. Non-woven geotextile fabric will cover the sides and bottom of the trench above the filter,

8.1.2.1.3.4. Engineered soil will be placed with the fabric to allow a percolation rate of at least 0.3"/hour,

8.1.2.1.3.5. A 3" mulch layer will be placed over the engineered soil mix,

8.1.2.1.3.6. Maximum Ponding Depth from top of mulch is 18 inches/1.5 feet, at which level an overflow system will be designed

8.1.2.1.3.7. Maximum Drain Time for the 18-inch ponding area is 48 hours.

8.1.2.1.4. The size of the bioretention swale is generally given below[36]:

8.1.2.1.4.1. The water quality treatment volume is equal to the amount of impervious cover area draining to the bioretention site x one inch of runoff; convert to cubic feet. This will treat the first flush (1 inch of runoff) from the impervious areas.

8.1.2.1.4.2. The ponded depth is equal to 48 hours times the infiltration rate (at least 0.3 inches per hour) in feet and must not exceed 1.5 feet.

8.1.2.1.4.3. The surface area of the bioswale is equal to the water quality treatment volume divided by the treatment depth or 1.5 feet, whichever is smaller, adjusting for side slopes.

8.1.2.1.4.4. Any additional required stormwater detention may be reduced by the water quality treatment volume.

8.1.2.2. Filter Strips – See Appendix Page A-8

8.1.2.2.1. Vegetated Filter Strips (VFS) are densely vegetated areas often sandwiched between sources of nonpoint source pollution and receiving water bodies or the stormwater conveyance systems and used as an LID stormwater pre-treatment practice.

8.1.2.2.2. Filter strips can be placed upstream from a stormwater detention facility, a bioswale or a creek channel. They provide erosion control by spreading out concentration flow and are used in conjunction with curb cuts and level spreaders to convert concentrated flow into sheet flow.

8.1.2.2.3. Installing filter strips instead of impervious surfaces will reduce the overall stormwater quality treatment volume as well as the required stormwater detention volume.

8.1.2.2.4. Required maintenance includes:

8.1.2.2.4.1. Accumulated sediment should be removed from the filter strip and level spreader or curb cut.

8.1.2.2.4.2. Vegetation should be maintained at 4-6"

8.1.2.2.4.3. Debris and trash removal are required.

8.1.2.3. Pervious Pavement – See **Appendix Page A-9**

8.1.2.3.1. Pervious pavement is an LID practice that has been shown to provide environmental benefits while maintaining the structural surface in many instances. Parking lots, streets, alleyways, and sidewalks are a few examples of applications where PP have been successfully installed. This technique is particularly useful in highly urban setting where the impervious surface may cover 75% to 100% of the total area, leaving limited open space available to other LID practices.

[36] https://stormwater.pca.state.mn.us/index.php?title=Design_criteria_for_bioretention

8.1.2.3.2. Some types of Pervious Pavements

8.1.2.3.2.1. **Porous asphalt**. Like regular asphalt, porous asphalt is composed of aggregate and asphalt binder, but has a limited number of fines. The use of additives and higher quality binder are often utilized in porous asphalt systems.

8.1.2.3.2.2. **Pervious concrete**. Pervious concrete is typically made of aggregate, Portland cement, little to no fines, and additives. Additives may be included in the mix to improve strength, regulate curing rate, and increase workability.

8.1.2.3.2.3. **Permeable pavement block systems**. This includes a suit of different impervious blocks that are installed over a bedding material and aggregate storage base. Permeable interlocking concrete pavers (PICP) are impervious pavers that install in an interlocking configuration. Interlocking the paver greatly reduces paver movement under normal conditions. Permeable pavers are similar to PICP, though they are free setting on a bedding layer with aggregate filled joints to prevent movement. These also include permeable pavers that physically connect with cables or similar systems. PICP and permeable pavers should not be installed with a sand bedding layer or have sand filled pores. Typically, a #8 or similar coarse graded aggregate is appropriate for the bedding and joint material. Consult manufactures specification for detailed design guidance.

8.1.2.3.2.4. **Grid pavement systems**. These are structural grids that are on top of a bedding layer that have space within the grid for permeable material. Aggregate and turf grass commonly fill the spaces within the grids system. If turf grass is used, it should be maintained to a height at or below 3 inches. Aggregates should and must be replaced as needed.

8.1.2.3.3. Design for SWQV

8.1.2.3.3.1. The water quality treatment volume is equal to the amount of impervious cover area draining to the pervious pavement site x one inch of runoff; convert to cubic feet.

8.1.2.3.3.2. The open graded subbase for each of these systems is the location of the storage of SWQV required. A non-woven geotextile must be installed between the aggregate base and the in-situ soil. The non-woven geotextile should separate the aggregate on the bottom and the sides. The non-woven geotextile should be a minimum of 15 oz. fabric.

8.1.2.3.3.3. For the storage depth, the infiltration rate of the native soil must be known, requiring testing before design and construction. The depth of storage d_s is calculated as:

$$d_s(\text{in}) = \frac{\left(I\ \frac{\text{in}}{\text{hr}}\right)\ (48\ \text{hrs})}{\phi_S}$$

Where I = infiltration rate of the native soil
48 = the drawdown time (hours)
ϕ_S = porosity of the subbase storage material.

Or in other words, for a 40% porosity, the storage depth will be 2-1/2 times the depth of open space, above-ground detention.

8.1.2.3.3.4. The area of pervious pavement required to contain the SWQV is then computed as the required volume divided by the depth x the porosity.

8.1.2.3.3.5. The interface between pervious and non-pervious pavement must be sealed with non-pervious geotextile factor in order to prevent damage to non-pervious pavement in accordance.

8.1.2.3.3.6. Pervious pavement design is outside the scope of this document.

8.1.2.4. Wet or Retention Ponds – See **Appendix Page A-7**.

8.1.2.4.1. Wet ponds can be used for stormwater quantity control in conditions where stormwater detention is a requirement, by adding the required storage above the permanent as a "flood control" pool. This has several advantages:

8.1.2.4.1.1. As stormwater enters the facility, it encounters a flat water surface. There is no storage volume loss because of the "wedge" created by sloping the ground towards the outlet. Generally, the total rise in water surface elevation in the pond will be less than in a dry pond.

8.1.2.4.1.2. The desirable aquatic processes, assuming healthy vegetation is maintained, will provide some biological treatment for stormwater quality.

8.1.2.4.1.3. Outlet structure can be designed to prevent debris clogged more easily that with an open pipe outlet.

8.1.2.4.2. There are a number of factors to consider that are necessary for the health of the pond, including:

8.1.2.4.2.1. If the pond is located on highly permeable soils, the permanent pool may need to be supplemented or clay liner may be necessary.

8.1.2.4.2.2. A yield study should be performed to determine if the drainage basin is large enough to sustain the permanent pool through potential low-precipitation periods, again potentially requiring supplemental water.

8.1.2.4.2.3. Temperature and wind variations may impact the aerobic environment adversely as well as foster mosquitos and other pests. Aeration should be planned for the pond, either with surface aerators or compressed air features in the bottom of the pond.

8.1.2.4.2.4. Generally, a minimum depth for the ponds is 4 feet

9. CONSTRUCTION PHASE STORMWATER POLLUTION PREVENTION

9.1 Construction Phase Storm Water Pollution Prevention

9.1.1. Regulatory Basis

9.1.1.1. For construction sites, the Oklahoma Department of Environmental Quality (ODEQ) permitting process is conducted according to the rules established under the Oklahoma Pollution Discharge Elimination System (OPDES), as promulgated under Oklahoma Administrative Code (OAC) 252:605. Specifically, ODEQ regulates discharges associated with construction activities. Under Phase II, construction activities that disturb or plan to disturb 1 or more acres must obtain a General Permit for Construction Activities within the State of Oklahoma. This permit was issued to effect compliance with the Phase II Storm Water regulations issued December 8, 1999.

9.1.2. Activities Requiring a Stormwater Control Measures

9.1.2.1. Construction site operators for construction that will disturb soil of any area are required to implement and maintain site-specific storm water Best Management Practices (BMPs) to address erosion and sediment control for the duration of the project.

