Grand Prairie
TEXAS

DRAINAGE DESIGN MANUAL

VOLUME 1
January 2017

ACKNOWLEDGEMENTS

At the January 10th 2017 City Council meeting the City of Grand Prairie received the coveted Silver iSWM regional community award by the North Central Texas Council of Governments (NCTCOG) in applying the principles and exceeding the requirements set forth in the integrated Stormwater Management Program which achieves water quality protection, streambank protection and flood mitigation. See the picture on the Cover Sheet. Pictured from left to right are: Romin A. Khavari, City Engineer, Stephanie Griffin, City Stormwater Utility Manager / Floodplain Administrator, City Mayor Ron Jensen, Derica Peters, NCTCOG Environment and Development Planner and Christian Y. Agnew, City Project Manager.
I would like to acknowledge the efforts of Christian Agnew, P.E., who worked closely with Stephanie Griffin, P.E. CFM and Echo Rexroad in making this happen for the City of Grand Prairie.

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In conclusion, my sincere appreciation goes for the team effort it has taken to get this document released for public use in service to our Citizens.

Romin A. Khavari, P.E., CFM
City Engineer

CITY OF GRAND PRAIRIE DRAINAGE DESIGN MANUAL
JANUARY 2017 REVISIONS TO DECEMBER 2016 EDITION

SECTION	**ACTION**
Acknowledgments	Updated Acknowledgments page to include City of Grand Prairie receipt of Silver iSWM regional community award by NCTCOG.
Revisions	Added Revisions to January 2017 Edition.
APPENDIX G.2	Moved Appendix G.2 to Volume 1 and updated section G.2 to the current iSWM Technical Manual – Planning Category providing a link to the NCTCOG web site.
VOLUME 2	**Updated the Appendices as given below**:
APPENDIX G	Moved Appendix G.2 to Volume.
APPENDIX H	Updated to the current iSWM Technical Manual – Hydraulics Category.
APPENDIX I	Updated to the current iSWM Technical Manual – Site Development Controls Category providing a link to the NCTCOG web site.
APPENDIX J	Updated the reference for the appendix.
APPENDIX K	Updated to the current iSWM Technical Manual – Landscape Category providing a link to the web site.
APPENDIX L	Updated the reference for the appendix.

NO FURTHER CHANGES

CITY OF GRAND PRAIRIE DRAINAGE DESIGN MANUAL
DECEMBER 2016 REVISIONS TO AUGUST 2016 EDITION

SECTION	ACTION
Acknowledgments	Updated Acknowledgments page.
Revisions	Added Revisions to August 2016 Edition.
2.6.B.4	Updated required freeboard above the 100-year flood elevation for pavement of street to be consistent with UDC Article 15;
9.1 Bridge	Updated required freeboard above the 100-year flood elevation of low point of pavement at bridge crossings to be consistent with other manual and UDC Article 15 requirements.
9.2 Culverts	Updated required freeboard above the 100-year flood elevation of low point of pavement at culvert crossings to be consistent with other manual and UDC Article 15 requirements.

NO FURTHER CHANGES

CITY OF GRAND PRAIRIE DRAINAGE DESIGN MANUAL
AUGUST 2016 REVISIONS TO AUGUST 2015 EDITION

SECTION	**ACTION**
Acknowledgments	Updated Acknowledgments page.
Revisions	Added Revisions to August 2015 Edition.
4.3	Clarified area-weighted runoff coefficient determinations;
4.3. B	Provided for channelized flow velocity determinations for street gutter flows in Time of Concentration calculations and Total Time of Travel for channelized flow.
4.5	Specified rainfall distribution to be used in the SCS Method calculations;
	Specified Channelized Flow time of concentration determination method to be used;
	Made minor coefficient and labeling updates.
6.1	Corrected the section title spelling;
	Clarified calculations for non-depressed inlet flow capture calculations.
6.2	Updated the section to conform to latest edition of HEC-22 requirements for sag inlet flow capture determinations;
	Provided a flow capture table for City curb inlets in sags.
6.3	Clarified flow capture determinations for city Y-inlets.
6.4	Made grammar correction and reference update and rounded the value for g to be used in the calculations consistent with use elsewhere in the manual.
6.5	Clarified the flow capture methods to be used for non-recessed inlets.
7.6	Added a last paragraph to address pipe flow discharge velocity calculations and requirements for the low channel tailwater conditions.

NO FURTHER CHANGES

CITY OF GRAND PRAIRIE DRAINAGE DESIGN MANUAL
AUGUST 2015 REVISIONS TO JUNE 2015 EDITION

SECTION	ACTION
Acknowledgments	Updated Acknowledgments page.
Revisions	Added Revisions to June 2015 Edition.
TABLE OF CONTENTS	Updated to reflect changes in document.
LIST OF FIGURES	Updated figure numbers and page numbers. Removed Figures 6.4A through 6.4F and referred user to HEC-22 Manual for grate design details.
LIST OF TABLES	Updated table numbers and page numbers.
ALL SECTIONS	Added “(0.2% annual chance)” to 500-year flood descriptions. Added “(1% annual chance)” to 100-year flood descriptions. Added “(4% annual chance)” to the 25-year flood description. Added “10% annual chance)” to 10-year flood descriptions. Added “(50% annual chance)” to 2-year flood descriptions.
1.1	Updated and clarified the purpose of the Drainage Design Manual.
1.2	Added reference to UDC Articles 14 and 15.
2.1	Updated and clarified the purpose and scope of the drainage policy. Added reference to UDC Articles 14 and 15.
2.2	Updated FEMA terminology. Updated Minimum Finished Floor Elevation to read Lowest Floor Elevation to reflect update in FEMA terminology. Added text clarifying the owner’s responsibility to comply with UDC Articles 12, 14, and 15, as well as the fees described in UDC Article 22.
2.3	Added cable to list of utilities and replaced sewer with wastewater.
2.5	Updated “drainage management area” to read “drainage easement”. Replaced duplicate text regarding TPDES with instructions that refer user to UDC Article 14. Added clarification regarding permanent, post-construction BMPs referenced in iSWM. Added floodways to the list of areas in which retaining walls are not allowed. Clarified when engineering plans are required.
2.6	Updated text to read “fully developed watershed condition” to clarify “ultimate”. Updated Minimum Finished Floor Elevation to read Lowest Floor Elevation to reflect update in FEMA terminology. Added statement that a separate building permit is required for all retaining walls. Replaced duplicate text for floodplain requirements with instructions to refer to UDC Article 15. Added requirement that the base flood elevation be shown on plans for properties located within 200-feet of the regulated floodplain. Allows City staff to require engineering plans for retaining walls having a height less than four feet.
2.7	Expanded the CDC text to match the CDC Manual. Removed duplicative text that is covered in UDC Article 15.
3.0	Removed UDC Articles 14 and 15, which are stand-alone documents.
4.0	Became Section 3.0
5.0	Became Section 4.0

6.0	Became Section 5.0
7.0	Became Section 6.0.
7.4	Became 6.4. Removed inlet grate design details, formulas and figures and referred user to the HEC-22 Manual for appropriate details.
8.0	Became Section 7.0
9.0	Became Section 8.0
9.2	Became Section 8.2. Updated instructions to obtain floodplain development permit application from the Floodplain Administrator's office. Added 1% annual chance flood to the description of the 100-year flood.
10.0	Became Section 9.0. Added 1% annual chance flood to the description of the 100-year flood.
10.1	Became Section 9.1. Added 1% annual chance flood to the description of the 100-year flood.
10.2	Became Section 9.2. Added 1% annual chance flood to the description of the 100-year flood.
11.0	Became Section 10.0
11.1	Became Section 10.1. Added 1% annual chance flood to the description of the 100-year flood.
12.0	Became Section 11.0. Removed duplicative text with instructions to refer to UDC Article 15.
APPENDIX A.1	Removed.
APPENDIX A.2	Removed.
APPENDIX A.3	Removed.
APPENDIX A.4	Became Appendix A.1
APPENDIX A.5	Became Appendix A.2
APPENDIX B	Updated definitions to reflect current FEMA definitions.
APPENDIX C	Removed.
APPENDIX D	Became Appendix C.
APPENDIX E	Became Appendix D.
APPENDIX F	Became Appendix E.
APPENDIX G	Became Appendix F.
APPENDIX H	Became Appendix G.
APPENDIX I	Became Appendix H.
APPENDIX J	Became Appendix I.
APPENDIX K	Became Appendix J.
APPENDIX L	Became Appendix K.
APPENDIX M	Became Appendix L.

NO FURTHER CHANGES

CITY OF GRAND PRAIRIE DRAINAGE DESIGN MANUAL
JUNE 2015 REVISIONS TO SEPTEMBER 2014 EDITION

SECTION	ACTION
Acknowledgments	Updated Acknowledgments page.
Revisions	Added Revisions to June 2015 Edition:
2.6.D	In Section 2.6.D. Open Channels and Culvert/Bridge Plans. Deleted the last sentence of the second paragraph concerning parking lots in the 100-year floodplain of the West Fork Trinity River.

NO FURTHER CHANGES

CITY OF GRAND PRAIRIE DRAINAGE DESIGN MANUAL
SEPTEMBER 2014 REVISIONS TO JULY 2014 EDITION

SECTION	ACTION
Revisions	Added Revisions to September 2014 Edition;
VOLUME 2	Deleted obsolete NCTCOG iSWM Limited Use agreement portion.

NO FURTHER CHANGES

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1.0 INTRODUCTION

1.1 Purpose of Drainage Design Manual

The purpose of this manual is to establish standard criteria principles, procedures, and practices for design of storm drainage facilities within the City and its extraterritorial jurisdiction in a user friendly format. The design factors, formulas graphs and procedures described in the following pages are intended to serve as guidelines for the mitigation of drainage problems involving the volume and rate of flow, method of collection, storage, conveyance and disposal of stormwater and erosion protection from stormwater flows. Ultimate responsibility for actual design remains with the design engineer. Users of this manual should be knowledgeable and experienced in the theory and application of drainage engineering. Any deviation from the requirements of this manual must be approved by the City Engineer.

The purpose of this manual is to establish standard criteria principles, procedures, and practices for design of storm drainage facilities.

1.2 Application of Drainage Design Manual

The procedures, policies and standards of this manual govern storm drainage facilities within the City of Grand Prairie and its extraterritorial jurisdiction. This manual provides additional drainage requirements to those specified in City of Grand Prairie Unified Development Code (UDC) Article 14 Drainage and Article 15 Floodplain Management.

This manual applies to all areas within the City of Grand Prairie and its extraterritorial jurisdiction. The City will not approve a plat or subdivision that does not conform to the minimum FEMA regulations regarding floodplain management.

The City of Grand Prairie's UDC Articles 14 and 15 are not intended to repeal, abrogate or impair any existing easements, covenants, or deed restrictions. However, where they and another conflict or overlap, whichever imposes the more stringent restrictions shall prevail.

In the interpretation and application of this manual, all provisions shall be considered as minimum requirements and shall be liberally construed in favor of the City, and shall not be deemed a limitation or repeal of any other powers granted by State statutes.

1.3 References

A. U.S. Department of Commerce, "Rainfall Frequency Atlas of the United States, Technical Paper No. 40," Washington, D.C., May, 1961.

B. National Oceanic and Atmospheric Administration, "Five-to 60 Minute Precipitation Frequency for the Eastern and Central United States, Technical Memorandum NWS HYDRO-35," June, 1977.

C. Federal Highway Administration, "Urban Drainage Criteria and Design Manual, Hydraulic Engineering Circular No. 22," FHWA-NH1-01-021, Washington, D.C., September, 2009.

D. TxDOT, "Hydraulic Design Manual," March, 2004.

E. Federal Highway Administration, “Hydraulic Design of Highway Culverts, Hydraulic Design Series No. 5,” FHWA-ID-85-15, Washington, D.C., September, 1985.

F. U.S. Soil Conservation Service, “Urban Hydrology for Small Watersheds,” Technical Release No. 55, June, 1986.

G. Richard H. McCuen, “Hydrologic Design and Analysis”, Prentice Hall 2nd Edition, 1998.

H. U.S. Army Corps of Engineers Institute of Water Resources, “HEC-HMS Technical Reference Manual”, Hydraulic Engineering Center, April 2006.

I. U.S. Army Corps of Engineers, River Analysis System, HEC-RAS, Version 5.0, February, 2016.

Scanned in Charts and Tables have a letter reference after the name which corresponds to these references.

2.0 DRAINAGE POLICY

2.1 Purpose and Scope

The purpose of drainage policies and standards are to protect the general health, safety, and welfare of the public by reducing flooding potential, controlling excessive runoff, minimizing erosion and siltation problems, and eliminating damage to public facilities resulting from uncontrolled stormwater runoff. This manual provides additional drainage requirements to those specified in City of Grand Prairie Unified Development Code (UDC) Article 14 Drainage and Article 15 Floodplain Management.

It is the intent of the City of Grand Prairie to continue to working with the North Central Texas Council of Governments (NCTCOG) to enhance and develop this manual and future stormwater management throughout the city.

2.2 Drainage Plan Submittals

Optional Conceptual Study

Based upon the review of existing conditions and site analysis, the design engineer is encouraged to develop a conceptual site plan for the project. The conceptual site plan can be submitted to the City for review as part of the conceptual plan review, described in UDC Article 17.

As part of the conceptual plan, the site designer can perform most of the layout of the site including the conceptual stormwater management system design and layout. The conceptual site plan allows the design engineer to propose a potential site layout and gives the developer and local review authority a “first look” at the stormwater management system for the proposed development. The conceptual site plan can be submitted to the City before detailed preliminary site plans are developed.

The City encourages the use of integrated site design practices consistent with the City’s Comprehensive Master Plan and the NCTCOG *integrated Stormwater Management (iSWM) Design Manual for Site Development* as applicable to develop the site layout. The iSWM Design Manual for Site Development details appropriate steps to complete a conceptual plan and includes a reference checklist. The City’s recommended integrated site design practices can be found in Appendix G. These practices include:

- preserving the natural feature conservation areas defined in the site analysis;
- fitting the development to the terrain and minimizing land disturbance;
- reducing impervious surface area through various techniques; and
- preserving and utilizing the natural drainage system wherever possible

Perform screening and conceptual selection of appropriate structural stormwater controls and identification of potential siting locations. It is extremely important at this stage that stormwater system design is integrated into the overall site design concept in order to best reduce the impacts of the development to the pre-developed drainage conditions as well as provide for the most cost-effective and environmentally sensitive approach. Using hydrologic calculations, the goal of mimicking pre-development conditions can serve a useful purpose in planning the stormwater management system.

Preliminary Study Required

Drainage design plans prepared for projects in the City of Grand Prairie shall be consistent with the **City Wide Drainage Master Plan Road Map** and individual creek or internal study requirements where applicable. The owner may be required to provide, at such owner's expense, a preliminary drainage study of the area proposed for development, in conjunction with any preliminary plat submittal. Master hydrologic and hydraulic models are available from the City Engineer upon request. These models represent the most current information available and should be used for all preliminary studies. For revision to the master models refer to “Guidance for Revision to the City of Grand Prairie Master Hydrology and Hydraulic Models” in Appendix A. The preliminary drainage study shall be submitted to the City Engineer prior to approval of the preliminary plat by the Planning and Zoning Commission. The study shall include:

A. A contour map of the entire drainage area contributing runoff to the site on a surveyed or City topographic map (scale 1 inch=200 feet). Drainage areas greater than 400 acres may be shown on a map at a smaller scale subject to the concurrence of the City Engineer.

B. Sufficient design calculations showing preliminary sizes and locations of all on-site adjacent and nearby existing and proposed drainage facilities, including storm drains, culverts, channels, inlets, detention basins, floodplains, etc.

C. Design calculations and floodplain delineations for the fully developed floodplain & FEMA floodplain/floodway lines with flood elevations and lowest floor elevations for each proposed structure on lots within 200 feet of the floodplain or detention basin shown. The floodplain information required by UDC Article 15 “Floodplain Management” shall also be included in the report.

Final Plan Required

The owner shall provide, at the owner's sole expense, complete final plans and specifications for the drainage facilities associated with a final plat or building permit. The plans and specifications shall be prepared by a civil engineer licensed to practice in the State of Texas and experienced in municipal drainage work.

The plans and specifications shall be submitted to the City Engineer for review and concurrence prior to any construction.

No person shall fill, grade, excavate or otherwise disturb the surface of real property within the city without first having secured a clearing/grubbing/earthwork development permit from the City. It shall be the duty of each person owning or having control of real property within the city to prevent soil, mud, rock or other debris from such real property being deposited or otherwise transported onto the streets, alleys, utility facilities, rights-of-way, or easements or into creeks, lakes, channels, or other water bodies. The owner is responsible for obtaining all applicable permits as outlined in the City’s UDC Articles 12, 14, and 15 and pay all associated fees for said permit per UDC Article 22. In addition, the owner is responsible for obtaining all applicable State and Federal permits and pay all associated fees for said permits. The City Engineer may require information as necessary to evaluate the impacts of the proposed project.

The owner and owner's engineer shall be responsible for the accuracy of the information furnished in the design of the storm drainage facilities. The owner’s engineer shall submit as-built construction plans and the owner shall be responsible for the proper construction of all storm drainage facilities per the City approved plans.

Reports

A. A written report documenting the methodologies employed and the results of the study must be approved by the City. The report shall include a map showing the soil types of the contributing area. If the study involves a FEMA flood zone, then two reports should be submitted, one for fully developed watershed conditions for the City and one for existing watershed conditions with completed FEMA forms and review fees.

B. The reports should contain tables comparing pre- and post-project expansion and contraction coefficients, Manning's *n* values, flood storage within the project area, and flood flow rates. The report should also include a table comparing the duplicate, existing conditions, and post-project base flood elevations, channel and overbank velocities, and floodway width and elevation (where applicable). The report shall include the work maps, FIRM location map and preliminary or final plat (where required). For projects where the hydraulic model was developed by the engineer, the report should also contain tables with the reach length and channel width for each stream section. For hydraulic models, the report should contain plots of pre- and post-project stream profiles and cross sections. Electronic copies of the hydrologic and hydraulic models should be included.

C. Projects on streams that may impact the floodplain under the jurisdiction of other local governments the Engineer must acquire written approval from those local governments, and property owners in accordance with FEMA requirements. Note that this includes portions of Mountain Creek where the City of Dallas may require no increase in flood elevations. See *Section 2.9* Trinity River Corridor Development Certificate.

2.3 Floodplain Development

The National Flood Insurance Program (NFIP) is a federal program established by Congress in 1968 that allows property owners to purchase federally backed flood insurance within communities that participate in the program. The City of Grand Prairie is a participant as are most other cities. In return for this protection, the City of Grand Prairie must implement floodplain management measures to reduce flood risk to new and existing development in accordance with federal regulations. Article 8280-13 of Vernon's Texas Civil Statutes authorizes local governments to adopt regulations designed to minimize flood loses.

The purpose of floodplain regulations is to promote the public health, safety, and welfare and to minimize public and private losses due to flood conditions in specific areas by provisions designed to:

A. Protect human life and health,

B. Minimize expenditure of public money for costly flood control projects,

C. Minimize the need for rescue and relief efforts associated with flooding that are generally undertaken by the City at the expense of the general public,

D. Minimize prolonged business interruptions,

E. Minimize damage to public facilities and utilities, such as water and gas mains, electric, telephone, cable and wastewater lines, streets and bridges that are located in floodplains,

F. Help maintain a stable tax base by providing for the sound use and development of flood-prone areas in such a manner as to minimize future flood blight areas, and

G. Help potential buyers become aware of property that is subject to flooding.

2.4 Drainage Design Computations

The design factors, formulas, graphs and procedures presented or referred to herein are intended for use as engineering guides in the design of drainage facilities and in the solution of drainage problems involving the quantity, method of collection, transportation, and disposal of stormwater.

Methods of design other than those indicated or referred to herein may be considered in complex and difficult cases where experience clearly indicates they are preferable; however, these deviations shall not be attempted until approval has been obtained from the City Engineer.

The methods outlined or referred to herein, include accepted principles of surface drainage engineering and should be a working supplement to basic design information obtainable from textbooks and publications on drainage.

Some computer models that can be used to assist in this design are discussed in Appendix L. Other acceptable models are listed below. The latest available version should be used and the program application limitations shall be observed.

- InfoWorks (Innovyze Software)
- StormCAD (Bentley/Haestad Methods)
- XPSWMM and XPSTORM (XP-Solutions)
- Mike Urban (Danish Hydraulic Institute - DHI)

2.5 Easements and Construction of Drainage Facilities

A. All proposed storm drainage facilities (i.e. closed conduits, channels, graded swales, detention basins) which convey concentrated storm runoff beyond the boundary of a single property shall be placed within the limits of a dedicated drainage easement, stormwater management area, or public right-of-way adequate for maintenance purposes. Private storm drainage systems, which collect only on-site storm drainage runoff from one lot or tract, shall not be placed in a dedicated storm drainage easement. Easement width for storm drain conduits shall not be less than 15 feet and easement width for open channels shall be at least 20 feet wider than the width of the top of the channel banks. Where maintenance access is required, an additional drainage easement width of 15 feet beyond the channel top of bank shall be provided on one side.

   Public Maintenance – The City of Grand Prairie will provide maintenance, in accordance with the city’s current maintenance policies, of all public drainage facilities located within dedicated drainage easements and constructed to the City of Grand Prairie standards. Access to all public stormwater facilities shall be provided and dedicated to the City of Grand Prairie.

Private Maintenance:

- Private drainage facilities include those drainage improvements which are located on private property and are not in a dedicated drainage easement.
- Private drainage facilities may also include detention or retention basins, dams, and other stormwater controls which collect public water, as well as drainage ways which convey public water. Such facilities must be designed in accordance with City of Grand Prairie standards and reviewed, approved and inspected by the City.
- An agreement for perpetual maintenance of private drainage facilities serving public water shall be executed with the City prior to acceptance of the final plat and noted on the final plat. This agreement shall run with the land and can be tied to commercial property or to an owner's association, but not to individual residential lots.
- Access shall be provided by the developer/owner to all private drainage facilities where there may be a public safety concern for inspection by the City of Grand Prairie.

On lots or tracts where stormwater runoff has been collected or concentrated, it shall not be permitted to drain onto adjacent property except in existing creeks, channels, swales, or storm drains unless proper drainage easements or a letter of release of liability from the affected property owner, is filed for record with the County Deed of Records.

Channels delineated on the FEMA study and maps as adopted by UDC Article 15 and earthen channels accepted by the City as part of the development plan shall be placed within a dedicated drainage easement, stormwater management area, or public right-of-way of sufficient size to contain the 100-year (1% annual chance) fully developed flood with a minimum ten (10) feet overbank area within the floodplain on each side of the stream.

Retaining walls are not allowed in drainage easements, stormwater management areas, floodplains, floodways, or right-of-ways. Retaining walls shall be on private property (including the footings) and shall be the sole responsibility of the property owner. Retaining walls having a height greater than four (4) feet require plans to be sealed by a registered professional engineer in the State of Texas. City staff can require engineering documents for retaining walls with a height less than four (4) feet.

The sub-divider, developer, or builder shall provide and bear for the cost of all drainage improvements required for the development of such person's subdivision or other construction, including the cost of any necessary downstream off-site channels or storm drains as described in the UDC Article 14 and the cost for installation and acquisition of the required drainage easements, with the following exceptions:

- If the owner is unable to acquire the necessary off-site easements, such owner shall provide the City with documentation of such owner's efforts, including evidence of a reasonable offer made to the affected property owner. Upon such a written request for assistance, the City shall acquire these easements through either negotiations or condemnation. In either case, the cost of these easements shall be paid by the owner.

- In areas where the proposed off-site improvements are to be made within existing City right-of-way, an estimate of these off-site costs shall be prepared and submitted along with the plans. Subject to City Council approval, cost for such off-site improvements shall be prorated such that the owner pays for a percentage of the off-site cost based on the increase of the discharge originating within the limits of such owner's property.

All construction shall be in accordance with the standard specifications, current engineering standards, and construction details for street and drainage construction in the City.

In most cases, the City prefers that the floodplain be dedicated *fee simple* as a drainage easement. As a minimum, the floodplain (both FEMA and fully developed) shall be within a drainage easement adequate for maintenance purposes. A minimum 15 foot wide access road shall be provided within the ROW or easement at a grade not to exceed ten percent (10%).

If all required drainage easements and rights-of-way are not being dedicated by the plat, construction plans shall not be approved until any required drainage easement or ROW is conveyed to the City *fee simple* or conveyed as drainage easement by separate instrument. Stormwater management zones and drainage easements shall be maintained without buildings, fill or other obstructions to flood flows or loss of floodplain storage. Where possible, open space areas shall be preserved in an undeveloped, natural state to protect the natural beneficial functions of the floodplain.

B. Refer to the City's UDC Article 14 for details regarding the Texas Pollutant Discharge Elimination System (TPDES) General Permit for construction.

C. The City encourages the use of permanent, post-construction Best Management Practices (BMPs) to address stormwater quality through the North Central Texas Council of Governments (NCTCOG) *integrated* Stormwater Management (iSWM™) program. Guidance for Site Design Practices and post-construction BMPs can be found on NCTCOG's website. The link to the web site is given in **Appendix G** Site Design Practices, Part G.2 for Site Design Practices, and in the **Drainage Design Manual Volume 2**, **Appendix I** iSWM Stormwater Controls for Post Construction BMPs.