9.1.2.2. Construction disturbing greater than 1.0 acre of land, including disturbances less than 1.0 acres but within a planned development totaling greater than 1.0 acre, requires coverage under the OKR10 for Construction and must also have a Storm Water Pollution Prevention Plan (SWP3). For complete information on the OKR10 and to access the Addendums referred to in these sections, please refer to the ODEQ State Permit OKR10.

9.1.2.3. A copy of the SWP3 is required as part of land development submittals to the CITY. The CITY will review and approve SWP3s as part of the plan review process.

9.1.3. Timing of BMPs

9.1.3.1. Erosion and sediment control BMPs shall be initiated prior to construction soil disturbance and maintained throughout the construction process. Prior to soil disturbance, a construction site must pass a pre-construction erosion control (PCEC) inspection conducted by the CITY.

9.1.4. Activities to be Completed

9.1.4.1. Steps to be completed under OKR10 for a Storm Water Permit for Construction Activities include:

A. Preparation of a Storm Water Pollution Prevention Plan

B. Filing a Notice of Intent with ODEQ

C. Payment of Permit Fees

D. Completion of Best Management Practices and Inspections

E. Update SWP3 as necessary

F. Filing a Notice of Termination

9.1.5. Storm Water Pollution Prevention Plan (SWP3)

9.1.5.1. The initial step in obtaining a Storm Water Permit for Construction Activities is to develop a SWP3 according to ODEQ requirements. In general, the SWP3 will:

A. Identify potential sources of pollution that may reasonably be expected to affect the quality of storm water discharges from the construction site.

B. Describe Best Management Practices (BMPs) that will be used to divert flows from exposed soils, store flows, or otherwise limit runoff and the discharge of pollutants from exposed areas of the site, as well as non-structural practices that mitigate erosion and sediment movement.

C. Describe a program to assure compliance with the terms and conditions of your permit through monitoring and inspections.

9.1.5.2. A copy of the complete SWP3 must be submitted to the ODEQ for review if:

A. Any area of the construction site is located within the watershed of an Outstanding Resource Water (see Addendum F to OKR10). There are no Outstanding Resource Waters in the City of Moore.

B. Any area of the construction site is located within a sensitive water and watershed (see Addendum A to OKR10).

C. The area to be disturbed on the construction site is forty (40) acres or more.

9.1.5.3. A copy of the SWP3 and all related documents must be maintained on site at all times and updated as necessary during construction. This SWP3 and all associated documents must be made available to the CITY upon request.

9.1.5.4. If the permittee is disturbing 1 acre of greater, a land disturbance permit is required. In order to receive a Land Disturbance Permit, a copy of the SWP3 along with an executable NOI must be submitted to the CITY for review.

9.1.5.5. Steps to be completed under the CITY's Pollution Ordinance include:

A. When the site is > 1 acres, ensure the site has coverage under an OKR10.

B. Adequate BMPs are in place and maintained throughout the life of the project. Notice of Intent (NOI).

9.1.5.6. Prior to initiation of construction activities requiring an OKR10, the owner/operator must file a NOI with the ODEQ. An online NOI from ODEQ is available in Addendum B of OKR10. The NOI must be filed and authorization received from ODEQ prior to discharge of storm water from the construction activities. As stated in 1002.4.1, a copy of the NOI must be submitted to the CITY in order to Notice of Termination (NOT).

9.1.5.7. The OKR10 permit will remain in effect until a NOT is filed with the ODEQ and the CITY. A NOT must be filed within thirty (30) days of the following events:

A. Final stabilization has been achieved on all portions of the site for which the permittee is responsible.

B. At residential sites: temporary stabilization has been completed and the residence has been transferred to the homeowner.

C. When another owner/operator has assumed control over all areas of the site that have not been finally stabilized. The NOT must be submitted with the new owner/operator's NOI.

9.1.6. Inspections

9.1.6.1. BMPs shall be inspected every 14 days or within 24 hours of a precipitation event greater than ½" inch by the construction site operator. If any visible sediment is observed to be leaving the site within 48 hours, any corrective actions taken must be documented in the SWP3. Maintenance needs for BMPs, including replacement or sediment removal, shall also be assessed during inspections.

9.1.7. Best Management Practices for Construction Activities

9.1.7.1. Best Management Practices (BMPs) include structural and non-structural methods to prevent erosion and sediment from leaving the construction site through storm water runoff, tracking or wind-dispersion. A wide variety of BMPs are available for use on construction sites. A partial BMP list can be found in the City of Moore's Construction Stormwater/ Land Disturbance Permit Application, Table 2: Stabilization Practices and Other Pollution Controls.[37]

9.1.7.2. It is the responsibility of the construction site operator to select, implement and maintain the proper BMPs for the site. Hydrologic and hydraulic analysis methodology used to size construction phase BMPs shall be consistent with guidance provided in other sections of this MANUAL. Sizing and selection of BMPs shall also

[37] https://www.cityofmoore.com/uploads/Environmental-Services/SWP3-New-Commercial.pdf

be consistent with guidance provided in OKR10 and ensure compliance with Article D. – Stormwater Quality Management, of the City's Land Development Code.[38]

9.1.7.3. The list below is a summary of BMP's commonly used on construction sites:

A. Minimizing Disturbed Area. Minimize the amount of land stripped of vegetation and graded to reduce erosion. Undisturbed lands have their own natural soil erosion retardance that disturbed soils do not. Staggering of construction areas is one of the more effective ways of keeping erosion from occurring, or at least reducing it significantly.

B. Controlling Erosion. Permanent or temporary soil surface stabilization must be applied to all disturbed areas and soil stockpiles as soon as possible but no later than 14 days after final grade is reached on any portion of the site. Temporary soil surface stabilization should also be applied as soon as possible, but no later than 14 days after disturbance, to disturbed areas that may not be at final grade but will remain unused temporarily. Numerous erosion control products are available. Selection is based on soil types, slopes, and areas of disturbance.

C. Temporary Revegetation. Temporary revegetation is required on all disturbed areas as soon as practicable, at most within 14 days, if the area is to remain dormant, or absent of further disturbance, and before final stabilization takes place. All temporary seeding shall be properly mulched.

D. Revegetation. Vegetation is not considered established until a ground cover is achieved which is equivalent to at least 70 percent of the pre-existing vegetation and is sufficiently mature to control soil erosion and can survive severe weather conditions.

E. Providing Surface Roughening. Surface roughening may be performed after final grading to create depressions two to four inches deep and four to six inches apart.

F. Stabilizing Roads and Soil Stockpiles. Road cuts, road fills and parking lot areas should be covered as early as possible with the appropriate aggregate base course where this is specified as part of the pavement. Seed and mulch or otherwise

38 https://library.municode.com/ok/moore/codes/land_development_code?nodeId=PT12LADECO_CH14STMA_ARTDSTQUMA

stabilize using soil binders all non-paved portions of roads as soon as possible after final grading has occurred, but in no case later than 14 days after grading has been completed.

Seed and mulch soil stockpiles within 14 days after completion of stockpile establishment. Mulch without seeding is acceptable if expected to be in place 30 to 60 days. Seeding should be used if stockpile will be in place >60 days. If stockpiles are located within 100 feet of a waterway, additional sediment controls, such as diversion dikes or silt fences should be provided.

G. Minimizing Vehicle Tracking. Whenever construction vehicles enter onto paved roads, provisions must be made to prevent the transport of sediment (mud and dirt) by vehicles tracking onto the paved surface. Whenever sediment is transported onto a public road, regardless of the size of the site, at a minimum the roads shall be cleaned at the end of each day.

H. Providing Slope Diversion Dikes. Diversion dikes located above disturbed areas may be discharged to a permanent or temporary channel. Diversion dikes located mid-slope on a disturbed area must discharge through a sediment trap or basin to a temporary drain. Diversion dikes located at the base of a disturbed area must discharge to a sediment trap or basin.

I. Trapping Sediment. Sediment entrapment facilities include silt fences, wattles, silt dikes, silt socks, rock check dams, curb socks, and sediment basins/traps. Per OKR10, no straw bales will be allowed as BMP's. All runoff leaving a disturbed area shall pass through a sufficient number of sediment entrapment facilities to prevent sediment discharge from the site. The spacing and design of such devices is a function of device type, drainage areas, flow path lengths, soil types, and slopes.

J. Working Within or Crossing a Waterway. Construction vehicles shall be kept out of waterways to the maximum extent practicable. Where an actively flowing watercourse must be crossed regularly by construction vehicles, a temporary stream crossing and/or channel diversion must be provided.

K. Protecting Outlets. The outlets of temporary slope drains, culverts, sediment traps and sediment basins must be protected from erosion and scour.

L. Protecting Inlets. All storm sewer inlets made operable during construction must have sediment entrapment facilities installed to prevent sediment-laden runoff from entering the inlet. All storm sewer inlets receiving inflow from disturbed site areas be protected from soil entry during construction.

M. Properly Storing Chemicals, Oils and Other Materials. Areas used for staging of construction activities and the storage of chemicals, petroleum-based products and waste materials, including solid and liquid waste, shall be designed to prevent discharge of pollutants in the runoff from a construction site.

N. Prevent Air-borne Dispersal. Air-borne dispersal of soil prior to stabilization should be prevented by wetting with water or other methods.

O. Disposal of Temporary Measures. All temporary erosion and sediment control measures shall be removed within 30 days after final stabilization is achieved, or after the temporary measures are no longer needed, whichever occurs earliest, or as authorized by the municipality or other local jurisdiction.

P. Maintaining BMPs. All temporary and permanent erosion and sediment control practices shall be maintained and repaired by the Permit Holder or Owner during the construction phase as needed to ensure continued performance of their intended function. All facilities must be inspected and replaced if necessary, following each precipitation or snowmelt event that results in runoff in excess of 0.5 inches.

9.1.8. Selecting Storm Water Controls

9.1.8.1. The following guidelines are recommended in developing the site BMP’s:

A. Define the layout of buildings and roads. This will have been decided previously as a part of the general development plan. If building layout is not final, the road areas stabilized with pavement and the drainage features related to roads should be defined as they relate to the plan.

B. Determine the limits of clearing and grading. If the entire site will not undergo excavation and grading, or excavation and grading will occur in stages, the boundaries of each cut-and-fill operation should be defined. Buffer strips of natural vegetation may be utilized as a control measure, if effective.

C. Determine on-site drainage areas. The size of drainage catchments will determine the types of sediment controls to be used. Areas located off the site that contribute overland flow runoff to the site must also be addressed. Measures to limit the size of upland overland flow areas, such as diversion dikes, may be initially considered at this stage.

D. Determine extent of temporary channel diversions. If permanent channel improvements are a part of the plan, the route, sizing and lining needed for temporary channel diversions should be determined. Location and type of temporary channel crossings can be assessed.

E. Determine permanent drainage features. The location of permanent channels, storm sewers, roadside swales and post-construction storm water controls such as ponds, wetlands, bioswales, or rain gardens, if known, should be defined.

F. Select erosion controls. All areas of exposed soil will require a control measure be defined dependent on the duration of exposure. These can be selected based on the schedule of construction.

G. Select sediment controls. Select the controls needed for each phase of the construction project. Each phase will have different demands for the control of erosion and sedimentation. For example, overlot grading will require controls that may be of little use when individual homes are being built and each lot is being disturbed after the streets and drainage systems are in place.

H. Determine staging of construction. The schedule of construction will determine what areas must be disturbed at various stages throughout the development plan. The opportunity for staging cut-and-fill operations to minimize the period of exposure of soils needs to be assessed and then incorporated into the final SWP3, at which time the initial sequence for installing sediment controls and erosion controls is defined.

I. Identify locations of topsoil stockpiles. Areas for storing topsoil should be determined and then proper measures to control their erosion and sediment movement off these sites specified.

J. Identify location of temporary construction roads, vehicle tracking controls, and material storage areas. These three elements can be determined in the context of previously defined parts of the site construction management plan.

10. GLOSSARY

10.1 Definitions

Apron: A floor or lining of concrete, or other suitable material at the upstream or downstream end of a Reinforced Box Culvert or Reinforced Concrete Pipe, at the discharge side of a spillway, a chute, or other discharge structure, to protect the waterway from erosion, from falling water or turbulent flow.

Backfill: (1) The operation of filling an excavation after it has once been made, usually after some structure has been placed therein and (2) the material placed in an excavation in the process of backfilling.

Backwater: The water retarded above a dam, bridge, or culvert or backed up into a tributary by a flood in the main stream. In this Manual, backwater is also defined as the rise in the flood water surface due to the restrictions created by the construction of a bridge or culvert.

Backwater Profile: The term applied to the longitudinal profile of the water surface in an open channel.

Berm: A horizontal strip or shelf built into an embankment or cut, to break the continuity of an otherwise long slope, usually for the purpose of reducing erosion, improving stability, or to increase the thickness or width of cross section of an embankment.

Bridge: A hydraulic structure that is constructed with abutments and superstructures which are typically concrete, steel, or other materials, or a culvert or culverts with a clear opening of 20 feet in width or more. Bridges are generally constructed with earth or rock inverts. Since the superstructures are not an integral part of the abutments and could therefore potentially move, the hydraulic criteria for bridges are different than for culverts.

Box Culvert: Generally, a rectangular or square concrete structure for carrying large amounts of water under a roadway. This term is sometimes applied to long underground conduits.

Bypass Flow (or Carry-Over Flow): The quantity of water which continues past an inlet.

Building Pad Elevation: The final elevation of a prepared ground surface that will provide structural support for the lowest concrete floor of a structure that is not more than 1 foot below the required minimum finished floor elevation.

Channel: A natural or artificial watercourse of perceptible extent which periodically or continuously contains moving water, or which forms a connecting link between two bodies of water. It has a definite bed and banks which serve to confine the water. Also see Watercourse.

Channel Storage: The volume of water stored in a channel during a rainfall event. Generally considered in the attenuation of the peak of a flood hydrograph moving downstream.

Check Drop: A structure of concrete, rock or other materials used to flatten the grade of a channel to reduce erosion tendencies of the flow.

Coefficient of Roughness "n": A factor in the Manning formula, for computing the average velocity of flow of water in a watercourse or conduit, which represents the effect of roughness of the confining material of the watercourse or conduit upon the energy losses in the flowing water.

Course (of a stream): The path taken for passage of water.

Critical Depth: The particular depth of flow in an open channel with a given discharge at which the specific energy is at a minimum, i.e., the depth at which a given discharge flows in a given channel with a minimum specific energy. The given discharge may flow at an alternate depth above or below critical in the given channel but the specific energy of the flow at either alternate depth will be greater than for the flow at critical depth.

Critical Flow: Flow at critical depth.

Culvert: A closed conduit for the passage of water under an embankment, such as a road, railroad or trail. A culvert is distinguished from a storm sewer in the following manner: flow generally enters a culvert by an open channel, generally at a similar elevation, while flow generally enters a storm sewer by means of storm inlets above the sewer; the geometry of the culvert inlet plays a major role in determining the required size or capacity of the culvert, whereas the capacity of a storm sewer is generally determined by the slope of the sewer; a culvert generally crosses under a road, railroad or trail, while a storm sewer generally follows the street alignment.

Dam: A barrier constructed across a watercourse for the purpose of creating a temporary or permanent reservoir.

Datum: A plane, level or line from which heights and depths are calculated or measured.

Debris Basin: A basin formed behind a low dam, or an excavation in a stream channel, to trap debris or bed load carried by a stream. The value of a basin depends on cleaning-out of debris periodically to restore its capacity.

Department: The City of Moore Community Development Department.

Stormwater Detention: A temporary storage of a determined quantity of storm water runoff for a specified period of time with a release rate that is either fixed or variable, the purpose being to attenuate the peak of the inflow hydrograph.