   Developers proposing permanent stormwater management facilities to remain in place after construction shall provide to the city an executed agreement from the property owner(s) on the city approved agreement form for filing with the county records against the property. The developer and property owner(s) shall agree to comply with the city guidelines and policies for operation and maintenance of the stormwater management facilities and provide periodic inspections of the condition of the stormwater management facilities as outlined in the guidelines and inspection form and submit to the city a copy of the inspection report on an annual basis.

D. Drainage System Classifications

   The four different types of basic drainage features are as follows:

   1. Closed systems, i.e., storm drains
   2. Reinforced concrete-lined open channels
   3. Earthen open channels
   4. Detention/Retention ponds

2.6 Drainage Plan Requirements

A. Drainage Area Determination

The size and shape of each watershed and associated sub-basins shall be determined for each drainage facility. This determination should be made using City topographic maps (or the most detailed topographic maps available if outside the City) with a scale of 1 inch=200 feet (1”=200’) or greater. Where the contour interval is insufficient or physical conditions may have changed from those shown on the City topographic maps, it may be necessary to supplement the maps with field topographic surveys. The actual conditions should always be verified by a reconnaissance survey.

The outline of drainage areas must follow natural drainage features in non-urbanized areas. Flow diverted by fence or agricultural ridge rows will require a detailed ground survey and rigorous hydraulic analyses for verification. If it cannot be determined that such diversions were constructed per City Code, or if they appear to have occurred by sedimentation along a fence, it will be necessary to design any downstream storm drainage systems to accommodate runoff from such areas.

Consideration shall be given to man-made features in urbanized areas. In preparing drainage maps particular attention should be given to gutter and ditch configurations at intersections. The direction of flow in gutters (on- and off-site) should be shown on the maps and construction plans.

The owner or developer of property to be developed shall be responsible for all storm drainage flowing on such person's property. This responsibility includes the drainage directed to that property by prior development as well as drainage naturally flowing through the property due to topography.

Adequate consideration shall be given by the owner in the development of property to determine how the discharge leaving the proposed development will affect downstream property, with the velocity of said downstream drainage not to exceed the values shown in *Table 8.1*.

B. General Requirements

1. No critical facility shall be placed in the 500-year (0.2% annual chance) floodplain.

2. All roads providing exclusive access to emergency responder facilities or to critical care facilities shall pass the 100-year (1% annual chance) fully developed watershed condition flood plus 2 feet of freeboard to the top of pavement elevation.

3. For residential development the minimum lot size of properties within, or partially within, the 100-year (1% annual chance) fully developed floodplain shall be one (1) acre.

4. The finished elevation of proposed streets shall be no less than two (2) feet above the 100-year (1% annual chance) existing developed flood elevation or one (1) foot above the 100-year (1% annual chance) fully developed flood elevation, whichever is higher unless specifically approved otherwise.

5. The system shall ensure drainage at all points along streets, and provide positive drainage away from buildings and on-site waste disposal areas. Residential building lowest floor elevations shall be no lower than 0.5 feet above the top of curb or a street center elevation, and the grading plan shall provide for positive drainage away from the buildings. Lot to lot surface drainage is prohibited except for Single Family Estate (SF-E) lots.

6. All work shall be performed using the Texas Plane Coordinate System 1983 Projections using the North American 1983 (NAD83) Datum (Referenced ellipsoid GRS 80) Texas North Central Zone. The coordinates of the beginning point of design and one (1) other point shall be provided on all proposed plats, site plans, and infrastructure plans.

7. Provide and reference vertical control benchmarks tied to two (2) City of Grand Prairie GPS Control points.

8. Calculations shown for each sub area including runoff coefficients, intensities, times of concentration, and runoff for Q_2, Q_{10}, and Q_{100} with summation at system junctions.

C. Grading Plans

A grading plan shall be prepared for all projects. The plan shall ensure proper drainage considerations to prevent adverse effects to adjoining properties and include:

1. A contour map of existing elevations based on field survey of the entire site, any off-site areas to be graded as a part of the project, and approximately 100 feet beyond the limits of the project or as needed to confirm the direction of local drainage. As a minimum the map shall have a scale of not less than one inch = 40 feet (1"=40') with a one-foot (1') contour interval. In certain cases, it may be necessary to adjust the map scale, reduce the contour interval, or extend the distance of the field survey beyond the project limits to fully characterize local drainage.

2. Site layout including lot lines, buildings or pads, finished floor elevations for buildings adjacent to streets and floodplains/floodways, paving, retaining walls, storm drainage features, FEMA and fully developed floodplains/floodways with elevations, water and wastewater facilities located in the floodplain, and any other structures that may influence drainage.

3. The plan shall present the proposed finished grades at one-foot (1') contour intervals. Spot grades shall be specified for retaining walls, to elaborate the detail on the plan and may be used on residential lots in lieu of contours.

4. The plan shall specify the base flood elevation (BFE) if the property is within 200-ft of the 100-year (1% annual chance) floodplain or detention basin.

5. The plan shall specify the lowest floor elevation (LFE) for all buildings. The LFE shall provide for positive drainage away from the buildings. Lots and property adjoining the 100-year (1% annual chance) floodplains may require LFE as specified by the City floodplain regulations in UDC Article 15.

6. Earthen grades for drainage being conveyed across the lot it originated on shall not be less than one percent (1.0%) and paved grades shall not be less than 0.5 percent (0.5%).

Maximum grades shall not exceed 25 percent (25%) without an engineering slope stability analysis.

7. Layouts and elevations of tops and bottoms of retaining walls shall be provided regardless of the height of the structures.

8. Structural design for retaining walls shall be provided. A separate building permit is required from Building Inspections for all retaining walls. Retaining walls having a height greater than four (4) feet require plans to be sealed by a registered professional engineer in the State of Texas. City staff can require engineering documents for retaining walls with a height less than four (4) feet.

9. Projects which involve modifications to an existing street intersection, or construction of a new street intersection shall require an intersection grading plan at a 0.2 foot contour interval as required by UDC Article 23.

As applicable, prior to release of a final building inspection, a licensed surveyor or engineer shall certify, refer to the Precise Grading Certificate in Appendix A, that lot grading is consistent with the City approved grading and drainage plans and that erosion control has been installed. Proper erosion control measures shall be shown on the plans and details provided.

D. Open Channels and Culvert/Bridge Plans

Plans shall be submitted for City approval prior to construction. The plans should typically be as per the Plan Checklist that is available on the City's website. The grading plan should include existing and proposed contours, fully developed floodplain limits with elevations, pre- and post-project FEMA floodplain and floodway, lowest floor elevation (LFE), drainage easements and right-of-ways. Refer to the City's UDC Article 15 for details regarding the floodplain design and map requirements.

Unless precluded by federal regulations, constructed or improved earthen channels of a permanent intent shall include a paved concrete flume invert with a width of at least 2 feet, an invert depth of at least 3 inches, a 12 inch to 18 inch wide 12 inch thick grouted rip rap on filter fabric border along the flume edges and at least 2 foot deep toe walls along the grouted rip rap edges as measured from the top of rip rap surface to provide erosion protection and ensure proper drainage.

E. Plat Requirements

If the property is being platted, the plat should depict the FEMA floodplain and floodway, the fully developed floodplain limits with elevations, lowest floor elevation (LFE) for lots adjacent to or within, in whole or in part, 200 feet the floodplain, drainage easements and stormwater management areas.

LFEs shall be the higher of two (2) feet above the FEMA base flood elevations or one (1) foot above fully developed flood elevations.

F. Erosion Hazard Setback

An erosion hazard setback zone determination is necessary for the banks of streams in which the natural channel is to be preserved as determined in individual creek studies. The purpose of the setbacks is to reduce the amount of structural damage caused by the erosion of the bank. With the application of streambank erosion hazard setbacks, an easement is dedicated to the City such that no structure can be located, constructed, or maintained in the area encompassing the erosion hazard setback.

Variations to the setback policy are allowed by the City only with the approval of the City Engineer. The City may allow for streambank stabilization as an alternative to dedicating the erosion hazard setback zone.

Streambank erosion hazard setbacks may be required to extend beyond the limits of the regulatory floodplain. The procedure for determining the streambank erosion hazard setback zone is as follows:

1. Locate the toe of the natural stream bank.
2. From this toe, construct a line sloping at 4 horizontal to 1 vertical towards the bank until it intersects natural ground.
3. From this intersection, add 10 feet in the direction away from the stream to locate the outer edge of the erosion hazard setback.

The erosion hazard setback area may be reduced in places where the streambanks are composed partially or entirely of rock. In these areas, the interface of the natural streambank with the top of the unweathered rock strata should be located with the assistance of a qualified geotechnical engineer or geologist. From this point, a line sloping at 3 horizontal to 1 vertical is constructed until its intersection with natural ground. The erosion hazard setback is located 10 feet in the direction away from the stream from this intersection.

G. Landscaping for Drainage Facilities

Species of vegetation selected for landscape plans should be adopted to local climatic conditions and soils to be encountered on the site. Drought resistant vegetation is recommended for typical sites. Further information and guidance is provided in the City of Grand Prairie UDC Article 8 Landscaping and in Appendix L of this manual.

Projects shall meet all City, state and federal requirements.

2.7 Trinity River Corridor Development Certificate (CDC)

The Trinity River Corridor is defined as the bed and banks of the river segments from the dams of Lake Lewisville, Grapevine Lake, Lake Worth, Benbrook Lake, Lake Arlington, and Mountain Creek Lake, downstream to the area near Post Oak Road and the Trinity River in southeast Dallas County, and all of the adjacent land area and all watercourses contained within the boundaries of the river floodplain as designated by the CDC Steering Committee. The Regulatory Zone of the Corridor Development Certificate (CDC) is the FEMA 100-year (1% annual chance) regulatory floodplain of the Trinity River Corridor, minus areas of Specific Prior Development, produced from the Clear Fork, West Fork, Elm Fork, and main stem of the Trinity River. The outer boundary of the Regulatory Zone within the tributaries, such as Mountain Creek, is determined from the backwater effect from the main stem of the Trinity River.

Projects within the CDC Regulatory Zone must meet additional requirements as set forth in the current edition of the Corridor Development Certificate Manual. An application for a Corridor Development Certificate must be prepared and submitted to the Floodplain Administrator. Additional information regarding the CDC application process is included in UDC Article 15.

3.0 DESIGN RAINFALL AND LOSS METHODOLOGIES

The Rational method (Q=CIA) shall be used for determining the design discharge on small watersheds of 200 acres or less. The modified rational method may be used for sizing detention basins draining watersheds of 25 acres or less. Unit hydrograph techniques shall be used for all other watersheds and for sizing detention basins draining more than 25 acres. See *Section 4.0* for design rainfall determination.

Rainfall intensity at a design point may be calculated as:

$$I = \frac{b}{(T_c + d)^e}$$

where:

I = Rainfall intensity, in/hr;
T_c = Time of concentration, minutes (min); and
b, d, and e = Constants for the design frequency storm.

Values for the rainfall intensity constants b, d, and e, are specified in Table 3.0A. Table 3.0B lists the intensities for minimum inlet times and varying design storms.

Table 3.0A
Rainfall Intensity Constants

Rainfall Frequency	1-Year	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
b	44	54	68	78	90	101	106
d	8.7	8.3	8.7	8.7	8.7	8.7	8.3
e	0.796	0.791	0.782	0.777	0.774	0.771	0.762

Table 3.0B
Rainfall Intensity Table

Minimum Time of Concentration	Rainfall Frequency						
	1-Year	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
5	5.478	6.973	8.782	10.206	11.869	13.425	14.755
10	4.276	5.417	6.885	8.014	9.329	10.562	11.57
15	3.541	4.475	5.721	6.667	7.766	8.798	9.625
20	3.041	3.837	4.925	5.745	6.696	7.591	8.299

3.1 Rainfall Excess

Regardless of which methodology is used precipitation losses occur due to evaporation, interception, depression storage and infiltration. The losses are evaluated and subtracted from the total rainfall amount to determine the rainfall *excess.* The rainfall excess is the portion of the rainfall that reaches the storm drainage system. Rainfall used for analysis and design shall the Synthetic Method discussed in

Rainfall excess shall be determined by one of these two loss methodologies: Curve Number (*CN*) loss model or initial and constant-rate loss model, and shall at a minimum, comply with FEMA *Guidelines and Specifications for Flood Hazard Mapping Partners*. The *CN* approach shall be used in conjunction with the SCS Dimensionless Unit Hydrograph technique (Section 4.5) and the initial and constant-rate method shall be used with the Snyder's Unit Hydrograph technique (Section 4.6).

The types of soils in a sub-basin are determined from the latest versions of the *Soil Survey of Dallas County, Texas; Soil Survey Ellis County, Texas; Soil Survey of Johnson County, Texas; and Soil Survey of Tarrant County, Texas.* Soils in Hydrologic Soil Groups A and B are sandy soils, while soils in Hydrologic Groups C and D are clay soils. Copies of this information are available at the office of the City Engineer or online.

Electronic soils data in the Soil Survey Geographic (SSURGO) Database can be obtained free of charge from the National Resource Conservation Service (NRCS) at http://soildatamart.nrcs.usda.gov. The data is downloadable by county and includes extensive soil information, including hydrologic soil groups. The data is intended to be imported into a geographic information system (GIS) to allow for site-specific soil analysis of soil characteristics for storm design. All soil survey results can also be accessed online at http://websoilsurvey.nrcs.usda.gov/app/. Maps can be created and printed from this site without the use of a GIS.

Curve Number (CN)

For the Curve Number method a single parameter, *CN*, is used to evaluate the loss rate. Table 3.1A contains *CN* values based on type of soil and type of land use. If the *CN* value varies for a sub-basin, an area-weighted *CN* value must be determined.

The percent imperviousness of the sub-basin is included in the *CN* values for urban and residential districts. For all other land uses the percent impervious shall be determined by the engineer and applied to the base curve number found in Table 3.1A. For sub-basins with more than one land use, an area-weighted percent impervious is to be used.

Initial and Constant-Rate

Three parameters, an initial loss, a constant loss rate and a percent impervious, are used in the initial and constant-rate method to compute losses. In areas assumed to be pervious all rainfall is lost until the volume of the initial loss is satisfied then rainfall is lost at the constant rate for the remainder of the hyetograph. In areas assumed to be impervious no losses occur.

The initial and constant-rate parameters are determined based on the type of soil (i.e., clay or sand) in the sub-basin. When the sub-basin has both clay and sandy soils area-weighted values should be used for the initial loss and constant loss rate parameters. This method is most appropriate when calibration data is available but previous investigations can be used to estimate parameters. In absence of gage data or previous investigations the default values in Southwest Fort Worth Hydrology (SWFHYD or NUDALLAS) can be used. Those values can be found in Table 3.1B. If values other than the recommended ones are used, documentation is required justifying their use.

Table 3.1.a Runoff curve numbers for urban areas [1]

Cover description: Cover type and hydrologic condition	Average percent impervious area [2]	Curve numbers for hydrologic soil group: A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.) [3]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only) [4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
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					
Newly graded areas (pervious areas only, no vegetation) [5]		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c).					

1 Average runoff condition, and $I_a = 0.2S$.

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 CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.

4 Composite CN's for natural desert landscaping should be computed using figures 2-3 or 2-4 based on the impervious area percentage (CN = 98) and the pervious area CN. The pervious area CN's are assumed equivalent to desert shrub in poor hydrologic condition.

5 Composite CN's to use for the design of temporary measures during grading and construction should be computed using figure 2-3 or 2-4 based on the degree of development (impervious area percentage) and the CN's for the newly graded pervious areas.

Table 3.1B
Recommended Initial and Constant-Rate Losses

	Sand (Group A & B)		Clay (Group C & D)	
Storm	**Initial**	**Constant**	**Initial**	**Constant**
1 year	2.10	0.26	1.50	0.20
2 year	2.10	0.26	1.50	0.20
5 year	1.80	0.21	1.30	0.16
10 year	1.50	0.18	1.12	0.14
25 year	1.30	0.15	0.95	0.12
50 year	1.10	0.13	0.84	0.10
100 year	0.90	0.10	0.75	0.07
500 year	0.60	0.08	0.50	0.05
SPF	0.60	0.08	0.50	0.05

The percent impervious input represents the percentage of the sub-basin covered by impervious cover such as streets, parking lots and structures. All precipitation on impervious areas is considered excess and unlike the *CN* technique the impervious areas must be accounted for independently of the pervious land uses. Recommended percent impervious values can be found in Table 3.1C below.

Table 3.1C
Recommended Percent Impervious Values

Land Use	Approximate Percent Impervious
Residential	
1/8 Acre	65%
1/4 Acre	38%
1/3 Acre	30%
1/2 Acre	25%
1 Acre	20%
2 Acre	12%
Commercial	85%
Industrial	72%

4.0 DETERMINATION OF DESIGN DISCHARGE

All Hydrology models shall at a minimum, comply with FEMA *Guidelines and Specifications for Flood Hazard Mapping Partners.*

4.1 Design Frequencies

It is general practice to design municipal storm drainage systems to accommodate the runoff from 10-year or 100-year floods that have a ten percent (10%) or one percent (1%) chance of being equaled or exceeded in any given year, respectively.

Table 4.1 shows the flood frequencies to be used in the design of drainage facilities:

Table 4.1
Design Flood

Type of Facility	Design Flood
Storm drains (with inlets on grade)	10 year
Storm drains draining low point inlets	100 year
Culverts, bridges, channels, creeks, low point overflows[1]	100 year

[1] 100-year (1% annual chance) flood. Low point overflows must be contained within drainage easements. If low point overflows are in parking lots or access drives depth must not exceed 6 inches and must be contained in a dedicated drainage easement.

4.2 Computation Methods

The Rational method (Q=1.00833CIA) shall be used for determining the design discharge on small watersheds of 200 acres or less. Unit hydrograph techniques shall be used for areas greater than 200 acres. The technique and the data to be used for the determination of the design discharge shall be approved by the City Engineer prior to the calculations being completed. A complete set of all detail calculations must be submitted to the City Engineer for approval prior to the completion of the plans for the drainage system.

4.3 Rational Method

The formula for calculation of the peak flow rate by the rational method is:

$$Q = 1.00833 C\, I\, A$$

where:

Q = Peak flow, cubic feet per second (cfs);
C = Runoff coefficient;
A = Sub-basin area, acres (ac); and
I = Rainfall intensity, inches per hour (in/hr).
1.00833 = the unit conversion factor

Runoff coefficients for use in the rational formula are presented Table 4.3A as a function of land use. For sub-basins with more than one land use, an area-weighted runoff coefficient is to be used which uses the ultimate developed runoff coefficients for all land uses in the sub-basins. The undeveloped runoff coefficient for general undeveloped portions of the sub-basin shall not be included in the area-weighted runoff coefficient determination unless these undeveloped areas are being dedicated as open space areas on the plat or instrument of dedication.

A. Runoff Coefficients

Storm drainage shall be designed for ultimate development of the watershed and, therefore, runoff coefficients used shall consider these fully developed conditions. Master plans, zoning maps, land use plans and the Unified Development Code shall be used to determine the ultimate development. Table 4.3A gives values for runoff coefficients that shall be used in the determination of stormwater runoff.

Table 4.3A
Runoff Coefficient "C"

Type of Area or Land Use	Zoning Class*	Runoff Coefficient "C"
Parks and permanent open space	A, ESMNT	0.30
Single-family residential	SFE, SF1, SF118, SF2, SF216, SF218, SF3, SF316, SF318, SF4, SF5, SF516, SF6	0.50
Multi-family residential and schools	MH, MF1, MF2, MF3, SFA, SFZLL, SFT, 2F	0.75
Commercial/retail	O, O-1, NS, C-1, CO, C, PD, HD, GR, GR1	0.80
Industrial and manufacturing	HC, IP, LI, HI	0.85
Central Business District (CBD)	CA	0.90
Church	All Zoning Classes	0.75

* See UDC Article 3 for Zoning Class descriptions

B. Time of Concentration

The time of concentration (T_c) is the longest time of travel for water to flow from the upstream portion of the sub-basin to the downstream point of design. Typical site conditions will dictate that T_c is the minimum time to inlet per Table 4.3D. In special cases, T_c in excess of those presented in Table 4.3D may be calculated with the following procedure and such calculations and flow paths should be included with the data submitted for review by the City. Following the procedures specified herein, T_c can be computed with NRCS *TR-55, Urban Hydrology for Small Watersheds*.

In using the calculated procedure for determining T_c the following issues shall be considered. First, care shall be taken to ensure that the longest time of travel chosen is characteristic of the overall drainage within the sub-basin. Second, the interface between overland flow and shallow concentrated flow shall be carefully evaluated considering shallow concentrated flow paths on lawns, in swales, between structures, etc.

T_c is composed of four basic components, overland flow, shallow concentrated flow, channelized flow to inlet, and channelized flow downstream of the inlet to the point of design. Either this method or the minimum time to inlet must be used when determining T_c downstream of an inlet. Time of concentration at a design point is calculated as:

$$T_c = T_0 + T_s + T_h + T_t$$

where:
- T_c = Time of concentration, minutes (min);
- T_0 = Overland flow travel time, min;
- T_s = Shallow concentrated flow travel time, min;
- T_h = Channelized flow travel time to inlet, min; and
- T_t = Channelized flow time of travel downstream of inlet to the design point, min.

Overland Flow

The time of travel for the overland flow component (T_0) is computed using Manning's kinematic solution:

$$T_0 = 0.42 \frac{(nL)^{0.8}}{{R_2}^{0.5} S^{0.4}}$$

where:
- T_0 = Overland flow time of travel, min;
- n = Manning's coefficient for sheet flow;
- L = Flow length, feet (ft);
- R_2 = 4.0 inches which is the 2-year, 24-hour rainfall; and
- S = Slope of the hydraulic grade (assume it is equal the ground slope), ft/ft.

Manning's coefficient (n) for overland flow is based on soil cover. Values for n are presented in Table 4.3B. Overland flow length (L) is based on City topographic maps (or more detailed site survey data) for pre-project conditions and proposed grading plan for post project conditions. L shall not exceed the lengths presented in Table 4.3C. Larger L values for undeveloped and agricultural land use can be used for undeveloped pre-project conditions. The 300 feet maximum is set because after that distance, the flow is usually considered shallow concentrated flow.

Table 4.3B
Manning's *n* for Overland Flow

Soil Cover	*n* Value
Undeveloped - Cultivated soil, dense grass, range, or woods	0.24 - 0.410
Developed - Lawns, dense grass, or woods	0.240
Concrete, asphalt, gravel, or bare soil	0.011

Shallow Concentrated Flow

Overland flow becomes shallow concentrated flow in reels, shallow gullies, or swales, such as those between houses or businesses. Such flow in undeveloped areas extends from the overland flow to a stream as defined on a City topographic map or the most detailed topographic maps available (if it is outside the City). In developed areas, shallow concentrated flow extends from the end of overland flow to the curb or street ditch or swale. Flow in a gutter shall be treated as channelized flow. Areas with shallow concentrated flow with varying slopes or soil surfaces can be broken down into segments to better estimate the travel time. The total time of travel of the shallow concentrated flow is the sum of the times of travel for each segment.

Table 4.3C
Maximum Overland Flow Lengths

Land Use	Maximum *L* (ft)
Undeveloped, agricultural*****	100**
Parks, permanent open space, playgrounds	60
Single family residential (less than 3 lots per acre)	50
Single family residential, schools	40
Multi-family residential, commercial, industrial, manufacturing	20
Central business district (CBD), strip centers	10

* This length is a minimum, unless there is a defined stream on City topographic maps. An undeveloped site can assume a minimum time of concentration at 20 minutes with a run-off coefficient of 0.30.

** Revised maximum allowable, an exception to TR-55 which allows up to 300 feet for undeveloped agricultural areas.

Shallow concentrated flow is characterized by the soil cover as either paved or unpaved. The flow velocity is calculated using the following formula:

$$V_s = KS^{0.5}$$

where:
- V_s = Average velocity of flow, fps;
- K = 16.1 for unpaved and 20.3 for paved soil cover; and
- S = Slope of the watercourse, ft/ft.

The time of travel for shallow concentrated flow is calculated as:

$$T_s = \frac{L}{60V_s}$$

where:
- T_s = Shallow concentrated flow travel time, min;
- L = Flow length, ft; and

V_s = Average velocity of flow, fps.

Channelized Flow

Channelized flow is drainage in gutters, storm drains, channels, and streams. Generally, in the analysis of channelized flow it is necessary to breakdown the flow into a series of reaches, each reach having its own characteristics, to better estimate the travel time. The total time of travel of the channelized flow is the sum of the times of travel for each segment. Flow velocities are calculated using the Manning equation with Q_p for the 2-year (50% annual chance) flood.