Developer: Any person, persons, corporation, or other entity who in his or her own behalf, or as an agent of another, engages in development, subdivision, construction of structures, or alteration of land in preparation therefore.

Development: Any man-made change to improved or unimproved real estate, including but not limited to buildings or other structures, mining, dredging, filling, grading, paving, berming, diking, excavating, or drilling operations.

DIRECTOR: For the purposes of this MANUAL the term DIRECTOR shall mean "Community Development Director or His/Her Designee”. It shall be the duty of the Community Development Director or his or her designee to enforce the provisions of this MANUAL for all projects that are submitted for approval to the Community Development Department.

Discharge: The flow of water from a pipe or from a drainage basin. If the discharge occurs in some course or channel, it is correct to speak of the discharge of a stream or river.

Diversion: In hydrologic modeling, the flow rate that is removed from the drainage system at a certain point, usually to be returned to the system at a downstream point. As in the water diverted from the overland flow by a storm sewer.

Drainage: A general term applied to the removal of surface or sub-surface water from a given area. The term is commonly applied herein to surface water.

Drainage Area: The geographical area drained by a river and its tributaries; an area characterized by all runoff being conveyed to the same outlet Also called Catchment Area, Watershed, Drainage Basin, and River basin.

Drainage System: The surface and subsurface system for the removal of water from the land, including both the natural elements of streams, marshes, swales, and ponds whether of an intermittent or continuous nature, and the man-made element which includes culverts, ditches, channels, retention facilities, detention facilities, gutters, streets, and storm sewer systems.

Drainageway: A route or watercourse along which water moves or may move to drain an area.

Drawdown Curve: The longitudinal profile at the water surface of an open channel as it accelerates through supercritical flow, such as weir flow over a dam.

Drop Structures: A structure of concrete, rock or other materials used to flatten the grade of a channel to reduce erosion tendencies of the flow.

Easement: Land set aside for the limited use of another's adjacent property.

Energy Gradient: The total energy level of water at all points along a longitudinal line. It is the sum of velocity head, pressure head and elevation of a flowing body of water.

Environmental Design: Designing projects with a plan for the total environment. It is the process of addressing surrounding environmental constraints and opportunities.

Erosion: Wearing away of the lands by running water and waves, abrasion and transportation.

Drainage Facilities: Any drainage and/or flood control structure including but not limited to storm inlets, storm sewers, manholes, junction boxes, outlet structures, channels, erosion control structures and devices, culverts, bridges, dams and detention reservoirs.

Flood: Water from a river, stream, watercourse, ocean, lake, or other body of standing water that temporarily overflows or inundates adjacent lands, and which may affect other lands and activities through stage elevation, backwater and/or increased ground water level.

Base Flood (FEMA/FIS): The flood produced by a 1% Annual Chance (100-year) rainfall at current levels of urbanization as determined by FEMA in the flood insurance study. Also called the FEMA Regulatory Flood.

Flash Flood: A flood of short duration with a relatively high peak rate of flow, usually resulting from a high intensity rainfall over a small area.

Maximum Probable Flood: The largest momentary flood discharge believed possible from a consideration of meteorological conditions in the watershed. Generally used to size spillways for dams.

Flood Control: The elimination or reduction of flood losses by the construction of flood storage reservoirs, channel improvements, dikes and levees, by-pass channels, or other engineering works. Sometimes called the structural alternate.

Flood Frequency: The Annual Chance of occurrence each year of a flood expressed as a percent or in years. For example, a 1% Annual Chance (100 year) flood has a 1 percent chance of occurrence each year and a 2% Annual Chance (50 year) flood has a 2 percent chance of occurrence each year.

Floodplain: The relatively flat or lowland area adjoining a river, stream, watercourse, lake, or other body of standing water which has been or may be covered temporarily by flood water. For administrative purposes, the floodplain may be defined as the area that would be inundated by the regulatory flood.

Floodplain Management: The operation of an overall program of corrective and preventive measures for reducing flood damage, including but not limited to, emergency preparedness plans, flood control works, and floodplain management regulations.

Floodplain Regulations: A general term applied to the full range of codes, ordinances and other regulations relating to the use of land and construction as influenced by water. The term encompasses zoning ordinances, subdivision regulations, building and housing codes, encroachment line statutes, open-area regulations and other similar methods of control affecting the use and development of the areas.

Flood (or Detention) Storage: Storage of water during floods to reduce downstream peak flows.

Floodway: Floodway is that portion of the FEMA/FIS floodplain required for the reasonable passage or conveyance of the design flood. This is the area of significant depths and velocities and due consideration shoPavuld be given to effects of fill, loss of cross-sectional flow area, and resulting increased water surface elevations.

Freeboard: The vertical distance between the normal maximum level of the surface of the water in a channel, bridge, culvert or dam, etc., and the top of the channel, the inside top of a bridge, the inside top of a culvert that is 20' or more in width, or the top of a roadway or dam.

Froude Number: A flow parameter, which is a measure of the extent to which gravitational action affects the flow, computed as $F = V/(gD)^{0.5}$, where V is the velocity (ft/sec), g is the gravitational constant (32.2 ft/sec^2) and D is the flow depth (feet). A Froude number greater than 1 indicates supercritical flow and a value less than 1 indicates subcritical flow.

Gabion: A wire basket containing earth or stones, deposited with others to provide protection against erosion.

Grade: (1) The inclination or slope of a channel, canal, conduit, etc., or natural ground surface, usually expressed in terms of the percentage of number of units of vertical rise (or fall) per unit of horizontal distance. (2) The elevation of the invert of the bottom of a conduit, canal, culvert, sewer, etc. (3) The finished surface of a canal bed, roadbed, top of an embankment, or bottom of an excavation.

Gradient: The rate of change per unit of length, usually applied to such things as elevation, velocity, pressure, etc.

Gutter: The portion of the street adjacent to the curb that forms the triangular channel section for street flow to an inlet.

Head: The amount of hydrostatic pressure, measured in feet, required to pass a certain flow rate downstream.

Headwater: (1) The upper reaches of a stream near its source. (2) The water upstream from a structure.

Hydraulics: The branch of science that deals with practical applications of the mechanics of water movement, used generally to identify the depth of water or pressure head at a particular location in a stream or conduit for a particular flow rate.

Hydraulic Gradient: A hydraulic profile of the level of the water, representing the sum of the depth of flow and the pressure head. In open channel flow the hydraulic gradient is represented by the water surface.

Hydraulic Jump: The hydraulic jump is an abrupt rise in the water surface which occurs in an open channel when water flowing at supercritical velocity is discharged into water flowing at subcritical velocity. The transition through the jump results in a marked loss of energy, evidenced by turbulence of the flow within the area of the jump. The hydraulic jump is often used as a means of energy dissipation.

Hydrograph: A graph showing stage, flow, velocity, or other property of water versus time at a given point on a stream or conduit, due to the upstream watershed's response to a rainfall event.

Hydrology: The branch of science that deals with the processes governing the depletion and replenishment of the water resources of the land areas of the earth, and specifically herein relates to the determination of flow rates at particular locations due to the response of a watershed to rainfall events.

Impervious: A term applied to a material through which water cannot pass, or through which water passes with great difficulty. Surfaces of concrete, asphalt and compacted gravel are considered impervious.

Infiltration: The process by which water on the ground surface enters the soil.

Inlet: (1) An opening into a storm sewer system for the entrance of surface storm runoff, more completely described as a storm sewer inlet. (2) The upstream connection between the surface of the ground and a drain or sewer, for the collection of surface or storm water.

Intensity: As applied to rainfall, a rate usually expressed in inches per hour.

Invert: The flowline, bottom, or lowest portion of the internal cross section of a conduit, or open channel.

Lag Time: 1) The time difference between the center of mass of rainfall and the runoff peak. See Time of Concentration 2) the time required for a hydrograph peak to travel to the next point downstream.

Left Bank: The left-hand bank of a stream cross section when the observer is facing downstream.

Lining: The material such as earth, concrete, rock, etc., making up the sides and bottom of a ditch, channel, and reservoir.