For natural and constructed channels the velocity (V_h) may be calculated by assuming uniform bank full flow. For street gutters the following should be used to estimate gutter velocities (V_h) for 2-year storm gutter flow:

All gutter flow paths shall be considered channelized flow. For future fully developed conditions, it shall be assumed that street and alley ROWs are paved. Typical residential streets shall be assumed to be 27-foot wide, back of curb to back of curb, parabolic street sections with 6-inch curbs and 5-inch crowns. Typical minor arterial streets shall be assumed to be 45-foot wide, back of curb to back of curb, parabolic street sections with 6-inch curbs and 6-inch crowns. Roof top section (triangular) streets shall be assumed to have a ¼ inch per foot cross fall and typically are used for divided arterials. Roof top sections for divided streets are assumed to be 25-foot wide or greater, measured back of curb to back of curb, with 6-inch curbs.
The average flow velocity used for channelized gutter flow time of concentration calculations shall be determined using the following formula:

$V_h = F*S^{0.5}$

Where:
V_h = Average velocity of flow, fps
S = Average street grade, ft/ft
F = Flow Factor taken from the following table:

Street Section Type and Width	Flow Factor
Parabolic 27-ft	24.4
Parabolic 31-ft	24.5
Parabolic 37-ft	28.8
Parabolic 45-ft	28.9
Roof Top (triangular) 25-ft	32.2

Flow factors are based on 5-inch flow depths in the street gutter on one side of the street and a Manning's "n" value of 0.016.

For closed conduit systems on flat grades not being hydraulically analyzed for the project, it may be reasonable to calculate V_h assuming uniform half-full flow. After computing the velocity(s), the time of travel for channelized flow for each channelized segment is calculated with the following equation:

$$T_h = \frac{L}{60V_h}$$

where:
T_h = Channelized flow travel time, min;
L = Flow length, ft; and
V_h = Average velocity of flow, fps.

The total time of travel for channelized flow is the sum of the calculated travel times for each channelized flow segment.

Flow through ponds or lakes and where the calculated velocity for channelized flow for post project conditions is less than 3 fps, then the flow should be assumed to travel at wave celerity:

$$T_h = c = (g\ d_m)^{0.5}$$

where:
c = Wave celerity, fps;
g = 32.2 = Acceleration of gravity, feet per second per second (ft/sec^2); and
d_m = Average depth of flow, ft.

Time to Inlet

The time to inlet is the time of travel of the water flow to the inlet considering overland flow, shallow concentrated flow, and channelized flow. Minimum times of travel to the inlet are specified in Table 4.3D. These minimum times to inlet may be used for T_c at inlets in lieu of calculating T_c for post project conditions. However, the calculated time to inlet shall be used when determining T_c downstream of an inlet.

For undeveloped pre-project conditions, T_c shall always be calculated and overland flow shall be assumed to occur for the first 300 feet of flow, unless there is a defined stream depicted on City topographic maps. If the calculated T_c is less than 20 minutes, then the 20-minute minimum time to inlet shall apply. This 20-minute minimum time to inlet shall only be used for undeveloped pre-project conditions.

Time of Travel

T_c for design points downstream of inlets shall be calculated using the time to inlet (i.e., the calculated T_c to inlet or the minimum times to inlet from the Table 4.3D) plus the time of travel (T_t) of the flow through the channelized flow segments downstream of the inlet. For small drainage systems with short times of travel, the channelized flow segments downstream of the inlet for post project conditions may be neglected for design purposes. Time of travel (T_t) downstream of inlets shall be computed using the hydraulic procedures as previously specified for channelized flow (T_h).

Table 4.3D
Minimum Times to Inlet

Land Use	Minimum Time to Inlet (min)
Undeveloped, agricultural	20
Parks, permanent open space, playgrounds	15
Single family residential	15
Multi-family residential*****, schools, commercial, industrial, manufacturing, business, church	10
Central business district (CBD)	5

* Includes zoning classes: MH, SFA, SFZLL, SFT, 2F

4.4 Rainfall Depth and Distribution for Unit Hydrograph Methods

The point depth duration data given in Table 4.4 shall be used for all hydrologic analysis methods using the unit hydrograph method. A synthetic storm time distribution (hyetograph) shall be developed based on the point depth-duration data of Table 4.4 with the recommended areal adjustment factors used per Figure 15 in Weather Bureau Technical Paper No. 40 for models of drainage area sizes greater than 10 square miles.

The distribution shall be symmetrical about the center time of the total storm duration (12 hour point for the 24-hour storm). For drainage studies of watersheds the 24-hour total storm duration with a minimum time step of 15-minutes shall be used with all the depths for the duration of Table 4.4 incorporated into the distribution. For studies of watersheds of 200 acres in drainage area or less a 3-hour total storm duration may be used for all sub-basins. For the 3-hour storm duration the distribution shall be symmetrical about the center time of the total storm duration (1.5 hour point) with a minimum time step of 5-minutes and with all the depths for the duration of Table 4.4 for 3 hours and less incorporated into the distribution.

Table 4.4
Depth-Duration Data [A]

Return Period (years)	Point Rainfall Depths (inches)							
	5-min	15-min	1-hr	2-hr	3-hr	6-hr	12-hr	24-hr
1	0.39	0.76	1.49	1.81	1.99	2.41	2.80	3.21
2	0.49	1.04	1.85	2.22	2.45	2.91	3.45	3.95
5	0.57	1.22	2.45	3.00	3.30	3.90	4.70	5.40
10	0.63	1.36	2.86	3.55	3.85	4.65	5.50	6.40
25	0.73	1.56	3.35	4.15	4.55	5.45	6.50	7.50
50	0.80	1.71	3.82	4.65	5.15	6.20	7.35	8.52
100	0.87	1.87	4.25	5.20	5.70	6.92	8.40	9.55
500	1.00	2.20	5.40	6.60	7.40	8.80	10.50	12.00

4.5 SCS Hydrologic Method

The Soil Conservation Service (SCS) hydrologic method requires basic data similar to the Rational Method: drainage area, a runoff factor, time of concentration, and rainfall. The SCS approach, however, is more sophisticated in that it also considers the time distribution of the rainfall, the initial rainfall losses to interception and depression storage, and an infiltration rate that decreases during the course of a storm. Details of the methodology can be found in the SCS National Engineering Handbook, Section 4, Hydrology.

A typical application of the SCS method includes the following basic steps:

1. Determination of curve numbers that represent different land uses within the drainage area.

2. Calculation of time of concentration to the study point.

3. Using the Type II rainfall distribution, total and excess rainfall amounts are determined. Note: See Figure 4.5A for the geographic boundaries for the different SCS rainfall distributions. **For city applications using the SCS Hydrologic Method, the rainfall distribution discussed in section 4.4 Rainfall Depth and Distribution for Unit Hydrograph Methods shall be used in place of the SCS Type II rainfall distribution.**

4. Using the unit hydrograph approach, the hydrograph of direct runoff from the drainage basin can be developed.

Application

The SCS method can be used for both the estimation of stormwater runoff peak rates and the generation of hydrographs for the routing of stormwater flows. The Simplified SCS Peak Runoff Rate estimation method can be used for drainage areas up to 2,000 acres. Thus, the SCS method can be used for most design applications, including storage facilities and outlet structures, storm drain systems, culverts, small drainage ditches, open channels, and energy dissipaters.

Equations and Concepts

The hydrograph of outflow from a drainage basin is the sum of the elemental hydrographs from all the sub-areas of the basin, modified by the effects of transit time through the basin and storage in the stream channels. Since the physical characteristics of the basin including shape, size, and slope are constant, the unit hydrograph approach assumes there is considerable similarity in the shape of hydrographs from storms of similar rainfall characteristics. Thus, the unit hydrograph is a typical hydrograph for the basin with a runoff volume under the hydrograph equal to one (1.0) inch from a storm of specified duration. For a storm of the same duration but with a different amount of runoff, the hydrograph of direct runoff can be expected to have the same time base as the unit hydrograph and ordinates of flow proportional to the runoff volume. Therefore, a storm that produces two (2) inches of runoff would have a hydrograph with a flow equal to twice the flow of the unit hydrograph. With 0.5 inches of runoff, the flow of the hydrograph would be one-half of the flow of the unit hydrograph.

The following discussion outlines the equations and basic concepts used in the SCS method.

Drainage Area - The drainage area of a watershed is determined from topographic maps and field surveys. For large drainage areas it might be necessary to divide the area into sub-drainage areas to account for major land use changes, obtain analysis results at different points within the drainage area, combine hydrographs from different sub-basins as applicable, and/or route flows to points of interest.

Rainfall - The SCS method applicable to North Central Texas is based on a storm event that has a Type II time distribution. This distribution is used to distribute the 24-hour volume of rainfall for the different storm frequencies (Figure 4.5A).

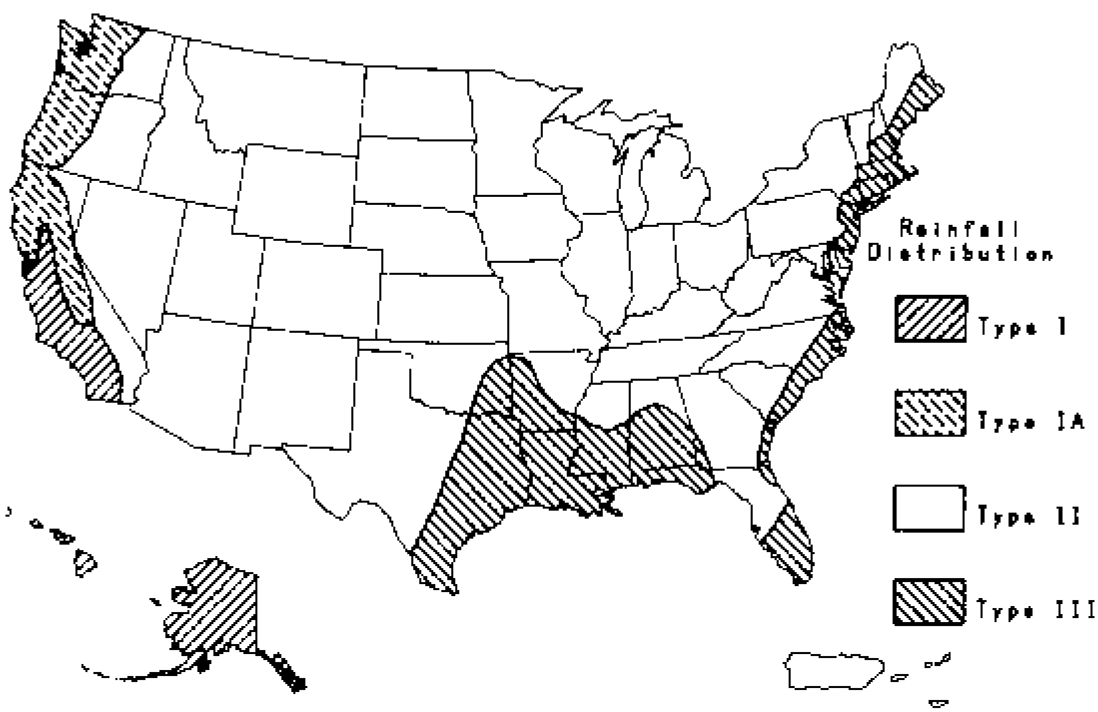


Figure 4.5A
Approximate Geographic Boundaries for SCS Rainfall Distributions

Rainfall-Runoff Equation - A relationship between accumulated rainfall and accumulated runoff was derived by SCS from experimental plots for numerous soils and vegetative cover conditions. The following SCS runoff equation is used to estimate direct runoff from the 24-hour storm rainfall. The equation is:

$$Q = (P - I_a)2 / [(P - I_a) + S]$$

where:

Q = accumulated direct runoff (in)
P = accumulated rainfall (potential maximum runoff) (in)
I_a = initial abstraction including surface storage, interception, evaporation, and infiltration prior to runoff (in)
S = 1000/CN - 10 where CN = SCS curve number

An empirical relationship used in the SCS method for estimating I_a is:

$$I_a = 0.2S$$

This is an average value that could be adjusted for flatter areas with more depressions if there are calibration data to substantiate the adjustment. Table 4.5A provides values of I_a for a wide range of curve numbers (CN).

Substituting $0.2S$ for I_a, the equation becomes:

$$Q = (P - 0.2S)^2 / (P + 0.8S)$$

Figure 4.5B shows a graphical solution of this equation. For example, 4.1 inches of direct runoff would result if 5.8 inches of rainfall occurred on a watershed with a curve number of 85. The curve number can be estimated if rainfall and runoff volume are known with the following equation (Pitt, 1994):

$$CN = 1000/[10 + 5P + 10Q – 10(Q^2 + 1.25QP)^{1/2}]$$

Table 4.5A
***I_a* Values for Runoff Curve Numbers**

Curve Number	I_a (in)	Curve Number	I_a (in)
40	3.000	70	0.857
41	2.878	71	0.817
42	2.762	72	0.778
43	2.651	73	0.740
44	2.545	74	0.703
45	2.444	75	0.667
46	2.348	76	0.632
47	2.255	77	0.597
48	2.167	78	0.564

Table 4.5A Continued
I_a Values for Runoff Curve Numbers

Curve Number	I_a (in)	Curve Number	I_a (in)
49	2.082	79	0.532
50	2.000	80	0.500
51	1.922	81	0.469
52	1.846	82	0.439
53	1.74	83	0.410
54	1.704	84	0.381
55	1.636	85	0.353
56	1.571	86	0.326
57	1.509	87	0.299
58	1.448	88	0.273
59	1.390	89	0.247
60	1.333	90	0.222
61	1.279	91	0.198
62	1.226	92	0.174
63	1.175	93	0.151
64	1.125	94	0.128
65	1.077	95	0.105
66	1.030	96	0.083
67	0.985	97	0.062
68	0.941	98	0.041
69	0.899		

Source: SCS, TR-55, Second Edition, June 1986

Travel Time Estimation

Travel time (T_t) is the time it takes water to travel from one location to another within a watershed, through the various components of the drainage system. Time of concentration (t_c) is computed by summing all the travel times for consecutive components of the drainage conveyance system from the hydraulically most distant point of the watershed to the point of interest within the watershed. Following is a discussion of related procedures and equations (USDA, 1986).

Travel Time

Water moves through a watershed as sheet flow, shallow concentrated flow, open channel flow, or some combination of these. The type of flow that occurs is a function of the conveyance system and is best determined by field inspection.

Travel time is the ratio of flow length to flow velocity:

$$T_t = L/3600V$$

where:

T_t	=	travel time (hr)
L	=	flow length (ft)
V	=	average velocity (ft/s)
3600	=	conversion factor from seconds to hours

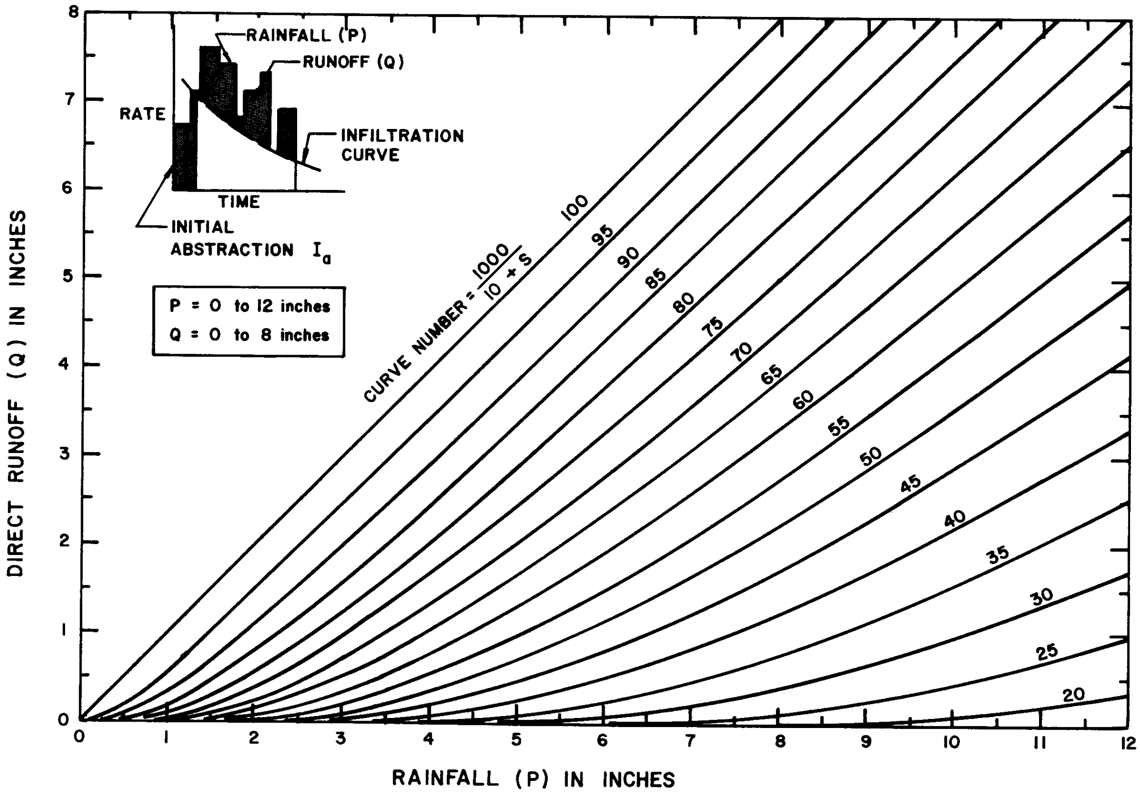


Figure 4.5B
SCS Solution of the Runoff Equation
(Source: SCS, TR-55, Second Edition, June 1986)

Sheet Flow

Sheet flow can be calculated using the following formula:

$$T_t = \frac{0.42\,(nL)^{0.8}}{60\,(P_2)^{0.5}(S)^{0.4}} = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5}(S)^{0.4}}$$

where:

T_t	=	travel time (hr)
n	=	Manning roughness coefficient (see Table 4.5B)
L	=	flow length (ft),
P_2	=	2-year, 24-hour rainfall
S	=	land slope (ft/ft)

Table 4.5B
Roughness Coefficients (Manning's n) for Sheet Flow[1]

Surface Description	n
Smooth surfaces	
(concrete, asphalt, gravel or bare soil)	0.011
Fallow	
(no residue)	0.05
Cultivated soils:	
Residue cover < 20%	0.06
Residue cover > 20%	0.17
Grass:	
Short grass prairie	0.15
Dense grasses[2]	0.24
Bermuda grass	0.41
Range	
(natural)	0.13
Woods[3]	
Light underbrush	0.40
Dense underbrush	0.80

[1] The n values are a composite of information by Engman (1986).
[2] Includes species such as bluestem grass, buffalo grass, grama grass, and native grass mixtures.
[3] When selecting n, consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.
Source: SCS, TR-55, Second Edition, June 1986.

Shallow Concentrated Flow

After 50 to 100 feet, sheet flow usually becomes shallow concentrated flow. The average velocity for this flow can be determined from Figure 4.5C, in which average velocity is a function of watercourse slope and type of channel.

Average velocities for estimating travel time for shallow concentrated flow can be computed from using Figure 4.5C, or the following equations. These equations can also be used for slopes less than 0.005 ft/ft.

Unpaved $V = 16.13(S)^{0.5}$
Paved $V = 20.33(S)^{0.5}$

where:

V = average velocity (ft/s)
S = slope of hydraulic grade line (watercourse slope, ft/ft)

In developed areas, shallow concentrated flow extends from the end of overland flow to the curb or street ditch or swale. Flow in a gutter or ditch shall be treated as channelized flow. (See Drainage Design Manual Section 4.3B **Shallow Concentrated Flow**)

After determining average velocity using Figure 4.5C or equations shallow concentrated flow equations above, use travel time equation above to estimate travel time for the shallow concentrated flow segment.

Open Channels

The provisions discussed in Design Manual Section 4.3B **Channelized Flow** shall be used in modeling channelized flow and determining the total channelized flow time.

Velocity in channels should be calculated from the Manning equation. Open channels are assumed to begin where surveyed cross section information has been obtained, where channels are visible on aerial photographs, where channels have been identified by the local municipality, or where stream designations appear on United States Geological Survey (USGS) quadrangle sheets. Manning's equation or water surface profile information can be used to estimate average flow velocity. Average flow velocity for travel time calculations is usually determined for bank-full elevation assuming low vegetation winter conditions.

Manning's equation is:

$$V = (1.486/n)\,(R)^{2/3}\,(S)^{1/2}$$

where:

V	=	average velocity (ft/s)
R	=	hydraulic radius (ft) and is equal to A/P_w
A	=	cross sectional flow area (ft^2)
P_w	=	wetted perimeter (ft)
S	=	slope of the hydraulic grade line (ft/ft)
n	=	Manning's roughness coefficient for open channel flow

After average velocity is computed using Manning’s equation, T_t for the channel segment can be estimated using the travel time equation.

Limitations

1. Equations in this section should not be used for sheet flow longer than 50 feet for impervious surfaces.
2. In watersheds with storm drains, carefully identify the appropriate hydraulic flow path to estimate T_c.
3. A culvert or bridge can act as detention structure if there is significant storage behind it. Detailed storage routing procedures should be used to determine the outflow through the culvert or bridge.

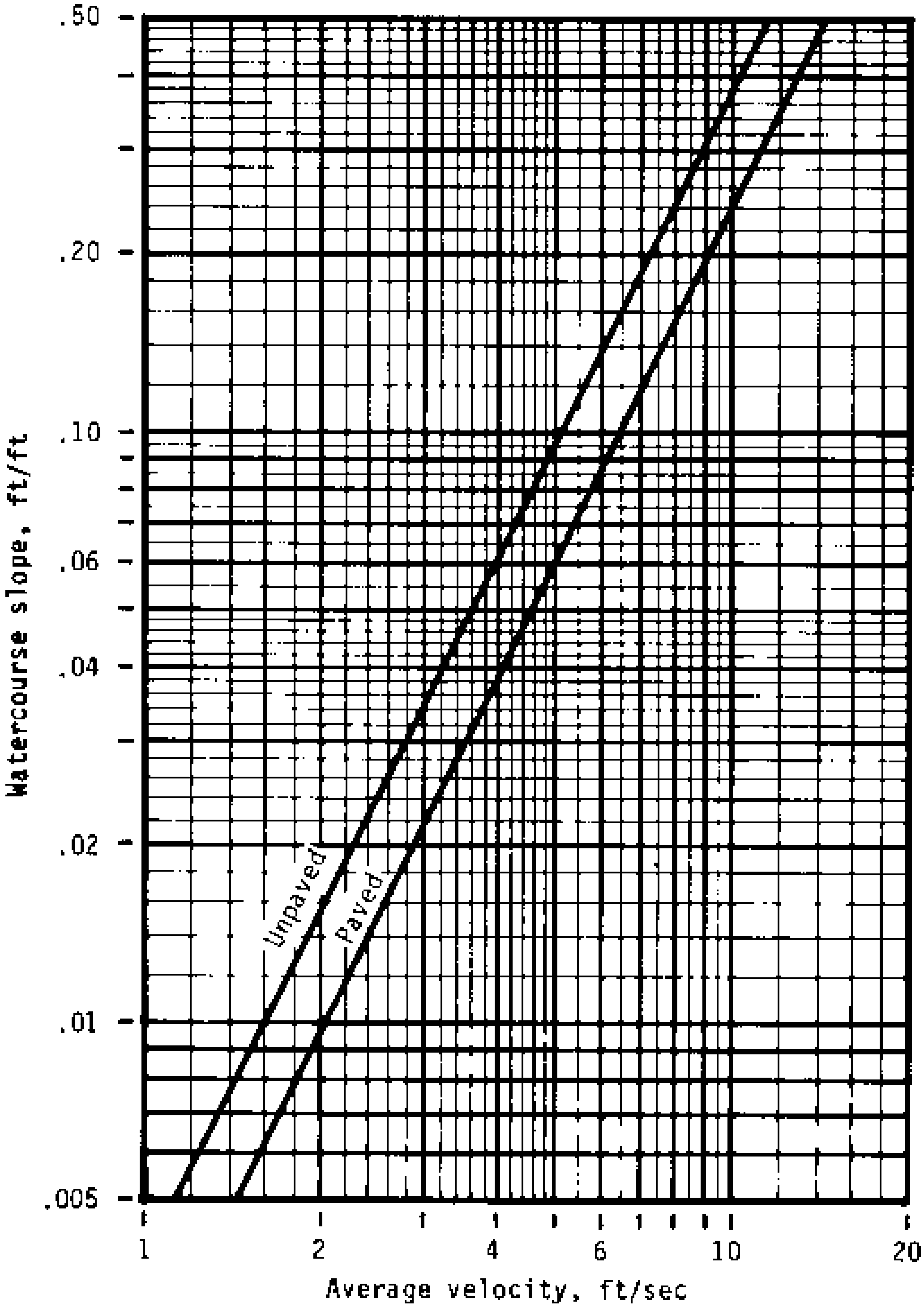


Figure 4.5C
Average Velocities - Shallow Concentrated Flow
(Source: SCS, TR-55, Second Edition, June 1986)

Simplified SCS Peak Runoff Rate Estimation

The following SCS procedures were taken from the SCS Technical Release 55 (USDA, 1986) which presents simplified procedures to calculate storm runoff volume and peak rate of discharges. These procedures are applicable to small drainage areas (typically less than 2,000 acres) with homogeneous land uses, which can be described by a single CN value. The peak discharge equation is:

$$Q_p = q_u A Q F_p$$

where:

Q_p = peak discharge (cfs)
q_u = unit peak discharge (cfs/mi^2/in)
A = drainage area (mi^2)
Q = runoff (in)
F_p = pond and swamp adjustment factor

Computations for the peak discharge method proceed as follows:

1. The 24-hour rainfall depth (P) is determined from the rainfall tables in Appendix A for the selected location and return frequency.
2. The runoff curve number, CN, is estimated from Table 3.1A and direct runoff, Q, is calculated using the rainfall runoff equation.
3. The CN value is used to determine the initial abstraction, I_a, from Table 4.5A, and the ratio I_a/P is then computed (P = accumulated 24-hour rainfall).
4. The watershed time of concentration is computed using the procedures in the travel time subsection and is used with the ratio I_a/P to obtain the unit peak discharge (q_u) from Figure 4.5D for the Type II rainfall distribution. If the ratio I_a/P lies outside the range shown in the figures, either the limiting values or another peak discharge method should be used. Note: Figure 4.5D is based on a peaking factor of 484. If a peaking factor of 300 is needed, these figures are not applicable and the simplified SCS method should not be used. Peaking factors are discussed further in the next subsection.
5. The pond and swamp adjustment factor, F_p, is estimated from below:

Pond and Swamp Areas (%*)	F_p
0	1.00
0.2	0.97
1.0	0.87
3.0	0.75
5.0	0.72

*Percent of entire drainage basin

6. The peak runoff rate is computed using the simplified peak runoff rate equation above.

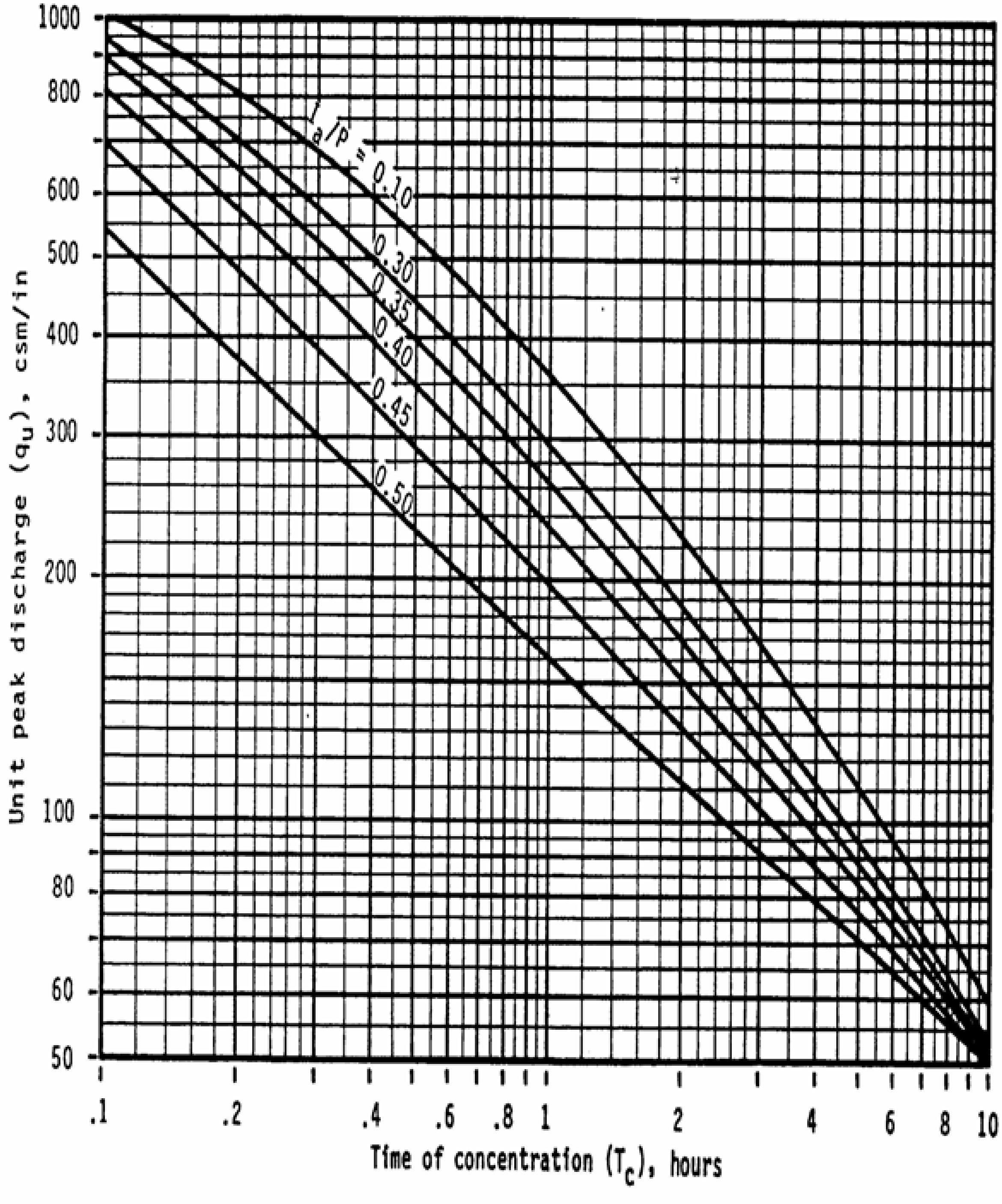


Figure 4.5D
SCS Type II Unit Peak Discharge Graph
(Source: SCS, TR-55, Second Edition, June 1986)

Hydrograph Generation

In addition to estimating the peak discharge, the SCS method can be used to estimate the entire hydrograph from a drainage area. The SCS has developed a Tabular Hydrograph procedure that can be used to generate the hydrograph for small drainage areas (less than 2,000 acres). The Tabular Hydrograph procedure uses unit discharge hydrographs that have been generated for a series of time of concentrations. In addition, SCS has developed hydrograph procedures to be used to generate composite flood hydrographs. For the development of a hydrograph from a homogeneous developed drainage area and drainage areas that are not homogeneous, where hydrographs need to be generated from sub-areas and then routed and combined at a point downstream, the engineer is referred to the procedures outlined by the SCS in the 1986 version of TR-55 available from the National Technical Information Service in Springfield, Virginia 22161. The catalog number for TR-55, "Urban Hydrology for Small Watersheds," is PB87-101580.

The unit hydrograph equations used in the SCS method for generating hydrographs includes a constant to account for the general land slope in the drainage area. This constant, called a peaking factor, can be adjusted when using the method. A default value of 484 for the peaking factor represents rolling hills – a medium level of relief. SCS indicates that for mountainous terrain the peaking factor can go as high as 600, and as low as 300 for flat (coastal) areas.

A value of 484 should be used for most areas of North Texas; however, there are flat areas where a lesser value may be appropriate.

The development of a runoff hydrograph from a watershed is a laborious process not normally done by hand calculation. For that reason, only an overview of the process is given here to assist the designer in reviewing and understanding the input and output from a typical computer program. There are choices of computational interval, storm length (if the 24-hour storm is not going to be used), and other "administrative" parameters, which are peculiar to each computer program.

The development of a runoff hydrograph for a watershed or one of many sub-basins within a more complex model involves the following steps:

1. Development or selection of a design storm hyetograph. Often the SCS 24-hour storm described in the Equations and Concepts portion of this subsection is used. This storm is recommended for use in North Central Texas.
2. Development of curve numbers and lag times for the watershed using the methods described in this manual.
3. Development of a unit hydrograph using the standard (peaking factor of 484) dimensionless unit hydrograph. See discussion below.
4. Step-wise computation of the initial and infiltration rainfall losses and, thus, the excess rainfall hyetograph using a derivative form of the SCS rainfall-runoff equation.
5. Application of each increment of excess rainfall to the unit hydrograph to develop a series of runoff hydrographs, one for each increment of rainfall (this is called "convolution").
6. Summation of the flows from each of the small incremental hydrographs (keeping proper track of time steps) to form a runoff hydrograph for that watershed or sub-basin.

To assist the designer in using the SCS unit hydrograph approach with a peaking factor of 484, Figure 4.5E and Table 4.5C have been developed. The unit hydrograph with a peaking factor of 300 is shown in the figure for comparison purposes, but, typically, should not be used for areas in North Central Texas.

The procedure to develop a unit hydrograph from the dimensionless unit hydrograph in the table below is to multiply each time ratio value by the time-to-peak (T_p) and each value of q/q_u by q_u calculated as:

$$q_u = (PF\ A) / (T_p)$$

where:

q_u	=	unit hydrograph peak rate of discharge (cfs)
PF	=	peaking factor (484)
A	=	area (mi^2)
d	=	rainfall time increment (hr)
T_p	=	time to peak = d/2 + 0.6 t_c (hr)

For ease of spreadsheet calculations, the dimensionless unit hydrograph for 484 can be approximated by the equation:

$$q / q_u = \left(t/T_p\ e^{[1-(t/T_p)]} \right)^X$$

where X is 3.79 for the PF=484 unit hydrograph.

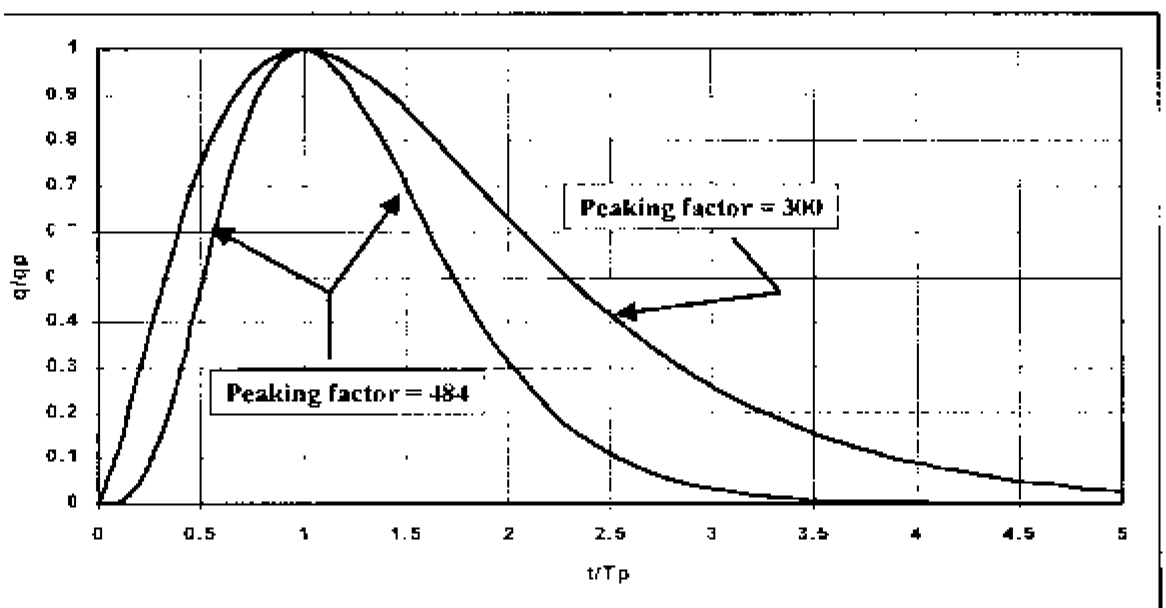


Figure 4.5E
Dimensionless Unit Hydrographs for Peaking Factors of 484 and 300

Table 4.5C
Dimensionless Unit Hydrograph
With Peaking Factor of 484

		484
t/T_t	q/q_u	Q/Q_p
0.0	0.0	0.0
0.1	0.005	0.000
0.2	0.046	0.004
0.3	0.148	0.015
0.4	0.301	0.038
0.5	0.481	0.075
0.6	0.657	0.125
0.7	0.807	0.186
0.8	0.916	0.255
0.9	0.980	0.330
1.0	1.000	0.406
1.1	0.982	0.481
1.2	0.935	0.552
1.3	0.867	0.618
1.4	0.786	0.677
1.5	0.699	0.730
1.6	0.611	0.777
1.7	0.526	0.817
1.8	0.447	0.851
1.9	0.376	0.879
2.0	0.312	0.903
2.1	0.257	0.923
2.2	0.210	0.939
2.3	0.170	0.951
2.4	0.137	0.962
2.5	0.109	0.970
2.6	0.087	0.977
2.7	0.069	0.982
2.8	0.054	0.986
2.9	0.042	0.989
3.0	0.033	0.992
3.1	0.025	0.994
3.2	0.020	0.995
3.3	0.015	0.996
3.4	0.012	0.997
3.5	0.009	0.998
3.6	0.007	0.998
3.7	0.005	0.999
3.8	0.004	0.999
3.9	0.003	0.999
4.0	0.002	1.000

4.6 Snyder’s Unit Hydrograph Method

Snyder’s unit hydrograph method is the primary method utilized by the Corps of Engineers Fort Worth District for the majority of hydrologic studies in the region, and is also commonly used by consultants and other entities within the NCTCOG region. It is similar in nature to the SCS method, in that it also considers the time distribution of the rainfall, the initial rainfall losses to interception and depression storage, and an infiltration rate that decreases during the course of a storm.

Application

Snyder's unit hydrograph method may be used for drainage areas 100 acres or larger. This method, detailed in the U.S. Army Corps of Engineers Engineering Manual (EM 1110-2-1405), *Flood-Hydrograph Analysis and Computations* and The Bureau of Reclamation’s “Flood Hydrology Manual, A Water Resources Technical Publication,” utilizes the following equations:

$$t_p = C_t\,(L\,L_{ca})^{0.3} \qquad (2.1.17)$$

$$t_r = t_p \div 5.5 \qquad (2.1.18)$$

$$q_p = C_p 640 \div t_p \qquad (2.1.19)$$

$$t_{pR} = t_p + 0.25(t_R - t_r) \qquad (2.1.20)$$

$$q_{pR} = C_p 640 \div t_{pR} \qquad (2.1.21)$$

$$q_{pR} = q_p\,t_p \div t_{pR} \qquad (2.1.22)$$

$$Q_p = q_p\,A \qquad (2.1.23)$$

The terms in the above equations are defined as:

t_r = The standard unit rainfall duration, in hours.

t_R = The unit rainfall duration in hours other than standard unit, t_r, adopted in specific study.

t_p = The lag time from midpoint of unit rainfall duration, t_r, to peak of unit hydrograph in hours.

t_{pR} = The lag time from midpoint of unit rainfall duration, t_R, to peak of unit hydrograph in hours.

q_p = The peak rate of discharge of unit hydrograph for unit rainfall duration, t_r, in cfs/sq. mi.

q_{pR} = The peak rate of discharge in cfs/sq. mi. of unit in hydrograph for unit rainfall duration, t_R.

Q_p = The peak rate of discharge of unit hydrograph in cfs.

A = The drainage area in square miles.

L_{ca} = The river mileage from the design point to the centroid of gravity of the drainage area.

L = The river mileage from the given station to the upstream limits of the drainage area.

C_t = Coefficient depending upon units and drainage basin characteristics.

C_p = Coefficient depending upon units and drainage basin characteristics.

The coefficient C_t is a regional coefficient for variations in slopes within the watershed. Typical values of C_t range from 0.4 to 2.3 and average about 1.1. The value of C_t for the East Fork

Trinity River is 2.0. C_t for a watershed can be estimated if the lag time, t_p, stream length, L, and distance to the basin centroid, L_{ca}, are known. The coefficient C_p is the peaking coefficient, which typically ranges from 0.3 to 1.2 with an average value of 0.8, and is related to the flood wave and storage conditions of the watershed. The C_p value for the East Fork Trinity River is 0.69. Larger values of C_p are generally associated with smaller values of C_t. Typical values of C_p are listed in Table 4.6A.

Table 4.6A
Typical Values of C_p

Typical Drainage Area Characteristics	Value of C_p
Undeveloped Areas w/ Storm Drains	
Flat Basin Slope (less than 0.50%)	0.55
Moderate Basin Slope (0.50% to 0.80%)	0.58
Steep Basin Slope (greater than 0.80%)	0.61
Moderately Developed Area	
Flat Basin Slope (less than 0.50%)	0.63
Moderate Basin Slope (0.50% to 0.80%)	0.66
Steep Basin Slope (greater than 0.80%)	0.69
Highly Developed/Commercial Area	
Flat Basin Slope (less than 0.50%)	0.70
Moderate Basin Slope (0.50% to 0.80%)	0.73
Steep Basin Slope (greater than 0.80%)	0.77

Urbanization Curves

To account for the effects of urbanization, another method was developed by the Corps of Engineers to adjust the t_p coefficient. Urbanization curves allow for the determination of t_p based on the percent urbanization and the type of soil in the study area. Urbanization curves for the Dallas-Fort Worth area were determined from the equation below:

$$t_p = 10\text{^}[0.3833*\log_{10}(L*L_{ca}/(S_{st})\text{^}.5)+(\log_{10}(Ip))-BW*(\%Urb/100)] \qquad (2.1.24)$$

$$S_{st} = (el_{85\%} - el_{15\%})/(0.7*L) \qquad (2.1.25)$$

where:

- t_p = The lag time from midpoint of unit rainfall duration, t_r, to peak of unit hydrograph in hours.
- L_{ca} = The river mileage from the design point to the centroid of the drainage area.
- L = The river mileage from the design point to the upstream limits of the drainage area.

S_{st}	=	The weighted slope of the flow path (ft/mi)
Ip	=	The calibration point, defined as t_p where $(L*L_{ca}/S_{st}$^.5) = 1 and urbanization = 0%.
BW	=	The bandwidth, equal to the log of the width between each 20% urbanization line.
%Urb	=	A value representative of the degree to which urbanization has occurred in the basin, in percent.
$el_{85\%}$	=	The elevation at a location 85% upstream of the given station.
$el_{15\%}$	=	The elevation at a location 15% upstream of the given station.

For the Dallas-Fort Worth area, the I_p values used are 0.94 for clay and 1.76 for sand. The bandwidth (BW) value for both of the soil types is 0.266. For a study area that is composed of both sand and clay, a weighted average of the two can be calculated by:

t_p *weighted = % sand**t_p *sand + % clay ** t_p *clay.*

Design runoff may be determined for a given watershed by applying the intensity-duration-frequency relationships to the unit hydrograph by multiplying each ordinate of the unit hydrograph by the rainfall intensity.

Determination of Percent Urbanization and Percent Sand

The lag time, t_p, is the critical parameter in establishing the timing of the response of a watershed to rainfall. The degree of urbanization is an important variable that determines the value of the lag time. Thomas L. Nelson, Fort Worth District, USACE, defined the general relationship between the lag time, t_p, and the percent of Urbanization, %Urb, and presented a set of Urbanization Curves for the Dallas-Fort Worth area in 1970.

The soil type of a watershed also plays an important role in its response to rainfall. It was found that predominantly sandy soils responded differently to rainfall than predominantly clayey soils. Therefore, two sets of Urbanization Curves were developed to better define the lag time, one set for sandy soils and one set for clayey soils. A paper by Paul K. Rodman, Fort Worth District, USACE presented urbanization curves in 1977 for both “clay loam” and “clay” in the Fort Worth-Dallas area and other Texas locations.

To obtain consistency of computational results, it is necessary to have a logical and routine procedure for the determination of Percent Urbanization (%Urb) and Percent Sand/Clay (%Sand/%Clay). Procedures for their determination are presented below.

Percent Urbanization

Urbanization is defined as the percentage of the basin which has been developed and improved with channelization and/or a stormwater collection network. Urbanization of natural and agricultural land converts pervious soils to impervious surfaces. Disturbed soils exhibit a lower infiltration capacity than natural soils. This results in less infiltration which translates to an increased volume of runoff.

Natural flow paths in the watershed may be replaced with prismatic channels. Significant drainage infrastructure may be added in a development composed of streets and gutters, storm sewers, open channels, and other drainage elements. This alteration of the original drainage system changes the watershed’s response to precipitation. The addition of drainage infrastructure along with the increase in imperviousness results in significantly increased peak discharges and a greater volume of runoff.

The determination of the percent urbanization (%Urb) as used in the Urbanization Curves defined by the equation above is somewhat subjective, but is related to the type and intensity of development. The U.S. Army Corps of Engineers (USACE) has worked over the years to define the relationship between the type of development and the degree of urbanization. The result of their effort is reflected in Table 4.6B. These are provided for the user's consideration and guidance.

Table 4.6B
Percent Urbanization and Imperviousness Summary with Associated Land Use Categories

Land Use	Description	Percent Imperviousness	Percent Urbanization
Low Density Residential	Single family: ½ – 2 units per acre; average 1 unit per acre.	25	30
Medium Density Residential	Single family: 2 – 3½ units per acre; average 3 units per acre.	41	80
High Density Residential	Single family: greater than 3½ units per acre; average 4 units per acre.	47	90
Multifamily Residential	Row houses, apartments, townhouses, etc.	70	95
Mobile Home Parks	Single family: 5–8 units per acre.	20	40
Central Business District	Intensive, high-density commercial	95	95
Strip Commercial	Low-density commercial; average 3 units per acre.	90	90
Shopping Centers	Grocery stores, drug stores, malls, etc.	95	95
Institutional	Schools, churches, hospitals, etc.	40	50
Industrial	Industrial centers and parks; light and heavy industry.	90	95
Transportation	Major highways, railroads.	35	80
Communication	Microwave towers, etc.	35	50
Public Utilities	Transformer stations, transmission line right-of-way, sewage treatment facilities, water towers, and water treatment facilities.	60	70
Strip Settlement	Densities less than ½ – 2 units per acre; average 1 unit per 3 – 5 acres.	10	20
Parks and Developed Open Space	Parks, cemeteries, etc.	6	10
Developing	Land currently being developed.	15	20
Cropland		3	5
Grassland	Pasture, short grasses.	0	0
Woodlands, Forest		0	0
Water Bodies	Lakes, large ponds.	100	100
Barren Land	Bare exposed rock, strip mines, gravel pits.	0	0

Determination of Percent Urbanization/Imperviousness in Watersheds, May 1, 1986, U.S. Army Corps of Engineers
SCS, TR-55, Second Edition, June 1986

Percent Sand/Clay

The Fort Worth District, USACE, evaluated methods for determining the percent sand in a watershed and concluded that the permeability rate method was the best method. The procedure was described in the referenced report as follows.

"The permeability rate method uses the range of permeabilities found in the table of physical and chemical properties in the SCS soil surveys for multiple soil classifications and assigns a percent sand to each of the seven ranges. A percent sand of 0 is given to any soil with a permeability less than 0.06 inches per hour which corresponds to the permeability of the Houston Blackland clay upon which the clay urban curves are based. Also, a percent sand of 100 is given to any soil with a rate of 0.6 to 2.0 inches per hour which corresponds to the Crosstell series soil upon which the sandy loam curves are based. The percent sand for the permeability ranges 0.06 to 0.2 inches, 0.2 to 0.6 inches, 2.0 to 6.0 inches, 6.0 to 10.0 inches, and greater than 20 inches are 33, 66, 133, 166, 200 percent sand, respectively. Each soil in the watershed is assigned a percent sand based upon its permeability and a weighted average is computed." (USACE, 1986).

Table 4.6C
Permeability Rating for the Determination of Percent Sand

Permeability (inches/hr)	Percent Sand Assignment (%)
< 0.06	0
0.06 to 0.20	33
0.20 to 0.60	66
0.60 to 2.00	100
2.00 to 6.00	133
6.00 to 20.00	166
> 20.00	200

The Houston Black soil series consists of moderately well-drained, deep, cyclic, clayey soils on wetlands. This series formed in alkaline, marine clay, and material weathered from shale. Land slopes range from 1 to 4 percent. The permeability is less than 0.06 inches per hour. This soil is the predominate series found in watersheds used to develop the Dallas-Fort Worth Clay Urbanization Curves. Therefore this soil has a percent sand of 8 for use with the urban curves. The Crosstell soil series consists of moderately well-drained, deep loamy soils on uplands that formed in shaley and clayey sediment containing thin strata of weakly cemented sandstone. Land slopes range from 1 to 6 percent. The permeability for this soil is in the range between 0.6 and 2.0 inches per hour. The Crosstell series is the major soil contained in watersheds used to derive the Dallas-Fort Worth Sandy Loam Urbanization Curves. This soil, therefore, has a percent sand of 100 for use with the urban curves.

Example: Procedure for the Determination of Percent Sand (%Sand).

Given the percent sand assignments below, determine the percent sand for Watershed B.

Watershed	Soil Type No.	Percent Sand	% of Area	% Sand * % Area
B	13	66	2.6	171.6
	23	33	39.7	1310.1
	32	133	31.4	4176.2
	51	33	1.7	56.1
	64	133	17.9	2380.7
	85	33	6.7	221.1
			100	8315.8

Weighted %Sand = 8315.8/100 = 83.2%

There is the possibility of computing greater than 100 percent sand for areas that are very sandy. Soil disturbances during development (urbanization) usually diminish the natural permeability of the soil. Often there is no data reflecting the permeability rate for an urban soil. Therefore, care should be used in applying this method. The percent sand assignment should be that of the controlling sublayer of the soil profile. Consideration should also be given to other factors affecting the initial and time rates of rainfall abstractions. For example, well-vegetated clayey soils may respond hydrologically more like a sandy soil. Urban lands are usually taken one step down (lower percent sand) from soil types shown in the SCS soil report. The engineer should evaluate all factors bearing on the soil response and determine whether there is a need to make adjustments.

Loss Rates

The following loss rate methodologies are acceptable for use with the Snyder's Unit Hydrograph Method:

- Initial and Constant Rate (Block and Uniform)
- SCS Curve Number

Initial and constant rate loss rates developed by the Corps of Engineers during the development of the urbanization curves are listed by clay and sand categories. Losses for a specific basin are determined by a weighting procedure. Adjustments to these values are allowed based on historic storm reproductions.