Manhole: A structure used in storm sewer line construction or an access hole usually with a flush cover, through which a person may pass to gain access to an underground or enclosed structure or storm sewer line.

Manual: The City of Moore Stormwater Management Criteria Manual

Stormwater Master Plan: The City of Moore Comprehensive Stormwater Management & Master Drainage Plan, dated January 2017, as adopted by the City Council.

Orifice: An opening with closed perimeter, and of regular form in a plate, wall, or partition, through which water may flow.

Outfall: The point of location where storm runoff discharges from a sewer or drain. Also applies to the outfall sewer or channel which carries the storm runoff to the point of outfall.

Peak Rate of Runoff: The maximum rate of runoff during a given runoff event.

Percolation: To pass through a permeable substance such as rainfall percolating into the ground.

Permeability: The property of a material such as the ground which permits the percolation of water.

Pervious: A term applied to a material such as natural ground through which water passes relatively freely.

Stormwater Pollution: The result of rainwater or snowmelt that picks up pollutants and sediments as it runs off roads, highways, parking lots, lawns, agricultural lands, septic systems, and other land-use activities that can generate pollutants.

Precipitation: Any moisture that falls from the atmosphere, including snow, sleet, rain and hail.

Rainfall Excess: As applied to runoff analysis, refers to the portion of rainfall which becomes surface runoff.

Rational Formula: A formula for estimating the peak rate of runoff from a given drainage basin.

Reach: Any length of river or channel.

Record Drawings: Those drawings which show the "as constructed" information on the construction plans.

Recurrence Interval: The average interval of time within which a given event will be equaled or exceeded once. For an annual series (as opposed to a partial duration series) the probability of occurrence in any one year is the inverse of the recurrence interval. Thus, a flood having a recurrence interval of 100 years has a 1 percent annual probability of being equaled or exceeded in any one year.

Regulatory Storm: The storm produced by the 1% Annual Chance (100-year) storm rainfall.

Regulatory Storm Discharge: The flow rates produced by the Regulatory Storm based on current levels of upstream urbanization.

Regulatory Floodplain: Those areas subject to inundation by the Regulatory Storm Discharges.

Return Period: The average interval of time within which a given event will be equaled or exceeded once. For an annual series (as opposed to a partial duration series) the probability of occurrence in any one year is the inverse of the recurrence interval. Thus, a flood having a recurrence interval of 100 years has a 1 percent annual probability of being equaled or exceeded in any one year.

Riprap: Broken stones or boulders placed compactly or irregularly on dams, levees, ditches, dikes, channels, etc., for protection of earth surfaces against erosion.

Right Bank: The right-hand bank of a stream cross section when the observer is facing downstream.

Routing, Hydraulic: The derivation of an outflow hydrograph of a channel or stream from a known inflow hydrograph by determining progressively the timing and shape of the flood wave at successive points along a stream or channel.

Runoff: That part of the precipitation which reaches a stream, drain, sewer, etc., directly or indirectly.

Runoff Coefficient: A decimal number used in the Rational Formula which defines the runoff characteristics of the drainage area under consideration.

Scour: The erosive action of running water in streams or channels in excavating and carrying away material from the bed and banks.

Sediment: Material of soil and rock origin transported, carried, or deposited by water.

Stilling Basin: A basin or reservoir installed in a storm drainage system to retard velocity, causing sedimentation, and providing storage for deposited solids.

Siphon: Inverted siphons (sometimes called sag culverts or sag lines) are used to convey water by gravity under roads, railroads, other structures, various types of drainage channels and depressions. An inverted siphon is a closed conduit designed to run full and under pressure. Siphons are not allowed in the City of Moore.

Slope: See Grade.

Slope, Critical: The slope or grade of a channel that is exactly equal to the loss of head per foot resulting from flow at a depth that will give uniform flow at critical depth; the minimum slope of a conduit which will produce critical flow.

Slope, Friction: The friction head or loss per unit length of channel or conduit. For uniform flow the friction slope coincides with the energy gradients, but where a distinction is made between energy losses due to bends, expansions, impacts, etc., a distinction must also be made between the friction slope and the energy gradient. The friction slope is equal to the bed or surface slope only for uniform flow in uniform open channels.

Spillway: A passageway in or about a dam or other hydraulic structures, for the escape of excess water.

Storage with Respect to Channel Design:

Overbank Storage: The temporary storage volume of storm runoff water in the overbanks, away from the main channel.

Channel Storage: The volume of storm runoff water present in the channel.

Detention Storage: The volume of water that is temporarily stored in a detention basin.

Depression Storage: That portion of the rainfall that is collected and held in natural or man-made depressions and does not become part of the general runoff.

Storm, Design: The time variation of rainfall. For analysis purposes, a "design storm" or pre-determined rainfall pattern is used based on "typical" Oklahoma storms. The City of Moore requires the use of a 24-hour storm for hydrologic analyses.

Storm frequency: The chance of occurrence each year of a flood expressed as a percent or in years. For example, a 1% Annual Chance (100 year) flood has a 1 percent chance of occurrence each year and a 2% Annual Chance (50 year) flood has a 2 percent chance of occurrence each year.

Storm Sewer: A continuous closed conduit for conducting storm water that has been collected by inlets or collected by other means. A storm sewer system is a system of inlets, pipes, manholes, junctions, outlets, and other appurtenant structures designed to collect and convey storm runoff to a defined drainage way.

Stream Flow: A term used to designate the water that is flowing in a stream channel, canal, ditch, etc.

Street Flow: The total flow of storm runoff in a street, usually being the sum of the gutter flows on each side of the street. Also, the total flow where there are not curbs and gutters.

Watercourse: A natural or artificial channel for passage of water.

Watershed: The contributing drainage area to drainage facility expressed as acres, square miles, or other unit of area. Also see "drainage area".

Appendix A – Excerpted from Low Impact Development, a design manual for urban areas[39]

[39] University of Arkansas Community Design Center, FAY JONES SCHOOL OF ARCHITECTURE, UNIVERSITY OF ARKANSAS PRESS, A COLLABORATION, FAYETTEVILLE 2010, http://www.bwdh2o.org/wp-content/uploads/2012/03/Low_Impact_Development_Manual-2010.pdf

LID

Low Impact Development

a design manual for urban areas

UACDC

Low Impact Development: a design manual for urban areas

Arkansas

How to Use This Manual

Environmental planning typically enjoys success in natural settings, but the real challenge for developing sustainable cities is ensuring ecosystem integrity within urban contexts. Naturally-determined ecosystems have been irrevocably altered by human activity. Indeed, the greatest ongoing problem in planning involves designing within human-dominated ecosystems. After all, 80% of the US population now lives in urban areas. *Low Impact Development: A Design Manual for Urban Areas* introduces general audiences to designing landscapes for urban stormwater runoff—a primary source of watershed pollution. This manual can be reviewed episodically, much like a lifestyle publication, or read in its entirety for a comprehensive understanding. The goal is to motivate awareness and implementation of LID in a wide cross-section of stakeholders, from property owners to municipal governments that regulate infrastructure development. Though not exhaustive in its coverage of LID techniques (i.e., you will not be able to engineer a LID project from this manual), this manual does provide a holistic framework in which a novice homeowner and an experienced developer can each find an equally transformative role to enact.

If you live in the blue, this manual is for you!