Table 4.6D
Hydrologic Loss Rates

Frequency	Losses			
	Clay		Sand	
	Initial (in)	Constant Rate (in/hr)	Initial (in)	Constant Rate (in/hr)
2-year	1.5	0.20	2.1	0.26
5-year	1.3	0.16	1.8	0.21
10-year	1.12	0.14	1.5	0.18
25-year	0.95	0.12	1.3	0.15
50-year	0.84	0.1	1.1	0.13
100-year	0.75	0.07	0.9	0.10

4.7 Stream Routing

As flood waves travel downstream, the hydrograph is translated in time and becomes attenuated by the storage effects of channels and streams. Translation and attenuation are achieved by routing the hydrograph through channel and stream reaches. The Modified Puls or Muskingum-Cunge methods are to be used for stream routing.

A Muskingum-Cunge Routing

The Muskingum-Cunge routing method uses a technique to approximate convective diffusion based on the channel physical properties and inflow hydrograph. This technique is to be used for prismatic channels where there are little or no backwater effects. The Floodplain Administrator should be consulted prior to using this method for non-prismatic stream reaches. This method employs the classical Muskingum equation to quantify storage in the reach. Four parameters are needed for this method: Manning's coefficients (for the channel and overbanks), reach length, slope of the energy grade (may be assumed to equal the channel slope), and the shape of the channel.

The time and distance steps must be selected to ensure stability and accuracy. It may be necessary to subdivide a reach into two or more reaches to meet these conditions. The channel geometry can be circular, rectangular, trapezoidal, or an eight-point format with a main stream and overbank floodplains.

B Modified Puls Routing

The modified Puls routing method uses a finite difference approximation to the hydrologic budget and, therefore, requires a stage-discharge-storage relationship. This relationship should be developed by conducting backwater analyses of the stream or channel reaches for multiple flood profiles with hydraulic modeling.

Again, the time and distance steps must be selected to ensure stability and accuracy. It may be necessary to subdivide a reach into two or more reaches to meet these conditions. Typically, the travel time of a flood wave through a reach can be estimated as 60 percent (60%) of the travel time through the same reach based on the velocity of the flow computed in the hydraulic analysis.

5.0 STREET FLOW

5.1 Flow in Gutters

The drainage capacities of streets and gutters shall be determined by Manning's Formula using an 'n' value of 0.016 for concrete streets. Streets and curb inlets shall be designed to flow not more than curb deep during a 10-year (10% annual chance) flood. Streets and curb inlets shall be designed to flow not more than ROW deep for the 100-year (1% annual chance) flood, special attention shall be paid to sag locations. Minor thoroughfares shall maintain one dry lane (assuming ultimate construction of three lanes in each direction) in each direction during a 10-year (10% annual chance) flood. Major thoroughfares shall maintain one dry lane in each direction during a 100-year (1% annual chance) flood. When an existing street slope is less than five (5) feet per 1,000 or at sag locations, the hydraulic capacity of the street and right-of-way shall be determined assuming a slope of three (3) feet per 1,000. Where a flow of water is directed toward a curb and is required to turn in direction, the height of the curb against which the water is directed shall not be less than the depth of water flow plus the velocity head of the water plus two (2) inches. Where water is discharged from a street directly into an open watercourse, it shall be discharged through an approved type of catch basin or through a concrete lined structure.

Computed gutter flow depths shall be shown on the plans in tables with the location (include sags and false sags), flood frequency, flow, type and size of street, and slope of street. There shall be two tables of gutter flow depths, one for the 10-year on-grade and 100-year at sag and the other for the 100-year (1% annual chance) flood. When existing street slope is less than 0.50 percent (0.50%) or at sag locations, inlets shall be provided to remove stormwater from the street such that flows at sump inlets do not exceed the values presented in Table 5.1A.

Split curb gutters shall be appropriately analyzed for area, wetted perimeter, and hydraulic radius for use in Manning's formula. Streets and gutters on grade shall be designed to flow not more than curb deep for the 10-year (10% annual chance) flood and the 100-year (1% annual chance) flood shall be contained within the right-of-way (ROW). Streets and gutters at sags shall be designed to flow not more than curb deep for the 100-year (1% annual chance) flood and the 100-year (1% annual chance) flood shall be contained within the ROW in other areas. At sags, split curbs, where grades are one percent (1.0%) or less, and other locations where grades are relatively flat it shall be demonstrated that the 10-year (10% annual chance) flood is within the gutter and the 100-year (1% annual chance) flood is conveyed within the ROW.

Table 5.1A
Maximum Flow at Sags

	Maximum Flow (cfs)	
Type of Street	Flow Confined to Street	Flow Confined to ROW
27’ B-B	10.5	30.3
31’ B-B	12.2	35.2
37’ B-B	9.1	33.5
41’ B-B	10.1	37.3
45’ B-B	11.1	41.2
2x25’ B-B Divided	25.5	65.6

Note: B-B = back of curb to back of curb. Data in table are the total flow in both gutters (or both sides of divided street) from one direction and are not applicable to streets with split curbs.

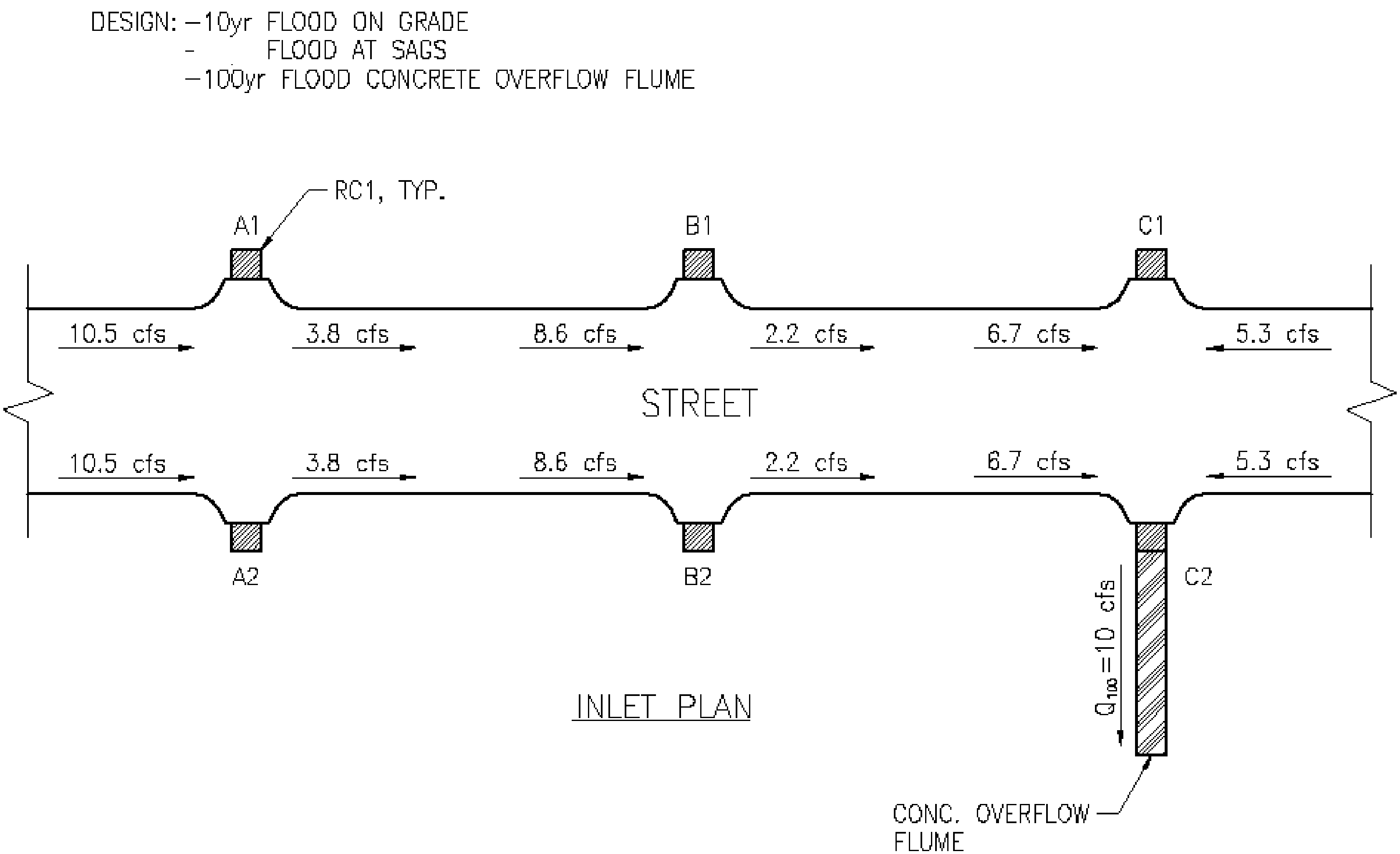


Figure 5.1
Inlet Plan

At sags in streets, the 100-year (1% annual chance) flood shall be collected in the storm drain. In all cases, the downstream storm drainage system shall be adequate to collect and convey the 100-year (1% annual chance) flow.

Special cases arise when a street is designed with “false sags”. A false sag is a sag that has a street grade PVI crest point adjacent to it. If the flow backups at the inlets in a false sag, it can overflow the adjacent street crest point and continue down the street grade to the next collection point, thereby, minimizing the depth of flooding at the false sag. Generally, to limit grade changes in the street and still minimize the possible depth of flooding, false sags are located near intersections. Inlets in false sags may be designed for the 10- or 100-year (1% annual chance) flood. If the inlets in a false sag are designed for the 10-year (10% annual chance) flood, then the 100-year (1% annual chance) flood overflows may bypass the false sag by flowing over the street PVI crest point. Bypass flow depth and average velocity over the crest point shall be determined using the normal depth method for subcritical flow states and the critical flow depth method for critical and supercritical flow states. The state of flow will be determined by the down grade of the street from the crest PVI. The 100-year pool elevation at the false sag will be controlled by both the inlet head using the inside inlet HGL and the weir or orifice equation at the inlet opening and by the energy grade for the bypass flow at the crest point. The bypass flow shall be adjusted so that these two elevation calculations are the same in order to determine the bypass flow to be used. The energy grade at the crest point equals the crest point gutter elevation plus the specific energy (E_{sp}) of the bypass flow at the crest point. The 100-year pool elevation must be contained at the false sag within the street ROW. The hydraulic grade in a street at a false sag shall be computed with the following formula and compared with the inlet head HGL calculation to insure the correct bypass flow is being used. Values of specific energy (E_{sp}) to be used for various bypass flows in City standard streets are presented in Table 5.1B.

$$HG = El_{gutter} + d_{n/c} + V_{n/c}^2 / 2g = El_{gutter} + E_{sp}$$

where:

HG = Elevation of the hydraulic grade at the false sag, feet (ft);
El_{gutter} = Flowline elevation of the gutter at the adjacent crest point, feet (ft);
$d_{n/c}$ = Normal or Critical depth of overflow in the crest point gutter as discussed above, feet (ft);
$V_{n/c}$ = Normal depth velocity or Critical velocity of bypass overflow in the crest point gutter, feet per second (fps);
g = 32.2 = Acceleration of gravity, ft/sec/sec; and
$E_{sp} = d_{n/c} + V_{n/c}^2 / 2g$ = Specific energy at normal depth or critical flow depending on the flow state as discussed above, feet (ft).

Table 5.1B
Bypass Flow in Streets as a Function of Specific Energy

Specific Energy (ft)	False Sag Bypass Discharge for Various Street Sizes (cfs)						
	27' Wide (B-B) Street Parabolic 5" Crown	31' Wide (B-B) Street Parabolic 5" Crown	37' Wide (B-B) Street Parabolic 6" Crown	41' Wide (B-B) Street Parabolic 6" Crown	45' Wide (B-B) Street Parabolic 6" Crown	51' Wide (B-B) Street Parabolic 7" Crown	One 25' Wide (B-B) Street Straight 1/4 In per Ft Cross Fall
0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10
0.15	0.30	0.35	0.35	0.40	0.40	0.40	0.40
0.20	0.65	0.75	0.75	0.80	0.90	0.90	0.85
0.25	1.20	1.35	1.35	1.50	1.65	1.60	1.60
0.30	1.95	2.25	2.20	2.45	2.75	2.65	2.60
0.35	2.90	3.35	3.25	3.65	4.05	3.90	3.85
0.40	4.30	4.95	4.75	5.30	5.80	5.50	5.45
0.45	5.95	6.85	6.60	7.35	8.10	7.65	7.35
0.50	8.50	9.80	8.75	9.75	10.75	10.20	9.60
0.55	11.35	13.15	11.55	12.85	14.15	13.25	12.20
0.60	14.15	16.40	14.40	16.25	18.15	15.95	15.20

Note: B-B stands for back of curb to back of curb. Data in table are the total flow in both street gutters and are not applicable to streets with split curbs.

5.2 Flow in Driveways and Intersections

At any intersection, only one street shall be crossed with surface drainage and this street shall be the lower classified street. Where an alley or street intersects a street, inlets shall be placed in the intersecting alley or street whenever the combination of flow down the alley or intersecting street would cause the capacity of the downstream street to be exceeded. Inlets shall be placed upstream from an intersection whenever possible. Surface drainage from a 10-year (10% annual chance) flood may not cross any street classified as a thoroughfare or collector according to the Master Thoroughfare Plan. Not more than 4.0 cfs per gutter may be discharged through an intersection in a 10-year (10% annual chance) flood. Not more than 5.0 cfs in a 100-year (1% annual chance) flood may be discharged per driveway at a business, commercial, industrial, manufacturing, or school site. In all cases, the downstream storm drainage system shall be adequate to collect and convey the flow, and inlets provided as required.

The cumulative flows from existing driveways shall be considered and inlets provided as necessary where the flow exceeds the specified design capacity of the street.

Where adjacent property is undeveloped, it may be necessary to provide Y-inlets outside of the street right-of-way to intercept concentrated flows before reaching the street. Laterals and mains shall be designed to provide drainage for fully developed flows as required by these standards and the y inlets shall be designed to intercept existing development 100-year (1% annual chance) flows from the offsite undeveloped property.

Intersection grading plans shall be provided in accordance with Section 2.6.C of the Drainage Design Manual.

6.0 INLET DESIGN

Two types of inlets are approved for use: a recessed curb inlet for streets and a Y-inlet for open areas or channels. Recessed curb inlets shall be 5-, 10-, 15-, or 20-feet in length. No more than 20 linear feet - of inlets shall be placed along one gutter at any given location. At sags, at least one curb inlet shall be a minimum of 10 feet in length. Computations for flow in to inlets shall be shown on the construction plans. Inlets shall be placed at intersecting property lines when possible.

6.1 Flow Into Inlets on Grade

Flow from triangular street gutters in to curb inlets on grade shall be computed with the following formulas.

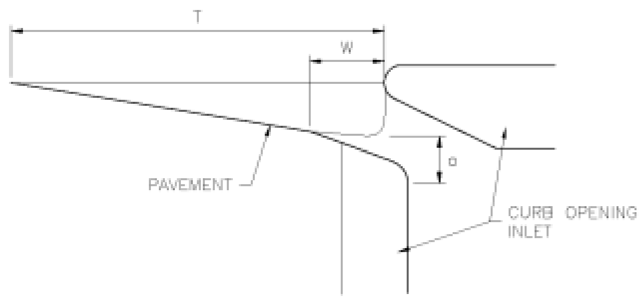


Figure 6.1A
Triangular Gutter Section

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

1. The ponded width can be determined by:

$$T = \frac{d}{S_X} \qquad or \qquad T = \left(\frac{Qn}{K_U S_X^{1.67} S_L^{0.5}}\right)^{0.375}$$

where:

T = ponded width (ft or m)
K_U = 0.56 (0.376 for metric)

2. $$d = z \left(\frac{QnS_x}{S_L^{0.5}}\right)^{0.375}$$

where:

d = depth of water in the curb and gutter cross section (ft or m)
Q = gutter flow rate (cfs or m^3/s)
n = Manning's roughness coefficient (0.016)
S_L = longitudinal slope (ft/ft or m/m)
S_X = pavement cross slope (ft/ft or m/m)
z = 1.24 for English units (1.443 for metric)

3. Determine the ratio of the width of flow in the depressed section (*W*) to the width of total gutter flow (*T*) by:

$$E_O = \frac{K_W}{K_W + K_O}$$

where:

E_O = ratio of depressed flow of total flow
K_W = conveyance of the depressed gutter section (cfs or m^3/s)
K_O = conveyance of the gutter section beyond the depression (cfs or m^3/s)

The conveyance of a cross section can be computed by:

$$K = \frac{zA^{2/3}}{nP^{2/3}}$$

where:

z = 1.486 for English units (1.0 metric)
A = area of cross section (ft^2 or m^2)
n = Manning's roughness coefficient (0.016)
P = wetted perimeter (ft or m)

4. Determine the area of the cross section in the depressed gutter section by:

$$A_W = WS_X\,(T - {}^{W}\!/_{2}) + \tfrac{1}{2}aW$$

where:

A_W = area of depressed gutter section (ft^2 or m^2)
W = depression width for an on-grade curb inlet (ft or m)
S_X = cross slope (ft/ft or m/m)
T = calculated ponded width (ft or m)
a = curb opening depression (ft or m)

5. Determine the wetted perimeter in the depressed gutter section by:

$$PW = \sqrt{(WS_X + a)^2 + W^2}$$

where:

P_W = wetted perimeter of depressed gutter section (ft^2 or m^2)
W = depression width for an on-grade curb inlet (ft or m)
S_X = cross slope (ft/ft or m/m)
a = curb opening depression (ft or m)

6. Determine the area of cross section of the gutter section beyond the depression by:

$$A_O = \frac{S_X}{2}(T - W)^2$$

where:

A_O = area of gutter/road section beyond the depression width (ft^2 or m^2)
S_X = cross slope (ft/ft or m/m)
W = depression width for an on-grade curb inlet (ft or m)
T = calculated ponded width (ft or m)

7. Determine the wetted perimeter of the gutter section beyond the depression by:

$$P_O = T - W$$

Where:

P_O = wetted perimeter of the depressed gutter section (ft or m)
T = calculated ponded width (ft or m)
W = depression width for an on-grade curb inlet (ft or m)

B. Flow from gutter sections to recessed curb inlets on grade shall be computed with the following formulas:

$$E_O = 1 \Bigg/ \left\{ 1 + \frac{S_W / S_X}{\left(1 + \dfrac{S_W / S_X}{\dfrac{T}{W} - 1} \right)^{2.67} - 1} \right\}$$

$$Q_W = Q - Q_S$$

$$Q_S = \frac{K_U}{n} \; S_X^{\;1.67} \, S_L^{\;0.5} \, T^{\,2.67}$$

$$Q = \frac{\mathrm{Q_S}}{1 - \mathrm{E_O}}$$

where:

K_U = 0.53 (0.376 for metric)
n = Manning's roughness coefficient (0.016)
S_L = longitudinal slope (ft/ft or m/m)
S_X = pavement cross slope (ft/ft or m/m)
Q_W = flow rate in the depressed section of the gutter, m^3/s (ft^3/s)
Q = gutter flow rate, m^3/s (ft^3/s)
Q_S = flow capacity of the gutter section above the depressed section, m^3/s (ft^3/s)
E_O = ratio of flow in a chosen width (usually the width of a grate) to total gutter flow Q_W/Q)
S_W = $S_X + a/W$

C. Flow from parabolic gutter sections to curb inlets on grade shall be computed with the following formulas:

A parabolic cross section can be described by the equation:

$$y = ax - bx^2$$

where:

a = 2H/B
b = H/B^2
H = crown height

B = street half width

The relationships between a, b, crown height, H, and street half width, B, are shown in Figure 6.1B.

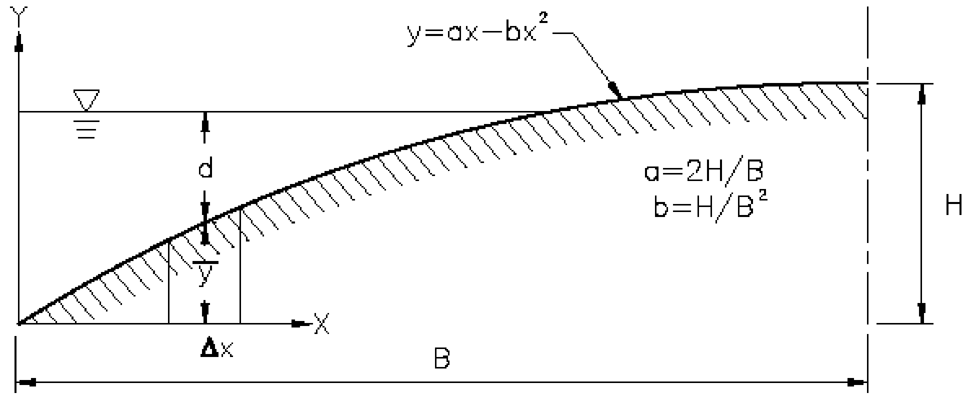


Figure 6.1B
Properties of a Parabolic Curve

To determine depth of flow in a gutter in a parabolic street, first determine conveyance (K).

$K = Q/S^{1/2}$

where:
Q = single gutter discharge (cfs)
S = gutter slope (ft/ft)

Depth of flow in a gutter (y) shall be determined by the following formula:
$y = mK^p$

m and p are coefficients that vary depending on street width and crown height. Table 5.1B provides coefficients for calculating depth in parabolic streets.

Table 6.1
Conveyance Coefficients

Street Width	m	p	H (ft)	Conveyance (K) for y = crown height	Conveyance (K) for y = 0.5 ft. (full depth
27 feet	0.1005	0.3692	0.42	44.1	96.2
31 feet	0.0952	0.3692	0.42	51.0	111.0
37 feet	0.0954	0.3693	0.5	83.0	83.0
45 feet	0.0884	0.3696	0.5	103.0	103.0

Note: Values of K are for a single gutter.

Spread of water can be calculated according to the following formula:

Spread of Water (T) = B – ((H-y)/b)$^{0.5}$

where:

H, B, & y are all in feet

D. Determine the equivalent cross slope (S_E) for a depressed curb opening inlet by:

$$S_E = S_X + \frac{a}{W} E_O$$

where:

- S_E = equivalent cross slope (ft/ft or m/m)
- S_X = cross slope of road (ft/ft or m/m)
- a = gutter depression (ft or m)
- W = gutter depression width (ft or m)
- E_O = ratio of depressed flow to total flow

E. Calculate the length of curb inlet required for total interception by:

$$L_R = zQ^{0.42} S_L^{0.3} \left[\frac{1}{nS_E}\right]^{0.6}$$

where:

- L_R = length of curb inlet required (ft or m)
- z = 0.6 for English units (0.82 metric)
- Q = flow rate in gutter (cfs or m^3/s)
- S_L = longitudinal slope (ft/ft or m/m)
- n = Manning's roughness coefficient
- S_E = equivalent cross slope (ft/ft or m/m)

If no carryover is allowed, the inlet length is assigned a dimension of at least L_r. Use a nominal length available for standard curb opening inlets. If carryover is considered, round the curb opening inlet length down to the next available standard curb opening length and compute the carryover flow.

F. Determine carryover flow by:

$$Q_{CO} = Q\left[1- \frac{L_A}{L_R}\right]^{1.8}$$

where:

- Q_{CO} = carryover flow (cfs or m^3/s)
- Q = total flow in gutter (cfs or m^3/s)
- L_A = design length of proposed the curb opening inlet required to intercept the total flow (ft or m)

Carryover rates usually should not exceed about 0.5 cfs (0.03 m^3/s) or about 30 percent (30%) of the total flow in gutter. Greater rates can be troublesome and cause a significant departure from the principles of the Rational Method Application. In all cases, you must accommodate any carryover rate at some other specified point in the storm drain system.

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

$$Q_I = Q - Q_{CO}$$

where:

Q_I = intercepted flow (cfs or m^3/s)
Q = total flow in gutter (cfs or m^3/s)
Q_{CO} = carryover flow (cfs or m^3/s)

H. If the curb inlet opening is not depressed, the intercepted flow shall be reduced by 20 percent (20%), and the carry overflow shall be increased by the same amount.

6.2 Curb Inlets at Sags

The flow into a public street curb inlet in a sag may be roughly estimated as 2.0 cfs/ft. for concept planning purposes provided the flow is confined to the street right-of-way and street sag for the 100-year (1% annual chance) flood and provided the required dry lane (10 foot wide minimum) is provided for arterial streets.

For weir flow control where the inlet is not subject to submergence, Section 4.4.5.2 *Curb Opening Inlets* of *HEC-22 Urban Drainage Design Manual* (FHWA, September 2009) shall be used for inlet flow capture determinations using the general equation (4-28) in section 4.4.5.2:

$$Q_I = C_d (L+1.8W) d^{1.5}$$

Where:

Q_I = Intercepted flow, cfs;
$^*C_d$ = Discharge coefficient (3.0 for City curb inlets);
L = Length of Curb Inlet Opening, ft;
W = Lateral Width of Depression (in gutter flow path), ft; and
d = Depth of Flow at approach in normal street gutter measured at normal street cross slope, ft.

* No cloggage factor is normally used for curb inlets. HEC-22 has C_d set to 2.3 using 3 times a cloggage factor of 0.75.

For curb inlet without depression, W = 0. Depression moves the location of critical depth and the weir section to the top of the depressed section providing more weir length for the inlet.