FAY JONES SCHOOL OF ARCHITECTURE
UNIVERSITY OF ARKANSAS PRESS
A COLLABORATION
FAYETTEVILLE 2010

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LID

Low Impact Development

a design manual for urban areas

University of Arkansas Community Design Center

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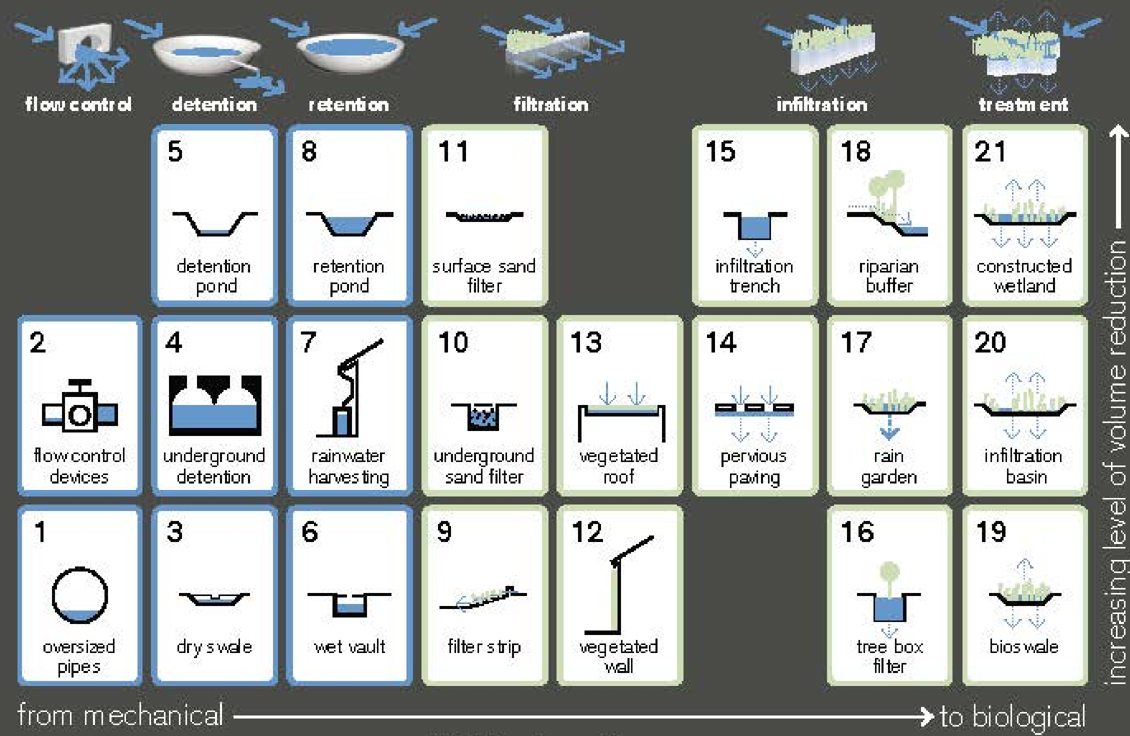
FAYETTEVILLE 2010

Contents

What are the LID facilities?

The LID Facilities Menu organizes facilities based on increasing level of treatment service (quality) as well as increasing level of volume reduction (quantity). Therefore, number one (1), flow control devices offer the least amount of treatment services while number twenty-one (21), constructed wetland offers the most. Most municipalities require drainage infrastructure to manage 100-year storm events. Though one facility alone will not likely satisfy performance requirements, facilities with varying levels of service in a treatment network will provide superior levels of treatment and volume reduction.



LID Facilities Menu

LID Facility Selection

Selection of the optimum LID facility or combination of facilities for a project or site depends on the desired hydrologic outcomes. While site planning techniques can greatly reduce the hydrologic impacts of development, additional measures are likely needed to mimic pre-development hydrology—the goal of LID. Once the site hydrology has been analyzed for pre-development conditions and post-development objectives, the site can be shaped through design of a LID treatment network.

The first step in determining LID facility selection requires an evaluation of site opportunities and constraints. Opportunities involve physical conditions of a site, such as soils (infiltration rate), water table depth, bedrock depth, climate, drainage area, precipitation patterns, slope and available land. These physical conditions influence the types of facilities to be used. Therefore, an understanding of the project site is critical to LID facility selection (see "The LID Watershed Approach" pp. 30-43). Remember to think small in regards to the size of the managed area, which encourages a network of smaller, distributed facilities in place of one large facility.

The second step in determining LID facility selection is to define the type of hydrologic controls required. Hydrologic controls include flow control, detention, retention, filtration, infiltration and treatment. Control functions are quantifiable for pre-development conditions based on various design parameters such as stormwater runoff volume, peak discharge, and frequency and duration of discharge. Like conventional stormwater management infrastructure, LID networks must meet local stormwater rate, volume, and water quality treatment mandates for post-development conditions.

Other site feasibility factors include maintenance and management protocols, community acceptance, and cost. Facility selection is not just a matter of choosing from a menu of preferred practices, but rather is part of a larger planning and design process. The facilities by themselves may not be sufficient in restoring the hydrologic functions of a site. LID solutions are most effective when combined with other site planning practices described in "How can we implement LID?" pp. 44-141.

Third, facilities are arranged on site in varying configurations to determine if they meet both site constraints and local regulations, measured by hydrological modeling. Due to multiple variables, facilities are arranged and sized until hydrologic control objectives are optimized. This interactive process usually identifies several design options that meet the development goals. The final configuration can then be determined by space requirements, site aesthetics and cost.

If LID facilities alone cannot meet the hydrologic control objectives of a project it may be necessary to create a hybrid solution, incorporating conventional hard engineering practices. Severe site constraints, such as soils with low infiltration rates, the location of the water table, and intensive land uses, may render LID facilities insufficient. Nonetheless, LID facilities should be used wherever possible, supplemented by conventional practices such as detention ponds and pipes to meet hydrologic design objectives.

Information on LID facility selection from: *Low-Impact Development Design Strategies: An Integrated Design Approach*, prepared by Prince George's County, Maryland

optimal level of service
detention/infiltration

location in LID network
optimally placed after filtration facilities to prevent excessive sedimentation

scale
maximum watershed runoff area is 25 acres

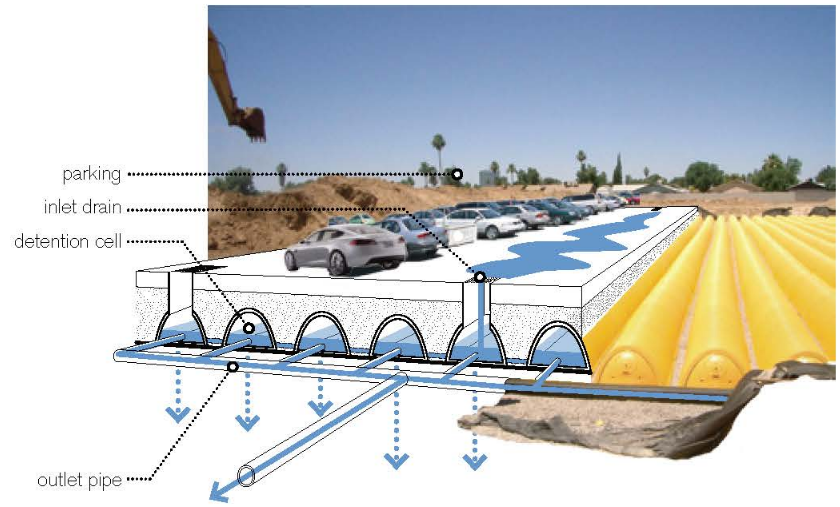
management regime
inspection and sediment cleanout

Underground Detention

Underground detention systems detain stormwater runoff prior to its entrance into a conveyance system.

Underground storage systems store and slowly release runoff into the LID network. Some systems can infiltrate stormwater if the soil beneath is permeable. Underground storage is employed in places where available surface area for on-grade storage is limited.

Underground storage reduces peak flow rate through metered discharge and has potential for infiltration. Improved water quality is achieved by sedimentation, or the settling of suspended solids. Though at first costly, underground detention systems are easy to access and maintain.



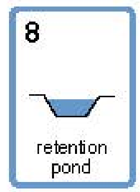
References:
Low Impact Development Manual for Michigan
Urban Design Tools–Low Impact Development
Minnesota Urban Small Sites BMP Manual

optimal level of service
retention/treatment

location in LID network
downstream of catchment and runoff, usually constructed at the lowest point of the site

scale
can be used for residential, commercial, and industrial sites, with watershed runoff areas no smaller than 10 acres depending on regional precipitation

management regime
inspected semiannually to confirm that drainage is functioning properly and to remove sediment, accumulated trash, and debris

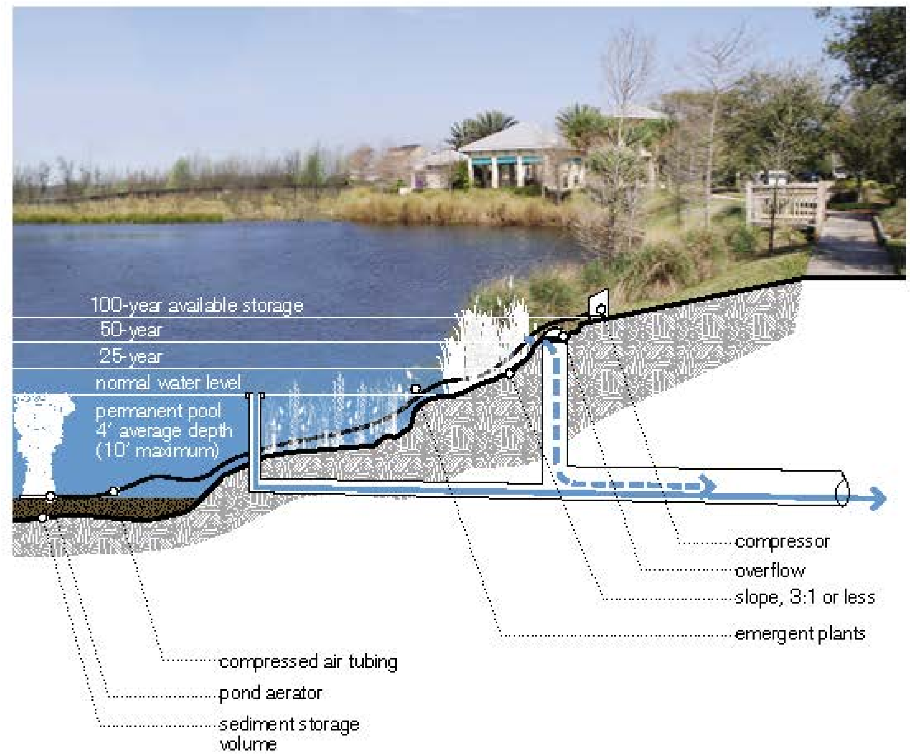


Retention Pond

A retention pond, also known as a wet pool or wet pond, is a constructed stormwater pond that retains a permanent pool of water, with minor biological treatment.

Wet ponds remove pollutants through biological uptake processes and sedimentation. The amount of pollutants that are removed from stormwater runoff is proportionate to the length of time runoff remains in the pond, as well as the relation of runoff to retention pond volume. Since retention ponds must maintain a permanent pool, they cannot be constructed in areas with insufficient precipitation or highly permeable soils, unless the soil is compacted or overlain with clay. Generally, continual drainage inputs are required to maintain permanent pool levels.

One advantage of a retention pond is the presence of aquatic habitat when properly planted and maintained. The use of a pond aerator is necessary to prevent stagnation and algae growth that can lead to eutrophication, or an anaerobic environment. A balanced aerobic environment is a necessary condition for aquatic life and pest control. Regular maintenance inspections are needed to ensure proper drainage, aerobic functioning and aeration, and vegetative health. Trash, debris, and sediment will need to be removed periodically.



References:
Low Impact Development Manual for Michigan
Minnesota Urban Small Sites BMP Manual
EPA Storm Water Technology Fact Sheet-Wet Detention Ponds

optimal level of service
filtration

location in LID network
upstream of major treatment systems

scale
from a small slope at streetside to the size of a large field

management regime
trash and sediment removal, and mowing

9

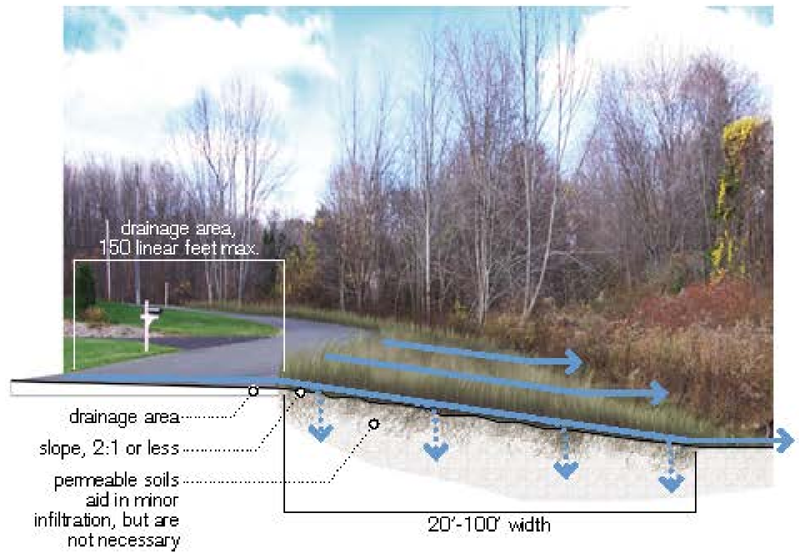
filter strip

Filter Strip

A filter strip is a sloped medium that attenuates stormwater runoff by converting it into sheet flow, typically located parallel to an impervious surface such as a parking lot, driveway, or roadway.

Filter strips use vegetation to slow runoff, allowing suspended sediment and debris loads to drop out of runoff flow. This prevents clogging of stormwater drainage systems or receiving waterbodies. Stormwater runoff should be uniformly distributed along the top of the entire filter strip using a flow control facility such as a level spreader. Other treatment facilities, such as a swale, should be used for channelized flows. The drainage area should not exceed 150 linear feet to ensure proper functioning of the filter strip. The lateral slope of the filter strip should be one to two percent. A series of stepped level spreaders could compensate for slightly steeper slopes.

Filter strip areas cannot be used for construction material storage or activities that could disturb the ground surface. Regular inspection and maintenance are required to prevent clogging by sediment and/or debris. Filter strips should be located in sunny areas to dry out between rain events.



References:
Low Impact Development Design Strategies–An Integrated Design Approach
Low Impact Development Manual for Michigan
United States Department of Housing and Urban Development
Minnesota Urban Small Sites BMP Manual

optimal level of service
filtration/infiltration/treatment

location in LID network
apply upstream of treatment systems to provide sediment removal and to reduce runoff volume

scale
from a parking stall to a parking lot or street

management regime
vacuum-based sediment removal from paving; turf paver systems may need to be mowed and irrigated to maintain vegetation

14

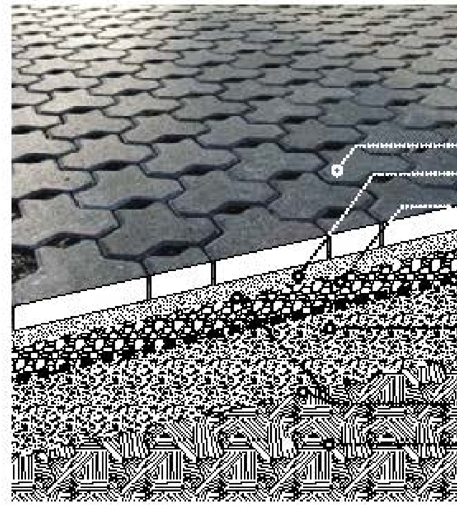
pervious paving

Pervious Paving

Pervious, or permeable, paving allows water to vertically flow through hard surfaces. As substitutes for impervious paving, they support both pedestrian and vehicular traffic.

A pervious paving system includes a subsurface base made of course aggregate for stormwater storage. In some designs, pervious pavement is supported by underground layers of soil, gravel and sand to increase storage and maximize infiltration rates. Pervious paving removes sediment and other pollutants. It acts to reduce and distribute stormwater volume, encouraging groundwater infiltration. Multiple types of pervious paving, including modulated precast pavers, poured in place systems, porous asphalt, porous concrete, and gravel, offer varying levels of service. Reduction of the urban heat island effect is possible when using high-albedo, lightly colored systems.

Large scale vacuums must be used to clean out gravel, paver, and porous systems. Turf paver systems may need occasional mowing and irrigation (see "Surface Materials" pp. 78-79).