All curb inlets shall be in compliance with the Standard Construction Details as currently amended. For city Standard Recessed Curb Inlets and Modified Combination Curb Inlets 1 foot of the 3 foot wide depression is in the gutter flow path so W = 1. For City Standard Curb Inlets, W = 3. Using the HEC-22 formula with the above W values gives the following inlet capture capacities for curb high flow:

Sag Inlet Capacities (cfs) at 6 Inch Curb High Flow Depth In Approach Gutters				
Type Inlet	**5-Foot Inlet**	**10-Foot Inlet**	**15-Foot Inlet**	**20-Foot Inlet**
Standard Curb	11.03	16.33	21.64	26.94
Standard Recessed	7.21	12.52	17.82	23.12
Modified Combination Curb	*	12.52	*	*

* Inlet comes only in 10-foot size.

When the normal depth of flow above the inlet invert, d_n, rises to 1.40 times the inlet throat opening or higher, the inlet opening becomes fully submerged, causing orifice flow to begin to govern the head-discharge relationship. The general orifice flow equation is as follows:

$$Q_I = C_d A_0 (2gHW)^{0.5}$$

where:

- Q_I = Intercepted flow, cfs;
- C_d = Discharge coefficient;
- A_0 = Area of inlet opening, ft^2;
- g = Gravity acceleration constant, 32.2 ft/s^2; and
- HW = Headwater depth above centerline of inlet opening height, ft.

Rewriting headwater, HW, as the depth of flow above the inlet opening height centerline, h/2 we have

$$Q_I = C_d A_0 ((2g)(d-h/2))^{0.5}$$

where:

- d = Depth of flow above inlet invert, ft;
- h = Vertical height of inlet opening , ft.

Substituting the area of inlet opening, A_0, as the length of inlet opening, L, measured in feet, multiplied by the height of inlet opening, h, we have

$$Q_I = C_d (hL)((2g)(d-h/2))^{0.5}$$

By substituting 32.2 ft/s^2 for g and obtaining a C_d of 0.60 from an orifice discharge coefficient table, we have

$$Q_I = (0.60)(hL)((2)(32.2)(d-h/2))^{0.5}$$

Finally, simplifying produces the following orifice flow equation for city curb inlets:

For $d_n \geq 0.88'$: $Q_I = 3.01\ L\ (d - 0.313)^{0.5}$ *where* $h = 0.625$ *feet*

No clogging assumption is required for curb inlet calculations. It should be confirmed that the calculated inlet headwater elevation is confined to the street right-of-way and sag topography and if not, additional inlet capacity shall be required. If the street is an arterial it should be confirmed that the required 10-foot wide dry lane is also provided for the required frequency storm.

6.3 Y-Inlets

Flow into Y inlets shall be calculated using either the weir flow formula for unsubmerged inlets with no depression or the orifice flow formula for a submerged inlet. First, weir flow will be considered. Weir flow occurs when an inlet is unsubmerged, which generally occurs at depths of flow, d, less than 1.40 times the inlet throat opening height (equals 0.82 feet above the inlet invert for city standard Y inlets.) Beginning with the general weir flow equation, we have the following:

$$Q_I = C_d \, L \, d^{\,1.5}$$

where:

Q_I	=	Intercepted flow, cfs;
C_d	=	Discharge coefficient;
L	=	Total Length of Y-inlet openings perpendicular to the flow, ft; and
d	=	Head measured above weir crest assumed to occur at the bottom edge of Y-inlet opening, ft.

The weir flow equation can now be applied to Y inlets to obtain the following:

For $d < 0.82'$: $Q_I = 2.25 \, L \, d^{\,1.5}$

where:

Q_I	=	Flow in to inlet, cfs;
2.25	=	Weir coefficient adjusted for 25% clogged inlet throat;
L	=	Length of throat opening, ft; and
d	=	Depth of flow at inlet throat, ft.

When the depth of flow above the inlet invert, *d*, rises to 0.82 feet or higher, the inlet opening becomes submerged, causing orifice flow to begin to govern the head-discharge relationship. The general orifice flow equation is as follows:

$$Q_I = C_d A_0 (2gHW)^{0.5}$$

where:

Q_I	=	Intercepted flow, cfs;
C_d	=	Discharge coefficient;
A_0	=	Area of inlet opening, ft^2;
g	=	Gravity acceleration constant, 32.2 ft/s^2; and
HW	=	Headwater above centerline of inlet opening height, ft.

Rewriting headwater, *HW*, as the depth of flow above the inlet opening height centerline, which is 0.292 feet for the city standard Y-inlet for an opening height of 0.583 feet, we have

$$Q_I = C_d A_0 ((2g)(d-0.292))^{0.5}$$

where:

d = Depth of flow above inlet invert, ft.

Substituting the area of inlet opening, A_0, as the length of inlet opening, *L*, measured in feet, multiplied by the height of inlet opening, 0.583 feet, we have

$$Q_I = C_d(0.583L)((2g)(d\text{-}0.292))^{0.5}$$

By substituting 32.2 ft/s^2 for *g*, obtaining a C_d of 0.60 from an orifice discharge coefficient table, and assuming that 25% of the inlet throat is clogged, leaving 75% open to flow, we have

$$Q_I = (0.60)(0.75)(0.583L)((2)(32.2)(d\text{-}0.292))^{0.5}$$

Finally, simplifying produces the following orifice flow equation for city standard Y inlets:

For $d \geq 0.82'$: $Q_I = 2.11\ L\ (d - 0.292)^{0.5}$

where:

2.11 = Constant for English units which accounts for the inlet throat height, the acceleration of gravity and orifice coefficient adjusted for 25% clogged inlet throat.

6.4 Grate Inlets

The use of grate inlets in public-right-of-way and easements shall require special approval from the City Engineer.

Grate inlets generally lose capacity with increase in grade, but to a lesser degree than curb opening inlets. The principal advantage of grate inlets is they are installed along the roadway where the water is flowing. The disadvantage is that they may be clogged by floating trash or debris. For safety reasons, preference should be given to grate inlets where out of control vehicles might be involved. Additionally, where bicycle traffic occurs, grates should be bicycle safe. Grate inlets shall only be used with the approval of the City Engineer for street and public drainage system construction. Private systems may include grate inlets as outlined in this manual.

The allowable types of grates for use in the City depend on the inlet condition. Standard grate designs are provided in the City construction detail standards for drainage construction as currently amended which may be used in paved areas. The engineer is responsible for the selection of appropriate grates to be used on private property and shall provide construction details and capacity calculations to confirm the drainage requirements of this manual are met. Y-inlets normally are required in unpaved areas for drainage flow collection to limit clogging problems that may occur in these areas. If the Y-inlet is not appropriate, the TXDOT Type E grate inlet may be used for unpaved areas to collect drainage flows.

Grate inlet designs shall comply with the requirements of HEC-22 Urban Drainage Design Manual (FHWA, September 2009) for grate inlets to ensure that the inlet and system drainage flow and collection and capacity requirements of this manual are met.

Grate Inlets on Grade

Refer to HEC-22 for design requirements for grate inlets on grade.

Grate Inlets in Sag

Grate inlets in sag vertical curves and depressed areas operate as weirs for shallow ponding depths and as orifices at greater depths. Between weir and orifice flow depths, a transition from weir to orifice flow occurs. The clear perimeter and clear opening area of the grate and the depth of water at the curb affect inlet capacity. The capacity at a given depth can be severely affected if debris collects on the grate and reduces the effective perimeter or clear opening area. Grates of larger dimension will operate as weirs to greater depths than smaller grates.

In general the following applies for grate inlet capacity calculations.

The capacity of grate inlets operating as weirs is:

$$Q_i = C_W P d^{1.5}$$

where:

P is the clear perimeter of the grate in ft disregarding the frame bar widths along the perimeter and the structure side against any adjacent curb
C_W is 3.0 for English Units
d is the average depth across the grate:

$$d=0.5(d_1 + d_2)$$

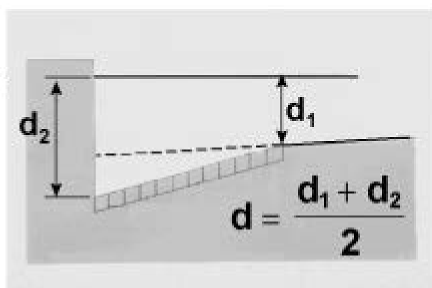


Figure 6.4
Average Depth across Grate

The capacity of a grate inlet operating as an orifice is:

$$Q_i = C_o A_g (2gd)^{0.5}$$

where:

C_o is the orifice coefficient = 0.60
A_g is the clear opening area of the grate (ft^2)
g = 32.2 (ft/s^2) rounded to one decimal place

This equation requires the clear opening area of the grate.

Some assumptions must be made regarding the nature of clogging in order to compute the capacity of a partly clogged grate. For grate inlets in sag conditions calculations should be performed with the assumption that 50 percent (50%) of the grate open area is clogged, and the clear perimeter of the grate (disregarding side adjacent to the curb) shall be reduced by 25 percent (25%).

Full flow determination details are provided in HEC-22. Grate inlet drainage flow-capture and capacity shall be computed per the procedures specified in *HEC-22 Urban Drainage Design Manual* (FHWA, September 2009). Copies of this manual are available free of change on the Internet under publications at *http://www.fhwa.dot.gov/*.

6.5 Non-Recessed Curb Inlets

The use of non-recessed curb inlets in public right-of-way and easements shall require special approval from the City Engineer.

They shall only be allowed to avoid conflicts with existing utilities.
Their design shall be computed per the procedures specified in *HEC-22 Urban Drainage Design Manual* (FHWA, September 2009). Design requirements for non-recessed curb inlets are provided in section **6.1 Flow Into Inlets on Grade** and in section 6.2 **Curb Inlets at Sags** of this manual.

Copies of this manual are available free of change on the Internet under publications at *http://www.fhwa.dot.gov/.*

7.0 STORM DRAIN DESIGN

Stormwater runoff shall be carried in a closed conduit when the runoff can be carried in a pipe of 72 inches in diameter or smaller; or where it is necessary for the protection of adjacent facilities that the stormwater be carried in an enclosed facility. Headwalls and erosion protection shall be constructed at the outfall of all storm drain systems. Refer to Section 2.4, Drainage Design Computations, and Appendix L for accepted stormwater model and information tools.

7.1 Design Criteria

All closed conduit storm drains shall meet the following criteria:

A. All driveway culverts shall be RCP class III minimum, engineering analysis shall be performed to determine class required. End walls or headwalls shall be provided at each end of the culvert.

B. Interceptor, trunk, and mains shall have a minimum diameter of 24 inches and laterals shall have a minimum diameter of 18 inches, except private lines that drain and are located in single family backyards may be 12 inches minimum diameter;

C. Curb inlets in sag locations shall have storm drains laterals with a minimum diameter of 24 inches;

D. Box closed conduit interceptor, trunk, mains, and laterals shall have minimum dimensions of two feet by two feet (2' x 2') (three-foot by one-foot (3' x 1') boxes may be allowed at driveways with height restrictions);

E. Velocities shall not be less than two (2.0) fps, nor greater than 15.0 fps. Manning's "n" value shall be 0.013 for circular pipe and concrete box sections;

F. Storm drains shall be tied together with factory pre-fabricated wyes at a 45° or 60° angle and be aligned vertically centerline to centerline; upstream end of storm drainage pipes shall be extended beyond the upstream wye connection at least 3 feet and plugged for future extension. All bends shall be factory pre-fabricated 30°, 45° or 60° bends.

G. City standard or TxDOT standard headwalls and erosion protection shall be constructed at all inlets and outfalls on closed conduits. Headwalls shall be placed at or outside the right-of-way lines;

H. Access points (manholes) shall be located at vertical drops in grade and no greater than 550 feet apart in storm drains less than five (5) feet in diameter or height and no greater than 1,000 feet apart in larger conduits;

I. Inlets shall be connected to mains with lateral conduits and shall not be used as manholes or junctions on mains;

J. Private storm drains (excluding roof drains and residential backyard drains) shall have a minimum diameter of 15 inches and shall be RCP or corrugated with smooth inside HDPE or PVC (specify pipe and embedment);

K. All storm drains shall be ASTM C-76 reinforced concrete pipe or ASTM C-1433 storm drain box, except those that drain and are located in single family backyards may be corrugated smooth inside HDPE or PVC (specify pipe and embedment);

L. Provide concrete collar, as per City standard, at pipe size changes, at grade breaks and at lateral connections to existing storm drain;

M. Storm drainage alignment must conform to manufacturer's recommendations for maximum allowable joint opening or "pull" of joints for storm drainage lines to be constructed on curves. If the pipe joint opening or "pull" on the curve exceeds the manufacturer recommendations, a note shall be added to require storm drainage pipe with beveled joints (called radius pipe). If pipe joint opening exceeds one half, concrete collars at the joints shall be required.

7.2 Design Parameters

In addition to the criteria listed above, there are several general design parameters to be observed when designing storm drains that will tend to alleviate or eliminate common problems of storm drain performance:

A. Select pipe size and slope so that the velocity of flow will increase progressively down the system or at least will not appreciably decrease at inlets, bends or other changes in geometry or configuration. Pipe size shall not decrease downstream unless approved by the City Engineer.

B. For all pipe junctions other than manholes and junction boxes, manufactured wye connections should be used, and the angle of intersection shall not be greater than 60 degrees. This includes discharges into box culverts and channels. Special circumstances may require cut-ins instead of manufactured wye connections; the use of cut-ins must be approved by the City Engineer.

C. Inlet laterals will normally connect only one inlet to the trunk line. Special circumstances requiring multiple inlets to be connected with a single lateral shall be approved by the City Engineer.

D. Storm drain pipes shall be reinforced concrete pipe, minimum Class III, or stronger as determined by the engineer.

E. Plastic pipe will not be allowed in public easements and rights-of-way. Plastic pipe may be used on private property only if authorized by the City Engineer.

F. The cover over the crown of circular pipe should be at least two feet and should be based on the type of pipe used, the expected loads and the supporting strength of the pipe. Box sections should normally have a minimum of one (1) foot of cover; however, direct traffic may be allowed in special situations with the approval of the City Engineer.

G. All storm drain outfalls shall be into channels, creeks or natural water ways. The angle of intersection shall not be more than 60°. The outfall structure shall be as per Figure 7.2 or as approved by the City Engineer.

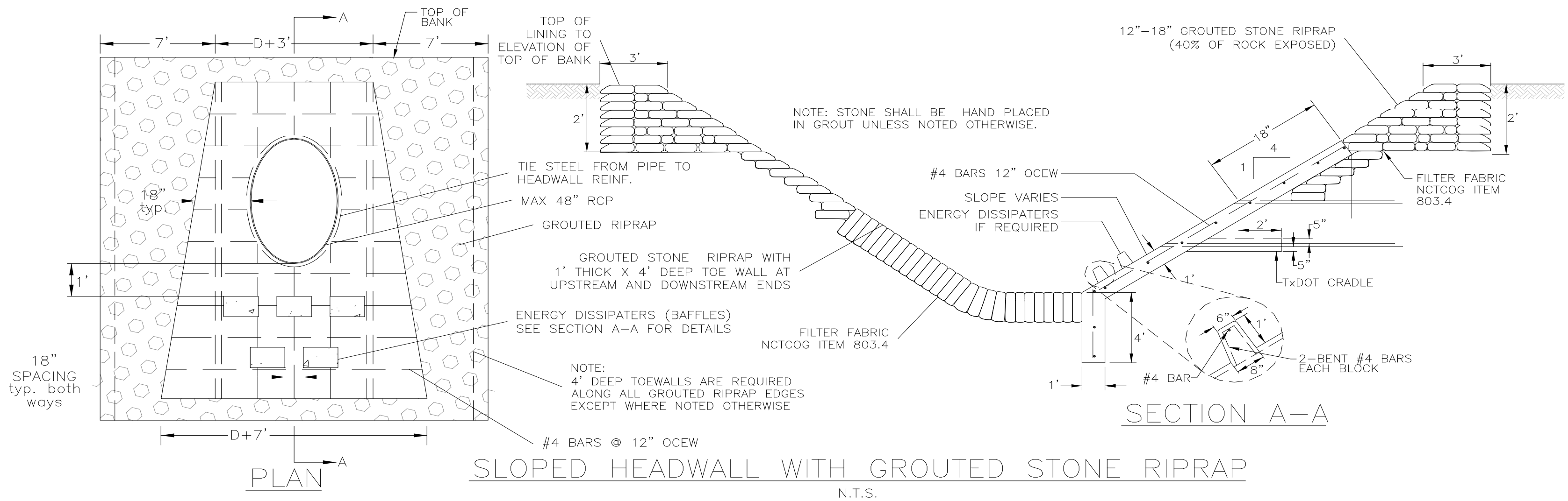


Figure 7.2
Storm Drain Outfall

7.3 Slug Flow

A. Slug flow occurs when air bubbles moving downstream in a closed conduit coalesce in to large air pockets that reverse flow and move upstream (refer to *Air-Water Flow In Hydraulic Structures*, H.T. Falvey, US Dept. of Interior, 1980). As the large air pockets or slugs move upstream, the hydraulic capacity of the conduit may be reduced. Closed conduit storm drains should be designed with slopes less than ten percent (10%) to avoid possible loss of hydraulic capacity resulting from slug flow.

B. When physical or design constraints requires a closed conduit storm drain to be designed with a slope (S_o) greater than ten percent (10%) (i.e., $S_o > 0.10$), the larger diameter pipe as determined based on hydraulics and slug flow shall be used in the design. The minimum pipe diameter (D_{min} in inches) for slug flow shall be determined using the following formulas.

Where $0.10 < S_o \leq 0.20$, then $D_{min} = 9\,Q^{0.4}$; or

where $0.20 < S_o < 1.00$, then $D_{min} = 6.6\,(Q^2 / S_o)^{0.2}$.

Computations for slug flow shall be presented on the construction plans for all conduits with slopes greater than ten percent (10%).
Full flow and partial flow velocities for main lines and for laterals exceeding 100 linear feet in length shall not exceed 15 feet per second.

7.4 Calculation of the Hydraulic Grade Line

A. The Bernoulli energy equation shall be used in all hydraulic grade calculations.

B. For closed conduits, the hydraulic grade for the 10-year (10% annual chance) flood and for the 100-year (1% annual chance) flood in sags shall be no higher than one (1.0) foot below the top of curb at inlets and manholes.

C. If the closed conduit is designed for the 100-year (1% annual chance) flood, the hydraulic grade shall be no more than one (1) foot below the top of curb at inlets and manholes.

When determining the beginning hydraulic grade, the engineer shall consider discharge flow conditions, conduit size and shape, existing and future site conditions, future extension of the storm drain, and downstream flow conditions. The beginning hydraulic grade for storm drain calculations shall be at the top of conduit, a known hydraulic grade, critical depth, or by the slope-area method, as appropriate for flow conditions. Hydraulic grade line computations should begin upstream for supercritical flow and downstream for subcritical and full conduit flow.

If a system is discharging directly in to a stream, then the analysis shall begin at the higher of the coincident flood elevation on the receiving stream, the top of conduit, or a calculated hydraulic grade line considering future downstream extension of the storm drain. If the hydraulic grade is based on future downstream extension, information on the future downstream system should be provided.

For storm drains being connected to an existing downstream storm drain, the hydraulic grade line should be tied to the hydraulic grade line for the coincident frequency flood in the downstream storm drain. To determine the starting hydraulic grade for the proposed storm drain, it is necessary to analyze the hydraulics of the downstream drainage system. It is the engineer's responsibility to evaluate all data employed in the analysis, including any data used from existing plans or provided by the City. If assumptions are required to avoid laborious calculations on the downstream drainage system, consult with the Floodplain Administrator. If the existing downstream system is undersized, downstream flooding cannot be increased (this may require detention) and the proposed system should be designed to accommodate future downstream drainage improvements.

The ending hydraulic grade line should be tied to the hydraulic grade line for the same frequency flood in the upstream existing or future storm drain. If the hydraulic grade is based on future upstream extension, information on the future upstream system should be provided to verify the proposed system is adequately sized for the future upstream hydraulic grades. This issue is of particular concern where there are flatland prairies, typical of Grand Prairie topography, located upstream of the project site.

For storm drains being connected to an existing upstream storm drain, the hydraulic grade line should be tied to the hydraulic grade line for the same frequency flood in the upstream storm drain. To determine the starting upstream hydraulic grade, it is necessary to analyze the hydraulics of the upstream drainage system. It is the engineer's responsibility to evaluate all data employed in the analysis, including any data used from existing plans or provided by the City. If assumptions are required to avoid laborious calculations on the upstream drainage system, consult with the Floodplain Administrator. If the existing upstream system is undersized, upstream flooding cannot be increased and the proposed system should be designed to accommodate future upstream drainage improvements.

All hydraulic grade line calculations shall be presented in the tabular format. Examples are provided in Appendix D.

7.5 Pressure Flow

Computation of the hydraulic grade line is to proceed by a direct procedure proceeding from downstream to upstream. The computations shall account for friction and other changes in the hydraulic grade caused by structures, bends, expansions, contractions, junctions, and obstructions.

Computation of the hydraulic grade line for branches or laterals shall begin with the hydraulic grade and velocity of the main line at the center line main and center line branch or lateral intersection and go to the starting branch or lateral hydraulic grade line at the centerline intersection using the energy equation. The computation to start the branch or lateral hydraulic grade line shall account for changes in the main line hydraulic grade caused by velocity changes in going from the main to the lateral and by the lateral junction.

Q_{Design}, $Q_{Capacity}$, V_{Design}, $V^2 / 2g$, and S_f shall be shown on the plans.

Friction losses shall be computed using Manning's formula with an *n* of 0.013 for concrete storm drain conduits. The following formulas shall be used to compute changes in the hydraulic grade caused by friction losses.

$$\Delta HG = L\, S_f$$

and

$$S_f = V^2 n^2 / (2.208\, R^{4/3})$$

and

$$R = A / P$$

Where:

ΔHG = Change in hydraulic grade, ft;
L = Length of closed conduit, ft;
S_f = Friction slope of flow in closed conduit, ft/ft;
V = Velocity of flow in closed conduit, fps;
n = Manning's coefficient;
R = Hydraulic radius, ft;
A = Cross-sectional area of closed conduit, square feet (sq. ft); and
P = Wetted perimeter inside closed conduit, ft.

Changes in the hydraulic grade caused by junctions, structures, enlargements, contractions and changes in the hydraulic grade in going from the main line to a branch or lateral are to be computed with the following formula using the appropriate k_j. The minimum change in hydraulic grade at a junction, structure, enlargement, contraction or going to a branch or lateral from the main shall be 0.00 feet (negative values for change of hydraulic grade shall be rounded up to zero head loss.) Values for k_j shall be obtained from Table 7.7.Note that the change in hydraulic grade at junctions and structures shall be computed independently for the main and each branch or lateral conduit.

$$\Delta HG = (V_2^2 - k_j V_1^2) / 2g$$

where:
ΔHG = Change in hydraulic grade, ft;
V_1 = Velocity of flow in upstream conduit, fps;
V_2 = Velocity of flow in downstream conduit, fps;
k_j = Loss coefficient; and
g = 32.2 = Acceleration of gravity, ft/sec/sec.

Changes in the hydraulic grade caused by bends and obstructions shall be computed with the following formula using the appropriate k_j. Values for k_j shall be obtained from Table 7.7.

$$\Delta HG = k_j V^2 / 2g \quad \text{where: } V = V_1 = V_2 \ ; or$$

$$\Delta HG = [\, V_2^2 - (1 - k_j)\, V_1^2 \,] \ / 2g \ \text{ where } V_1 \neq V_2 \, .$$

The hydraulic grade in inlets shall be the higher grade computed by inlet or pressure control. For inlet control, compute the headwater (HW) per the attached "Chart 1B: Headwater Depth for Concrete Pipe Culverts with Inlet Control". This chart is taken from *Hydraulic Design of Highway Culverts* (FHWA, 2001). Note that the computed headwater per Chart 1B is the depth of flow in the inlet based on the flowline of the storm drain conduit. Pressure control is computed with the following formula.

$$\Delta HG = 1.5\, V^2 / 2g;$$

where:
V = Velocity of pressure flow in the downstream conduit, fps.

Figure 7.5 illustrates hydraulic calculations for a drainage line.

An example of hydraulic calculations for a project on the standard Appendix D forms is given in Appendix D.5, EXAMPLE DESIGN PLAN HYDRAULIC CALCULATION TABLES.
The hydraulic grade in the street at the inlet shall be the inlet opening head as determined by the weir or orifice equations as is appropriate plus the higher of the hydraulic grade inside the inlet as computed above and the inlet crest (for weir control) or centroid of the inlet opening (for orifice control). See Section 6.0 Inlet Design for inlet opening head determination details.