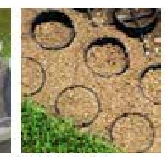
pervious surface m aterials


porous asphalt /concrete


open grid pavers


gravel paving


grass concrete paving


turf pavers

References:
Low Impact Development Manual for Michigan
Stormwater Management Handbook
Minnesota Urban Small Sites BMP Manual
Low Impact Development Technical Guidance Manual for Puget Sound

optimal level of service
filtration/infiltration/treatment

location in LID network
downstream of all LID facilities, before waterbodies

scale
100' to 300' wide

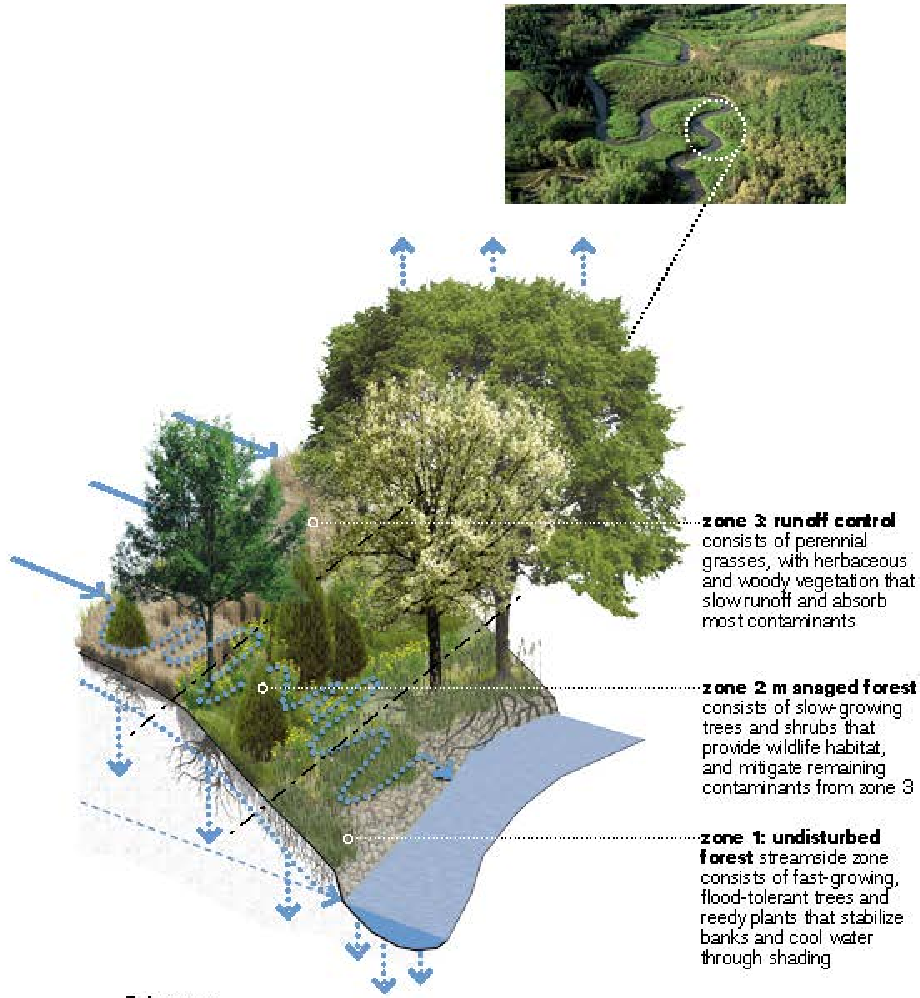
management regime
trash and sediment removal as necessary, and occasional mowing in zone 3

Riparian Buffer

A riparian buffer is a strip of hydric soil with facultative vegetation along the banks of a river or stream offering niche ecotone services.

Riparian buffers are a simple, inexpensive way to protect and improve water quality through local plant communities. Between 50 and 85 percent of stormwater pollutant loads can be filtered within 100 to 300 foot vegetated buffers. Buffer strips structurally stabilize banks and shorelines to prevent erosion and slumping. Trees and shrubs provide shade to maintain consistent water temperature necessary for nutrient exchange and the survival of some aquatic life. Buffer width is based on surrounding context, soil type, size and slope of catchment area, and vegetative cover.

Riparian buffers are most effective when combined with flow attenuation devices to avoid scouring from high velocity flows throughout a stream channel. Some management is required when riparian buffers are near urban development. Avoid disturbing zone 1 as tree litter aids in flow control and filtration.



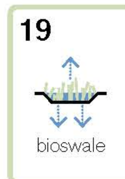
References:
Low Impact Development Manual for Michigan
Conservation Buffers: Design Guidelines for Buffers, Corridors, and Greenways

optimal level of service
filtration/infiltration/treatment

location in LID network
downstream of filtration components, but upstream of larger detention, retention, or treatment facilities

scale
2'-8' wide with 2"-4" optimal water depth

management regime
occasional removal of trash and pruning of vegetation

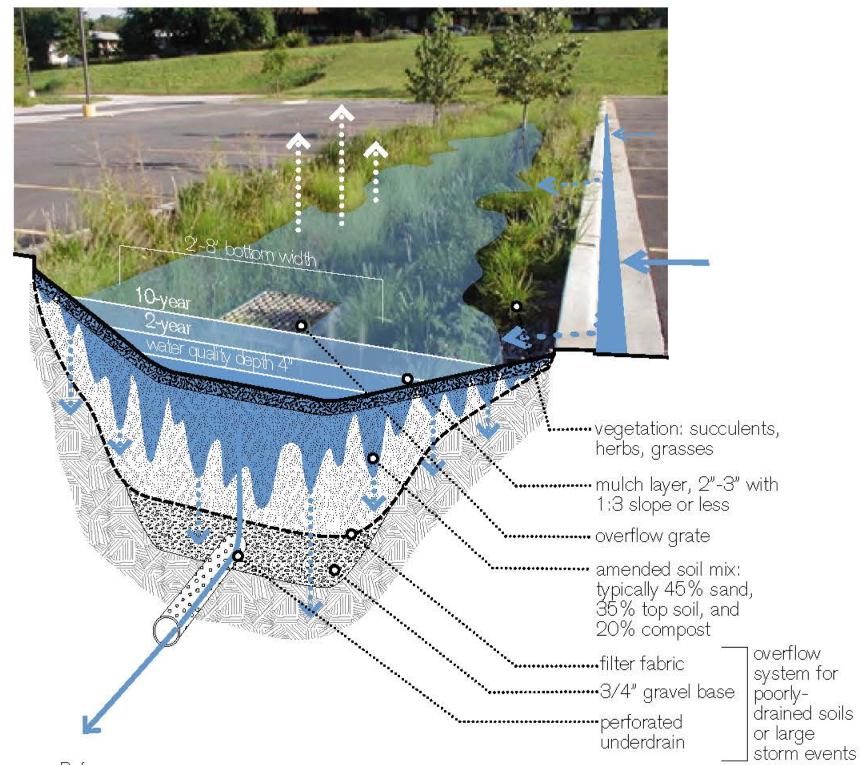


Bioswale

A bioswale is an open, gently sloped, vegetated channel designed for treatment and conveyance of stormwater runoff.

Bioswales are a bioretention device in which pollutant mitigation occurs through phytoremediation by facultative vegetation. Bioswales combine treatment and conveyance services, reducing land development costs by eliminating the need for costly conventional conveyance systems. The main function of a bioswale is to treat stormwater runoff as it is conveyed, whereas the main function of a rain garden is to treat stormwater runoff as it is infiltrated. Bioswales are usually located along roads, drives, or parking lots where the contributing acreage is less than five acres.

Bioswales require curb cuts, gutters or other devices that direct flow to them. They may require an underdrain where soil permeability is limited, as well as an overflow grate for larger storm events.



References:
Low Impact Development Design Strategies–An Integrated Design Approach
Low Impact Development Manual for Michigan
Low Impact Development Technical Guidance Manual for Puget Sound
United States Department of Housing and Urban Development
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