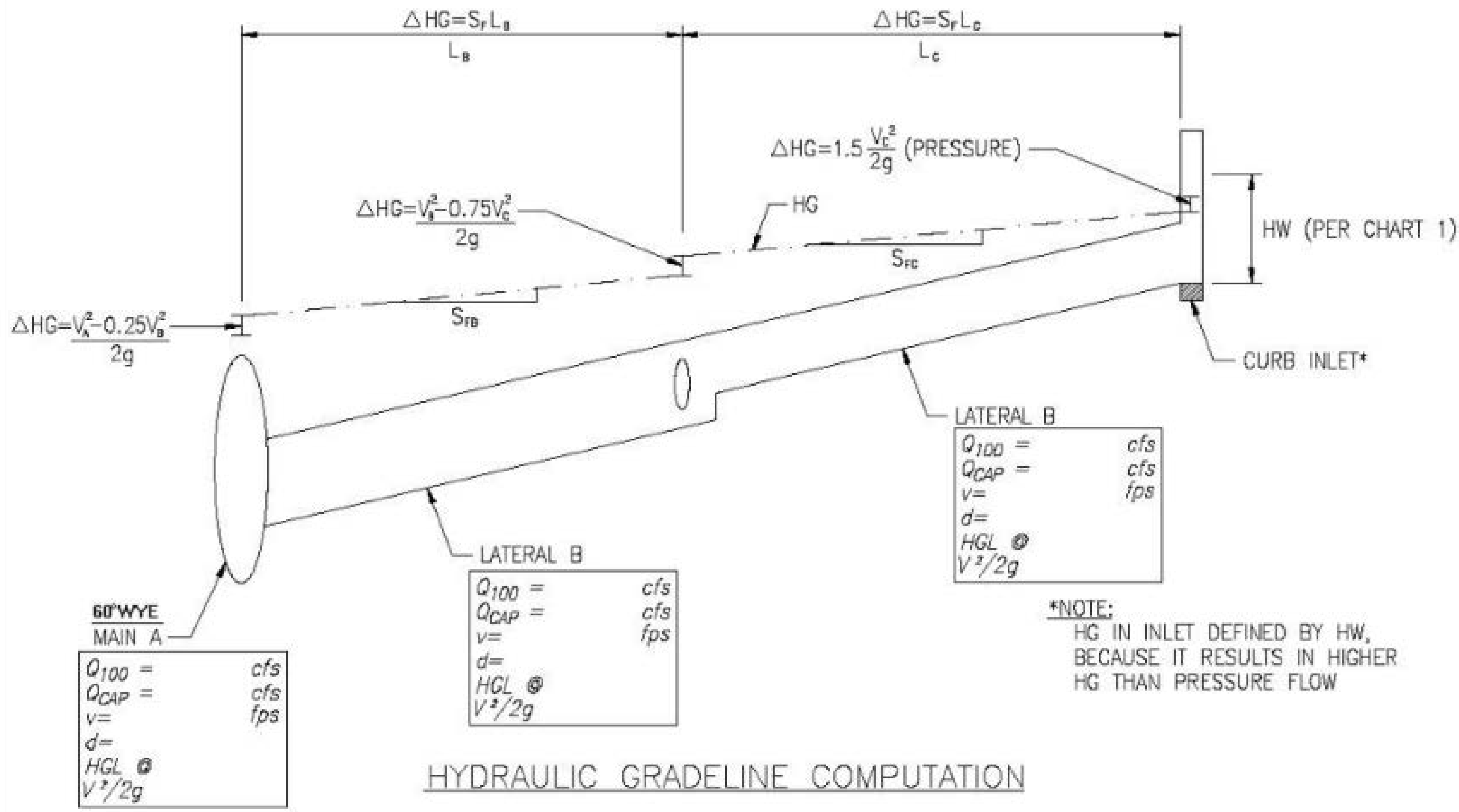


Figure 7.5
Hydraulic Grade Line Computation

7.6 Starting Tailwater Conditions

These guidelines may be used to determine coincident flood flows in a receiving stream at the confluence with a tributary. The flood elevation for the coincident flow in the receiving stream may be used for starting hydraulic grade line calculations for closed storm drain systems. These guidelines may only be used if the receiving stream has an upstream drainage of 200-acres or greater and are limited to closed storm drain systems draining 200 acres or less.

Table 7.6
Receiving Stream Coincident Frequency Flood

Tributary Frequency Flood (years)	**Basin Area Ratio**				
	≤ 3:1	>3:1	>50:1	>500:1	>5,000:1
1	1	1	1	1	1
2	2	1	1	1	1
5	5	2	2	1	1
10	10	5	5	2	1
25	25	10	10	5	2
50	50	25	10	10	2
100	100	50	25	10	2

The coincident frequency flood for a receiving stream is presented in Table 7.6 as a function of the flood frequency in the tributary and the basin area ratio. The basin area ratio is the drainage area of the receiving stream upstream of the confluence divided by the drainage area of the tributary.

An exception to the use of this guideline to determine a coincident flood is for the evaluation of the maximum velocity requirement for a tributary. When evaluating the maximum velocity requirement in a tributary, the flow in the receiving stream downstream of the confluence should be assumed to be the same as in the tributary.

For conduit discharges this evaluation requires the determination of the normal depth of flow and velocity at normal depth for the design storm for the conduit discharge to the receiving stream or swale. Normal depth conduit discharge velocities shall be reduced through invert flattening and or conduit upsizing to provide a non-erosive discharge velocity in accordance with Table 8.1 **Suggested Maximum Permissible Velocities** when the receiving stream is in low flow condition as defined above. The provisions of Section 7.9 **Outfall Design Guidelines** shall be followed in the outfall design for receiving stream low flow cases.

7.7 Minor Losses

Table 7.7
Loss Coefficients

	Loss Coefficient (k_j)
JUNCTIONS	
45° to 60° branch [1]	0.75
90° branch [1]	0.50
2- 45° to 60° branches [1]	0.50
True Y	0.60
MANHOLES [2]	
Straight run	0.75
Straight run w/45° branch [3]	0.50
Straight run w/90° branch [3]	0.25
90° bend	0.00
ENLARGEMENTS	
$A_2 / A_1 = 1.0$	1.00
$A_2 / A_1 = 1.4$	0.90
$A_2 / A_1 = 2.6$	0.65
$A_2 / A_1 = 4.0$	0.48
CONTRACTIONS	
$A_2 / A_1 = 1.0$	1.00
$A_2 / A_1 = 0.7$	0.92
$A_2 / A_1 = 0.4$	0.75
$A_2 / A_1 = 0.3$	0.64
BENDS	
Conduit on curve for 90° bend [4]	
Curve radius = 1.0 diameter	0.50
Curve radius = 4.0 diameters	0.40
Curve radius = 14.0 diameters	0.25
Curve radius ≥ 20.0 diameters	0.00
Bends where the curve radius equals the diameter	
90° bend	0.50
60° bend	0.43
45° bend	0.35
22½° bend	0.20
OBSTRUCTIONS	
$A_{Obstruction} / A_{Conduit} = 0.1$	0.25
$A_{Obstruction} / A_{Conduit} = 0.2$	0.66
$A_{Obstruction} / A_{Conduit} = 0.3$	1.28
$A_{Obstruction} / A_{Conduit} = 0.4$	2.94
$A_{Obstruction} / A_{Conduit} = 0.5$	5.55
INLETS	
At upstream end of conduit [5]	1.50
STARTING BRANCH/LATERALS	
Connection angle with main line	
30°	0.75
45°	0.50
60°	0.25
90°	0.00

[1] When $Q_{Branch} < 0.05\ Q_{Main}$, then $k_j = 1.00$ may be used for calculation of hydraulic grade on main.
[2] Specified values for k_j for manholes may also be used for analysis of existing inlets.
[3] When $Q_{Branch} < 0.05\ Q_{Main}$, then $k_j = 0.75$ may be used for calculation of hydraulic grade on main.
[4] For bends other than 90°, adjust k_j values as $k_j = c\ k_j'$ (k_j' is from the table) where c = 0.85 for a 60° bend, c = 0.70 for a 45° bend, and c = 0.40 for a 22½° bend.
[5] Specified k_j is for pressure control calculation. Use the higher hydraulic grade based on pressure or inlet control.

7.8 Partial Flow in Storm Drains

The following data shall be shown on the plans: Q, V, $V^2 / 2g$, S_f, V_p, and d_p, where V_p = velocity of open channel flow and d_p = depth of open channel flow.

Depth and velocity in partial flow conditions shall be based on the uniform flow assumption using Manning's formula with an *n* of 0.013 for all storm drain pipe or box systems. The friction slope (S_f) of the flow in the closed conduit shall be assumed to be equal to the slope of the conduit.

When open channel flow exists in a conduit downstream of junctions, structures, enlargements, contractions, obstructions, inlets, or changes in slope, it is necessary to evaluate the change in the hydraulic grade to determine if the flow is changing to pressure in the upstream conduit. Downstream open channel flow in a closed conduit transitions to pressure flow in the upstream conduit when the computed change in hydraulic grade (ΔHG) causes the upstream hydraulic grade to be equal to or greater than the top of the upstream conduit.

If partial flow is supercritical in the downstream conduit and the pressure flow friction slope (S_f) in the upstream conduit is equal to or greater than the slope of the conduit (S_o), the starting hydraulic grade for the upstream conduit shall be at the top of the conduit.

If partial flow is subcritical in the downstream conduit and the flow transitions to pressure flow in the upstream conduit, then conservation of energy shall be maintained in the hydraulic grade. Many times it is acceptable to calculate the change in the hydraulic grade with the following formula. The minimum change in hydraulic grade shall be 0.00 feet (negative values for change of hydraulic grade shall be rounded up to zero head loss.)

$$\Delta HG = (V_2^2 - k_j V_1^2) / 2g$$

where:

ΔHG = Change in hydraulic grade, ft;
V_1 = Velocity of pressure flow in upstream conduit, fps;
V_2 = Velocity of open channel flow in downstream conduit, fps;
k_j = Loss coefficient per Table 7.7; and
g = 32.2 = Acceleration of gravity, ft/sec/sec.

When subcritical flow exists in a closed conduit with junctions, structures, enlargements, contractions, obstructions, inlets, or changes in slope, it may be necessary to conduct a backwater analysis to evaluate the hydraulic grade line.

The hydraulic grade in all inlets where the downstream conduit is in partial flow, shall be computed as inlet control (i.e., headwater) per the procedure specified in Chart 1B of Appendix C.

Computations for possible transitions from partial flow to pressure flow shall be presented on the construction plans.

7.9 Outfall Design Guidelines

A. In the design of outfalls, the engineer should consider discharge flow conditions, conduit size and shape, existing and future site conditions, soil characteristics, and flow conditions of the receiving stream.

B. All outfalls shall have a reinforced concrete headwall. Headwalls shall be City or TxDOT standard; in special cases modified City or TxDOT standard or other types of engineered headwalls may be required to control erosion or to meet site conditions.

C. The outfall flowline should match the flowline of the receiving stream. Because of height restrictions, it is sometimes necessary to terminate the conduit at the floodplain fringe and have a channel extend to the stream flowline.

D. All outfalls, energy dissipaters, and erosion control shall have minimum three-foot (3') toe walls at the upstream and downstream ends and engineered toe walls on the side slopes. When an outfall is located at a receiving stream and is not discharging parallel to the receiving stream, three-foot (3') toe walls may be needed on all sides.

E. Velocities shall not exceed those values shown in Table 8.1 in upstream or downstream earthen channels or streams. If the velocity at the outfall is less than the value shown in Table 8.1 and flow is supercritical, a hydraulic jump will occur downstream and erosion control should be provided. Outfalls with velocities that exceed velocities shown in Table 8.1 shall have downstream erosion control extending to a point where the velocity slows to the value shown in Table 8.1. In addition, outfalls with velocities of nine (9) to 12 fps shall have engineered energy dissipaters and outfalls with velocities exceeding 12 fps shall have energy dissipaters designed per *HEC-14 Hydraulic Design of Energy Dissipaters for Culverts and Channels* (FHWA, 2000), *Hydraulic Design of Stilling Basins and Energy Dissipaters* (USBR, 1978), or *Open Channel Hydraulics* (Chow, 1959). In all cases, erosion control shall extend downstream the full length of the jump or to where the velocity is reduced below values show in Table 8.1 and as needed downstream for oscillating jumps (i.e., jumps that occur at $2.5<Fr<4.5$).

F. Design calculations for energy dissipaters shall be provided on the construction plans. Sample specifications are attached for information purposes. Outfall and energy dissipation design procedures can be found in iSWM Sections 4.6 and 4.7 located in Appendix H of this manual. Velocity requirements shall be in accordance with those shown in Table 8.1 of this manual. Loose rock riprap less than 12 inches (18 inches thick) will not be accepted. All riprap shall be grouted per city standards to hold it in place and must have a geotextile at the soil-rock interface. See Section 7.10 for example rock rip-rap gradations.

G. Discharge velocities to be used for the design of outfall terminal structures shall be determined based on tailwater conditions considering only the flow being discharged from the subject conduit. In most cases this will mean that the tailwater discharge velocity is to be based on the greater of the critical flow velocity or supercritical flow velocity within the conduit at the outfall. These velocities can are calculated with the Manning uniform flow formula. For conduits that have changes in slope and the upstream slope is greater than the slope at the outfall, it may be necessary to evaluate the water surface profile to determine the discharge velocity. If there is only a short segment of conduit at the outfall with the smaller slope, the discharge velocity can generally be calculated with the Manning uniform flow formula using the steeper upstream slope (since energy losses through a conduit are small). If there is a longer segment of conduit with the milder slope at the outfall, then evaluation of the water surface profile will generally be needed to evaluate the discharge velocity. Many times a detailed water surface profile analysis can be avoided by connecting a steep upstream conduit to a mild downstream conduit with a manhole (the manhole should be designed to act as a drop structure energy dissipater within the conduit).

H. Erosion control mats shall be placed after seeding on all disturbed earthen areas around outfalls and appurtenances.

I. Construction plans for outfalls shall include a plan, profile, sections, and details for the outfall and appurtenances. The plan should include the outfall, conduit, existing and proposed contours, proposed structures, and limits of the existing and proposed floodplain and floodway. The profile should include the existing profile at the centerline extending downstream of the proposed improvements, proposed centerline to where they tie into the existing ground, and hydraulic grade line. The profile of the outfall should continue to the receiving stream and include the opposite stream bank (if it is not entering the receiving stream parallel to the stream centerline). The profile should be annotated with the hydraulic grade, Q, V, V_{out}, $V^2/2g$, Fr (Froude Number), and Manning's *n*.

J. If the water discharging from the conduit or the water in the receiving stream is subject to Section 404 of the Clean Water Act, a permit must be issued by the US Army Corps of Engineers (Corps) prior to issuance of City permits. Generally outfalls are covered under a Corps' Nationwide Permit and do not require an individual permit. In no case will the City be responsible for construction or maintenance of Section 404 mitigation areas.

K. Impacts to the receiving channel shall be evaluated in accordance with Section 8.3 of this manual.

7.10 Example Specifications

Example Riprap **-** rock riprap shall be dry type with the minimum thickness specified on the plans. Material and work shall be per NCTCOG standard specification item 803.3. Bedding shall be ASTM D67, 3/4 inch, with a mean diameter of approx 3/8 inches and shall be placed a minimum of 6 inches thick. Geotextile shall be placed at all rock riprap-soil and bedding-soil interfaces. Rock riprap and bedding shall have the gradations specified on the plans. Certification of the gradations of the stones and bedding shall be submitted by the manufacturer to the city.

Example Grouted Riprap - grouted riprap shall be 2,000 psi class B concrete with a __" slump with __" to __" stones (typically 10-to 14 inches or 14- to 22 inches) placed in the concrete to a 6-inch (6") depth. Certification of the gradation of the stones shall be submitted by the manufacturer to the city.

Example Gabions - gabions shall be galvanized steel as manufactured by Maccaferri, Terra aqua, modular gabions, or approved equal. Baskets shall be twisted wire mesh (min 2 twists), welded wire will not be accepted. Materials and work shall be per NCTCOG standard specification items 803.2.2 and 803.2.3. Gabion rock shall be hard, durable, and free from structural defects. The rock shall vary from a four- to -eight–inch (4 - 8") diameter. 50 percent (50%) or more of the rock shall be in the five- to six-inch (5 – 6") range and rounded in shape. Geotextile shall be placed at all gabion-soil interfaces. Geotextile shall be as specified on the plans per NCTCOG standard specification item 803.2.3.1. Baskets shall be tied to headwall per detail.

Example Straw blankets - straw blankets shall be placed immediately after hydromulch seeding is completed and all work shall be per manufacturer's recommendations.

Example Rock Riprap Gradations

12" Rock Riprap – 18" Thick

Sieve Size – Square Mesh	% Passing
15"	100
12"	70-100
8"	45-75
3"	10-30
1½"	0-10

18” Rock Riprap – 27” Thick

Sieve Size – Square Mesh	% Passing
21”	100
18”	65-100
12”	35-65
8”	15-40
6”	5-25
4”	0-15

24” Rock Riprap – 36” Thick

Sieve Size – Square Mesh	% Passing
30”	100
24”	65-100
18”	45-75
12”	25-50
8”	10-30
6”	0-15

Bedding

Sieve Size – Square Mesh	% Passing
3”	100
1½”	55-100
3/4”	25-60
3/8”	5-30
No. 4	0-10

8.0 OPEN CHANNELS

When stormwater runoff cannot be carried in a pipe of 72 inches in diameter or smaller, or it is not necessary for the protection of adjacent facilities that the stormwater be carried in an enclosed facility, open channels may be used provided it is mutually agreeable to both the City and the owner.

Earthen Channels

Earthen channels are encouraged throughout the City, particularly for channels draining areas of greater than 4,000 acres, and shall meet all state and federal regulations. When earthen channels are to be preserved, improved or constructed, an application for an earthen channel shall be submitted to the City Engineer prior to approval of the preliminary plat, final plat, or building permit. This application shall contain topographic, hydrologic, and hydraulic information sufficient to properly evaluate the proposal and showing that:

A. All land having an elevation at or below the fully developed 100-year (1% annual chance) flood elevation is contained within an easement dedicated to the public for the purpose of providing drainage. This easement shall include a minimum ten-foot (10’) strip along the limits of the floodplain where maintenance access is required, one side shall be 15 feet wide. This strip area shall have grade not exceeding ten percent (10%) and shall be vegetated with native grasses. The channel easement shall have a minimum hydraulic capacity to accommodate a 100-year (1% annual chance) flood based on a fully developed watershed plus one (1) foot of freeboard, and shall be a minimum of 20 feet wider than the top width of the channel.

B. All channel improvements, such as reshaping, realignment, etc., are protected with sodding, back sloping, cribbing, or other bank protection that is designed and constructed to control erosion from the fully developed two-(2), ten-(10) and 100-year (1% annual chance) fully developed floods by allowing a maximum earthen channel and downstream discharge velocity not to exceed those values shown in Table 8.1. Improved or constructed earthen channels shall have the following minimum specifications:

1. Constructed or improved earthen channels shall consist of a pilot channel that conveys the 2-year fully developed flood with a floodplain area consisting of overbank and side slopes that will convey the 100-year (1% annual chance) fully developed flood plus (1) foot of freeboard;

2. Unless precluded by federal regulations, constructed or improved earthen channels of a permanent intent shall include a paved concrete flume invert with a width of at least 2 feet, an invert depth of at least 3 inches, a 12 inch to 18 inch wide 12 inch thick grouted rip rap on filter fabric border along the flume edges and at least 2 foot deep toe walls along the grouted rip rap edges to provide erosion protection and ensure proper drainage:

3. The pilot channel shall be trapezoidal with maximum 4:1 side slopes, minimum bottom width of six (6) feet, and a bottom width to depth of flow ratio of not less than 2:1 (for the fully developed 2-year (50% annual chance) flood);

4. The floodplain shall have maximum 4:1 side slopes and minimum ten-foot (10’) width of overbank (i.e., area from pilot channel top-of-bank to toe of floodplain side slope) on each side of pilot channel with two- to four-percent (2 – 4%) cross-slopes, see Figure 8.0. Access to channel bottom may require flatter side slopes in floodplain at point locations where required by the City Engineer.

5. Pilot channels may not be required for situations where the earthen channel is solely for the purpose of increasing conveyance under a bridge.

C. Interim check dams shall be provided to control erosion.

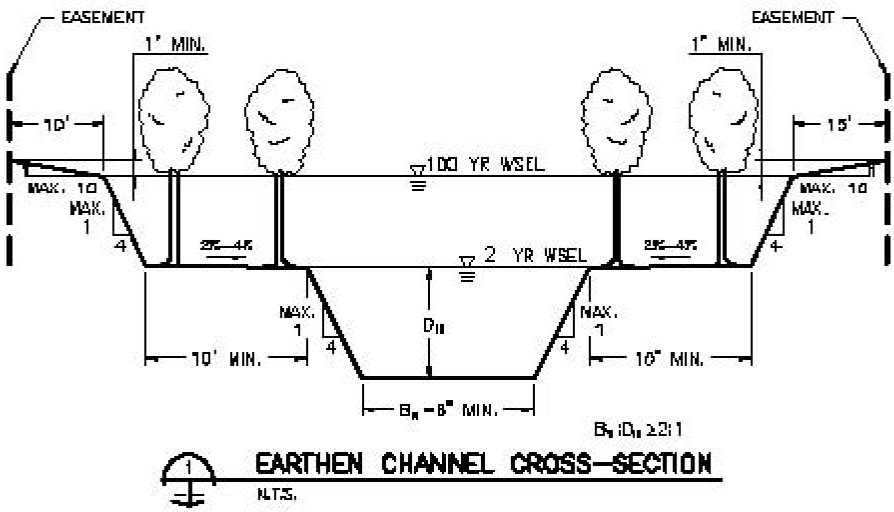


Figure 8.0
Earthen Channel Cross Section

Reinforced Concrete -Lined Open Channels

Concrete-lined channels should be used when the criteria for closed conduit and earthen channels is exceeded. Reinforced concrete-lined open channels are desired where applicable they shall meet all state and federal regulations and shall conform to the following:

A. Channels draining an area of 200 acres or less shall be lined with reinforced concrete in a manner which will contain the fully developed design flood plus one (1) foot of freeboard within the concrete lining.

B. Channels draining an area of 200 acres but not more than 1,000 acres shall be concrete-lined to contain the runoff from a fully developed 25-year (4% annual chance) flood with the balance of the required fully developed design flood contained within grassed slopes no steeper than four (4) horizontal to one (1) vertical and with a minimum of one (1) foot of freeboard.

C. Channels draining an area of 1,000 acres but not more than 4,000 acres shall be constructed with a reinforced concrete pilot channel not less than twelve (12) feet in width and a four (4) inch depressed invert. A stone riprap erosion protection mat four (4) feet wide shall be placed continuously on both sides of the pilot channel. The remainder of the fully developed design flood plus one (1) foot of freeboard shall be contained within earthen side slopes with proper vegetative cover on slopes not steeper than 4:1.

D. Channels draining an area of more than 4,000 acres shall be governed by the criteria for earthen channels.

Concrete pilot channels will be required for all projects under 4,000 acres, unless preempted by the USACE requirements. If pilot channels are removed for compliance, the n values used for modeling purposes shall be for un-maintained vegetation conditions (not less than 0.04). Any concrete-lined open channel that conveys less than 40 cfs is considered a flume. Flumes that convey less than 20 cfs do not require freeboard. All other flumes with subcritical flow must have a minimum of six (6) inches of freeboard and with supercritical flow must have one (1) foot of freeboard.

Alternative Channel Lining

The iSWM *Technical Manual, Hydraulics Category, Section 3.2 Open Channel Design*, included in the **Drainage Design Manual Volume 2 Appendix H**, can be used as a reference for alternative channel lining designs. Stone Rip Rap design Method #2 Gregory is the accepted design procedure by the City of Grand Prairie, Method #1 will not be accepted. Regardless of computed thickness, the minimum allowable rip rap thickness is 18 inches. A properly designed geotextile is required under the bedding layer. Table 4.4-4 will not be accepted for n-values. When modeling open channels, the values presented in Table 4.4-5 are accepted for use in the City of Grand Prairie.

8.1 Design Parameters

A. Channels shall be designed for subcritical flow with a minimum depth of 1.1 x critical depth.

B. Channels shall include engineered inlet structures, outlet structures, and, if applicable, drop structures with erosion control. All inlets, outlets, and drops with velocities that exceed those allowable for project soil conditions shown in Table 8.1 shall have downstream erosion control. If the velocity is less than those shown in Table 8.1 and flow is supercritical at outlets and drops, a hydraulic jump will occur downstream and erosion control should be provided the full length of the jump and as needed downstream for oscillating jumps (i.e., jumps that occur at $2.5<Fr<4.5$). Channel outlets and drops with velocities of 9 to 12 fps shall have engineered energy dissipaters. Channel outlets and drops with velocities exceeding 12 fps shall have energy dissipaters designed per *HEC-14 Hydraulic Design of Energy Dissipaters for Culverts and Channels* (FHWA, 2000), *Design of Small Dams* (USBR, 1973), or *Open Channel Hydraulics* (Chow, 1959). Calculations for energy dissipaters shall be included on the construction plans.

C. All inlet structures, outlet structures, drop structures, energy dissipaters, and erosion control shall have minimum three-foot (3’) toe walls at the upstream and downstream ends and engineered toe walls on the side slopes.

D. Design depth at bends shall include run-up on the outside channel bank. This will typically require hand calculation.

E. Erosion control mats shall be placed after seeding all earthen portions of channels and disturbed areas around channels and streams.

F. Construction plans for channels shall include a plan, profile, sections, and details for the channel and appurtenances. The plan should include the channel, existing and proposed contours, and limits of the existing and proposed floodplain and floodway. The profile should include the existing profile at the centerline and banks extending upstream and downstream of the proposed improvements, proposed centerline and banks to where they tie into the existing ground, and hydraulic grade line. The profile should be annotated with Q, V, $V^2/2g$, d, Fr (Froude Number), and Manning’s *n*.

G. According to the Corps of Engineers Manual EM1110-2-1601 *Hydraulic Design of Flood Control Channels* the following table lists the maximum permissible velocity for average channel velocities.

H. Refer to Section 2.4, Drainage Design Computations, and Appendix L for accepted stormwater models and information tools.

I. Lined channel invert grades should be no less than 0.5 percent (0.5%) if the site terrain permits.

Table 8.1
Suggested Maximum Permissible Velocities

Channel Material	Mean channel Velocity (fps)
Fine Sand	2.0
Course sand	4.0
Fine gravel	6.0
Earth	
Sandy silt	2.0
Silt clay	3.5
Clay	6.0
Grass-lined earth (slopes less than five percent (5%))	
Bermuda grass	
Sandy silt	6.0
Silt clay	8.0
Kentucky bluegrass	
Sandy silt	5.0
Silt clay	7.0
Poor rock (usually sedimentary)	10.0
Soft sandstone	8.0
Soft shale	3.5
Good rock (usually igneous or hard metamorphic)	20.0
Concrete	15.0

8.2 Model Development

A. All conveyance models shall conform to FEMA *Guidelines and Specifications for Flood Hazard Mapping Partners.*

B. The modeling of channels, streams and rivers, bridges, and culverts should follow the procedures and employ the methodologies specified in the *HEC-RAS Technical Manual*, *EM No. 1110-2-1601 Hydraulic Design of Flood Control Channels* (COE, 1994), *HEC-22 Urban Drainage Design Manual* (FHWA, 2001), and *Open Channel Hydraulics* (Chow, 1959).

C. Sections shall be taken downstream, upstream and through the study area to fully analyze the impacts of the project. The post project flood profile should be computed to within 0.01 feet of the pre-project profile both upstream and downstream of the project.

D. The downstream starting water surface shall be at a control (i.e., critical depth), known water surface elevation, or using uniform flow assuming that the slope of the channel is equal to the slope of the energy grade line (S_o). At stream confluences, the starting water surface elevation for the tributary should be normal depth (uniform flow) or the coincident flood elevation on the main stream (the floodplain should be delineated using the backwater from the main stream for the same frequency storm as the channel design until the flood elevation in the tributary controls). If uniform flow is used, the model must start at a distance far enough downstream that an error from ½ S_o to 2 S_o does not affect the water surface elevation through the project or downstream areas that may be impacted by the project.

E. All sections shall be taken perpendicular to the flowlines. This requirement causes some sections, particularly in meandering streams, to be a set of broken lines, not one straight line. In no case shall a section be parallel to the flow at any point on the section.

F. Interpolated sections may not be used. However, to limit field surveying, overbank sections may be taken from a topographic map and the channel may be interpolated between surveyed sections for data that are not ascertainable from the topographic map. City topographic maps may be used for off-site data.

G. Sections should be spaced to account for backwater effects and to properly simulate stream flow conditions. Sections with critical flow will not be accepted, unless it can be demonstrated that the sections are controls within the stream. On streams with steep sloped streambeds the sections should have maximum spacing of about 100 feet, on streams with moderately sloped streambeds sections should have maximum spacing of about 300 feet, and on streams with flat sloped streambeds sections should have maximum spacing of about 500 feet.

H. Care should be taken to determine where ineffective flow areas are within the stream. Typically such areas are located outside levees or berms, just upstream and downstream of culverts and bridges or other constrictions, and at tributaries or side areas that drain in to the stream being modeled. Ineffective flow areas should be blocked out of the appropriate sections and the section labeled to clarify why it does not match topography.

I. Stream banks should be determined based on stream geomorphology. Generally, field observation is required to complete this task. The top-of-bank is typically where vegetation begins, although this is not always the case. Examples of where this rule does not apply are on the outside bank of meanders (where the elevations of the bank should be similar on each side of the stream) and for severely incised channels (where the banks may be only a few feet up the eroded slopes for a small stream).

J. The Floodplain Administrator should be contacted for information on approved floodplain hydraulic models for fully developed watershed conditions available at the Floodplain Administrator's office. If a hydraulic model is not available from the City, then the engineer must develop it. Current effective FEMA models shall be obtained from the FEMA library for use with CLOMR and LOMR submittals. Modeling should be conducted with the current effective FEMA model format, or HEC-RAS computer programs (latest version). For prismatic channels with flows less than 100 cfs and no backwater conditions, uniform flow calculations may be used.

K. Floodplains shall be delineated based on the 100-year (1% annual chance) flood elevation, considering downstream backwater conditions and no maintenance of the floodplain or channels. Modeling shall be through reaches using the downstream discharge. All frequency floods shall represent fully developed watershed conditions. For the West Fork Trinity River (West Fork), the projected 2050 flood represents fully developed watershed conditions. Discharges for FEMA models shall be obtained from the FEMA library.

L. For channelization projects, the channel banks in the model typically should extend to the top of the channel and, if necessary, *n* values should vary spatially across the channel.

8.3 Channel Velocities And Streambank Erosion

A. The maximum flow velocity in earthen streams shall be as shown in Table 8.1 for soil conditions. If velocities already exist above those shown in Table 8.1, the proposed project cannot increase velocities above the existing velocities. Check dams shall be provided to help control erosive velocities.

1. This requirement applies to within, upstream, and downstream of the project and is evaluated for the 2- (50% annual chance) and 100-year (1% annual chance) flood by comparing pre- and post-project velocities. Pre-project velocities are evaluated using pre-project topography and pre-project development conditions. Post-project velocities are evaluated using post-project topography and adjusting the pre-project runoff to account for fully developed conditions at the project site. A rock chute may be one way to mitigate velocity increases. An example can be found below.

2. This requirement includes the analysis of reduced flood storage within floodplain areas.

3. The effect on backwater caused by coincident flow within the main stream may not be considered for velocity calculations on a tributary.

4. For projects where work will be conducted within the drainage way, an additional model shall be developed for post project conditions with *n* values that reflect post construction conditions prior to re-establishment of vegetation. This post construction model shall be analyzed for the 2- (50% annual chance) and 100-year (1% annual chance) floods. The maximum velocities resulting from this post construction model shall be used for engineering design of erosion control measures.

5. The maximum velocity requirement for downstream channels should not be changed by the project; however, if it can be mitigated by demonstrating no loss of valley storage through the project site for the 2-year (50% annual chance) and 100-year (1% annual chance) floods

B. For all earthen streams and channels (including natural channels), the engineer as a minimum shall submit a letter report with supporting information demonstrating the stability of stream meandering, erosion, and slopes. The report will certify that the *proposed drainage easement is of sufficient size to take into account any additional width to accommodate future bank erosion as determined by engineering slope stability calculation.* A future stable 4:1 earthen bank may be assumed in establishing the limits of the drainage easement.

C. If engineering design measures are proposed to mitigate future erosion and a detailed geomorphologic study is not presented, the letter report should, as a minimum, address stabilizing meanders and erosion areas, the streambed eroding to the flowline of the nearest downstream stabilized streambed (i.e., to the nearest culvert, lined channel, etc.) and stable slopes to property lines based on the reduced flowline.

D. Constructed and natural earthen banks shall have engineered 4:1 slopes. Typically maximum slopes of 4:1 are stable in clay soils and reduced slopes in sandy soils.

E. Design of erosion control measures at meanders and bends shall consider the increased velocity on the outside of the bend. This will typically require hand calculation.

9.0 BRIDGE AND CULVERT DESIGN

Culverts and bridges shall be designed to convey the fully developed 100-year (1% annual chance) flood. Headwater and tailwater velocities shall be used for the design of erosion control measures. In general, all culverts and bridges should be analyzed using HEC-RAS.

Where storm drains connect to a culvert, where there are bends in a culvert, or there are obstructions or manholes in a culvert, it may be necessary to conduct hand-calculations and adjust the appropriate model parameters to obtain the correct results. Such culverts and bridges should be labeled to identify why the model parameters have been adjusted.

9.1 Bridge

Bridge design shall be in accordance with the iSWM *Technical Manual, Hydraulics Category, Section 4 Bridge Design*, included in the **Drainage Design Manual Volume 2 Appendix H**, with the following modifications: bridges shall be designed so that the bottom of the lowest beam or span element is no lower in elevation than 1.0 foot above the ultimate 100-year flood hydraulic grade line. The hydraulic grade for the fully developed 100-year (1% annual chance) flood shall be a minimum of 1.0 feet below the lowest obstruction on a bridge. A variance may be issued with written authorization from the Floodplain Administrator. Headwater and tailwater velocities shall be used for the design of erosion control measures. The low gutter or edge of travel way at the lowest point of the creek crossing shall be no lower than two (2) feet above the 100-year (1% annual chance) existing developed flood elevation or one (1) foot above the 100-year (1% annual chance) fully developed flood elevation, whichever is higher unless specifically approved otherwise.

9.2 Culverts

Culverts shall be designed to convey the 100-year (1% annual chance) fully developed flood. Maximum culvert velocity shall be 15 feet/second. The low gutter or edge of travel way at the lowest point of the crossing shall be no lower than two (2) feet above the 100-year (1% annual chance) existing developed flood elevation or one (1) foot above the 100-year (1% annual chance) fully developed flood elevation, whichever is higher unless specifically approved otherwise. All culverts shall have headwalls on the upstream and downstream ends with three-foot (3') toe walls. Culvert control may oscillate from inlet to outlet control, however for this manual the concept of minimum performance applies. This will ensure that the culvert will not operate at a lower level of performance than calculated, but that it may operate more efficiently at times. The culvert design method is based on the use of design charts and nomographs taken from *HEC-5 Hydraulic Design of Highway Culverts* (FHWA, 2001).

Types of Control

A culvert flowing in inlet control has shallow, high velocity flow categorized as "supercritical" flow. For supercritical flow, the control section is at the upstream end of the barrel (the inlet). Conversely, a culvert flowing in outlet control will have relatively deep, lower velocity flow termed "subcritical" flow. For subcritical flow the control is at the downstream end of the culvert (the outlet). The tailwater depth is either critical depth at the culvert outlet or the downstream 100-year (1% annual chance) flood elevation, whichever is higher. In a given culvert, the type of flow is dependent on all of the factors listed in Table 9.2A.

A. Inlet Control

Examples of Inlet control

Figure 9.2A depicts several different examples of inlet control flow. The type of flow depends on the submergence of the inlet and outlet ends of the culvert. In all of the examples, the control section is at the inlet end of the culvert. Depending on the tailwater, a hydraulic jump may occur downstream of the inlet.

Table 9.2A
Culvert Control Factors

Factor	Inlet Control	Outlet Control
Headwater Elevation	X	X
Inlet Area	X	X
Inlet Edge Configuration	X	X
Inlet Shape	X	X
Barrel Roughness		X
Barrel Area		X
Barrel Shape		X
Barrel Length		X
Barrel Slope	*	X
Tailwater Elevation		X

*Barrel slope affects inlet control performance to a small degree, but may be neglected.

Figure 9.2A-A depicts a condition where neither the inlet nor the outlet ends of the culvert are submerged. The flow passes through the critical depth just downstream of the culvert entrance and the flow in the barrel is supercritical. The barrel flows partly full over its entire length, and the flow approaches normal depth in the culvert barrel.

Figure 9.2A-B shows that submergence of the outlet end of the culvert does not assure outlet control. In this case, the flow just downstream of the inlet is supercritical and a hydraulic jump forms in the culvert barrel.

Figure 9.2A-C is a more typical design situation. The inlet end is submerged and the outlet end flows freely. Again, the flow is supercritical and the barrel flows partly full over its entire length. Critical depth is located just downstream of the culvert entrance, and the flow is approaching normal depth at the downstream end of the culvert.

Figure 9.2A-D is an unusual condition illustrating the fact that even submergence of both the inlet and outlet ends of the culvert does not assure full flow. In this case, a hydraulic jump will form in the barrel. The median inlet provides ventilation of the culvert barrel. If the barrel were not ventilated, sub atmospheric pressure could develop which might create an unstable condition during which the barrel would alternate between full flow and partly full flow.

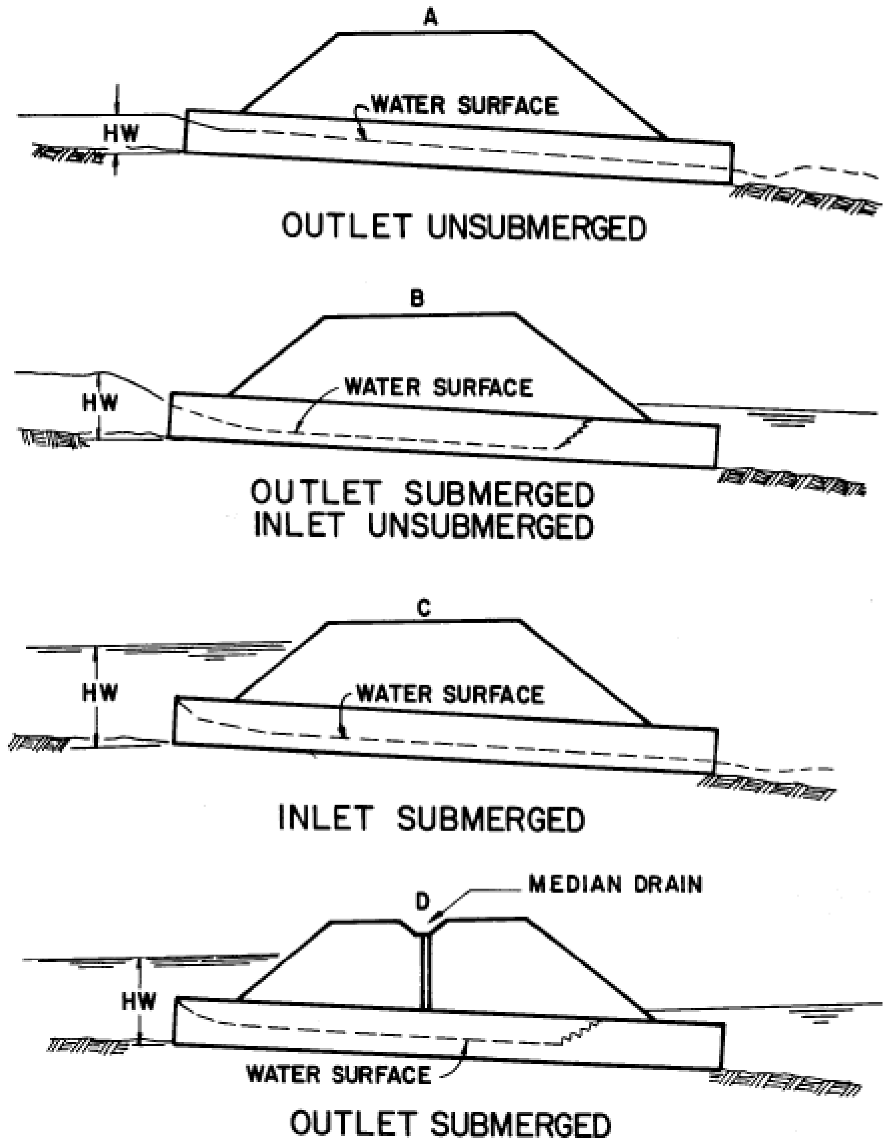


Figure 9.2A
Types of Inlet Control

Hydraulics

Inlet control performance is defined by the three regions of flow shown in Figure 9.2B: unsubmerged, transition, and submergence. For low headwater conditions, as shown in Figure 9.2A-A and 9.2A-B, the entrance of the culvert operates as a weir. A weir is an unsubmerged flow control section where the upstream water surface elevation can be predicted for a given flow rate. The relationship between flow and water surface elevation must be determined by model tests of the weir geometry or by measuring prototype discharges. These test or measurements are then used to develop equations for unsubmerged inlet control flow. The equations developed are as follows:

Form (1) $HW_i/D = H_C/D + K[K_uQ/AD^{0.5}]^M - 0.5S$ (26)

Form (2) $HW_i/D = K[K_uQ/AD^{0.5}]^M$ (27)

Equations (26) and (27) apply up to about $Q/AD^{0.5} = 3.5$

For headwaters submerging the culvert entrance, as shown in Figure 9.2A-C and 9.2A-D, the entrance of the culvert operates as an orifice. An orifice is an opening, submerged on the upstream side and flowing free on the downstream side, which functions as a control section. The relationship between flow and headwater for submerged conditions can be defined as follows:

$HW_i/D = c[K_uQ/AD^{0.5}]^2 + Y - 0.5S$ (28)

HW_i is the headwater depth above the inlet control section invert (ft)
D is interior height of culvert barrel 9ft)
H_C is the specific head at critical depth ($d_c + V_c2/2g$) (ft)
Q is the discharge (ft^3/s)
A is the full cross sectional area of culvert barrel (ft^2)
S is the culvert barrel slope (ft/ft)
K, M, c, Y are constants from Table 9.2A
K_u is 1.0 for English Units

For mitered inlets use +0.7S instead of -0.5S as the slope correction factor.
Equation (28) applies above about $Q/AD^{0.5} = 4.0$

The flow transition zone between the low headwater and the high headwater flow conditions is poorly defined. This zone is approximated by plotting the submerged and unsubmerged flow equations and connecting them with a line tangent to both curves, as shown in Figure 9.2B.

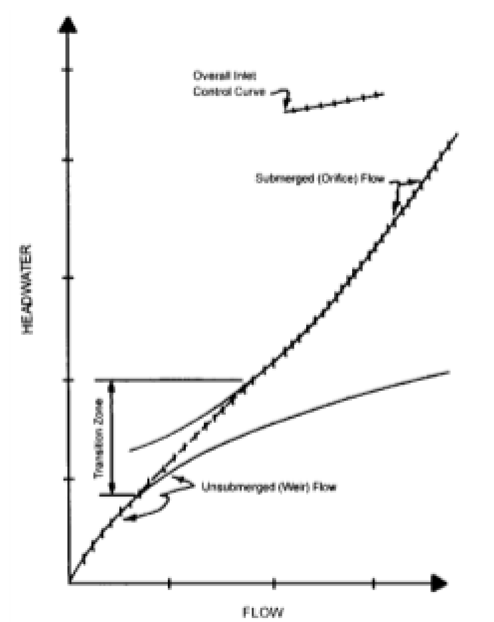


Figure 9.2B
Inlet Control Performance Curves

Table 9.2B
Constants for Inlet Control Culverts [E]

Chart No.	Shape and Material	Nomograph Scale	Inlet Edge Description	Equation Form	Unsubmerged		Submerged		References
					K	M	c	Y	
1	Circular Concrete	1	Square edge w/headwall	1	.0098	2.0	.0398	.67	56/57
		2	Groove end w/headwall		.0018	2.0	.0292	.74	
		3	Groove end projecting		.0045	2.0	.0317	.69	
2	Circular CMP	1	Headwall	1	.0078	2.0	.0379	.69	56/57
		2	Mitered to slope		.0210	1.33	.0463	.75	
		3	Projecting		.0340	1.50	.0553	.54	
3	Circular	A	Beveled ring, 45° bevels	1	.0018	2.50	.0300	.74	57
		B	Beveled ring, 33.7° bevels*		.0018	2.50	.0243	.83	
8	Rectangular Box	1	30° to 75° wingwall flares	1	.026	1.0	.0347	.81	56
		2	90° and 15° wingwall flares		.061	.75	.0400	.80	56
		3	0° wingwall flares		.061	.75	.0423	.82	8
9	Rectangular Box	1	45° wingwall flare d = .043D	2	.510	.667	.0309	.80	8
		2	18° to 33.7° wingwall flare d = .083D		.486	.667	.0249	.83	
10	Rectangular Box	1	90° headwall w/3/4" chamfers	2	.515	.667	.0375	.79	8
		2	90° headwall w/45° bevels		.495	.667	.0314	.82	
		3	90° headwall w/33.7° bevels		.486	.667	.0252	.865	
11	Rectangular Box	1	3/4" chamfers; 45° skewed headwall	2	.545	.667	.04505	.73	8
		2	3/4" chamfers; 30° skewed headwall		.533	.667	.0425	.705	
		3	3/4" chamfers; 15° skewed headwall		.522	.667	.0402	.68	
		4	45° bevels; 10°-45° skewed headwall		.498	.667	.0327	.75	
12	Rectangular Box 3/4" chamfers	1	45° non-offset wingwall flares	2	.497	.667	.0339	.803	8
		2	18.4° non-offset wingwall flares		.493	.667	.0361	.806	
		3	18.4° non-offset wingwall flares 30° skewed barrel		.495	.667	.0386	.71	
13	Rectangular Box Top Bevels	1	45° wingwall flares - offset	2	.497	.667	.0302	.835	8
		2	33.7° wingwall flares - offset		.495	.667	.0252	.881	
		3	18.4° wingwall flares - offset		.493	.667	.0227	.887	
16-19	C M Boxes	2	90° headwall	1	.0083	2.0	.0379	.69	57
		3	Thick wall projecting		.0145	1.75	.0419	.64	
		5	Thin wall projecting		.0340	1.5	.0496	.57	

Table 9.2B
Constants for Inlet Control Culverts (Cont.) [E]

Chart No.	Shape and Material	Nomograph Scale	Inlet Edge Description	Equation Form	Unsubmerged K	Unsubmerged M	Submerged c	Submerged Y	References
29	Horizontal	1	Square edge w/headwall	1	.0100	2.0	.0398	.67	57
	Ellipse	2	Groove end w/headwall		.0018	2.5	.0292	.74	
	Concrete	3	Groove end projecting		.0045	2.0	.0317	.69	
30	Vertical	1	Square edge w/headwall	1	.0100	2.0	.0398	.67	57
	Ellipse	2	Groove end w/headwall		.0018	2.5	.0292	.74	
	Concrete	3	Groove end projecting		.0095	2.0	.0317	.69	
34	Pipe Arch	1	90° headwall	1	.0083	2.0	.0379	.69	57
	18" Corner	2	Mitered to slope		.0300	1.0	.0463	.75	
	Radius CM	3	Projecting		.0340	1.5	.0496	.57	
35	Pipe Arch	1	Projecting	1	.0300	1.5	.0496	.57	56
	18" Corner	2	No Bevels		.0088	2.0	.0368	.68	
	Radius CM	3	33.7° Bevels		.0030	2.0	.0269	.77	
36	Pipe Arch	1	Projecting	1	.0300	1.5	.0496	.57	56
	31" Corner		No Bevels		.0088	2.0	.0368	.68	
	Radius CM		33.7° Bevels		.0030	2.0	.0269	.77	
41-43	Arch CM	1	90° headwall	1	.0083	2.0	.0379	.69	57
		2	Mitered to slope		.0300	1.0	.0463	.75	
		3	Thin wall projecting		.0340	1.5	.0496	.57	
55	Circular	1	Smooth tapered inlet throat	2	.534	.555	.0196	.90	3
		2	Rough tapered inlet throat		.519	.64	.0210	.90	
56	Elliptical	1	Tapered inlet-beveled edges	2	.536	.622	.0368	.83	3
	Inlet Face	2	Tapered inlet-square edges		.5035	.719	.0478	.80	
		3	Tapered inlet-thin edge projecting		.547	.80	.0598	.75	
57	Rectangular	1	Tapered inlet throat	2	.475	.667	.0179	.97	3
58	Rectangular	1	Side tapered-less favorable edges	2	.56	.667	.0446	.85	3
	Concrete	2	Side tapered-more favorable edges		.56	.667	.0378	.87	
59	Rectangular	1	Slope tapered-less favorable edges	2	.50	.667	.0446	.65	3
	Concrete		Slope tapered-more favorable edges		.50	.667	.0378	.71	

Inlet control performance curves are developed using either the inlet control equations shown or the nomographs found in Appendix C. If the design equations are used, both submerged and unsubmerged flow headwaters should be calculated for a series of flow rates bracketing the design flow. The resultant curves are then connected with a line tangent to both curves. Using the combined culvert performance curves, it is easy to determine the headwater elevation for any flow rate, or to visualize the performance of the culvert installation over a range of flow rates.

The inlet control calculations determine the headwater elevation required to pass the design flow through the selected culvert configuration in inlet control. The approach velocity head may be included as part of the headwater, if desired. The inlet control nomographs of Appendix C are used in the design process

B. Outlet Control

Examples of Outlet Control

Figure 9.2C illustrates various outlet control flow conditions. In all cases, the control section is at the outlet end of the culvert or further downstream. For the partly full flow situations the flow in the barrel is subcritical.

Figure 9.2C-A represents the classic full flow condition, with both inlet and outlet submerged. The barrel is in pressure flow throughout its length. This condition is often assumed in calculations, but seldom actually exists.

Figure 9.2C-B depicts the outlet submerged with the inlet unsubmerged. For this case, the headwater is shallow so that the inlet crown is exposed as the flow contracts into the culvert.

Figure 9.2C-C shows the entrance submerged to such a degree that the culvert flows full throughout its entire length while the exit is unsubmerged. This is a rare condition. It requires an extremely high headwater to maintain full barrel flow with no tailwater. The outlet velocities are usually high under this condition.

Figure 9.2C-D is more typical. The culvert entrance is submerged by the headwater and the outlet end flows freely with a low tailwater. For this condition, the barrel flows partly full over at least part of its length and the flow passes through critical depth just upstream of the outlet.

Figure 9.2C-E is also very typical, with neither the inlet nor the outlet end of the culvert submerged. The barrel flows partly full over its entire length, and the flow profile is subcritical.

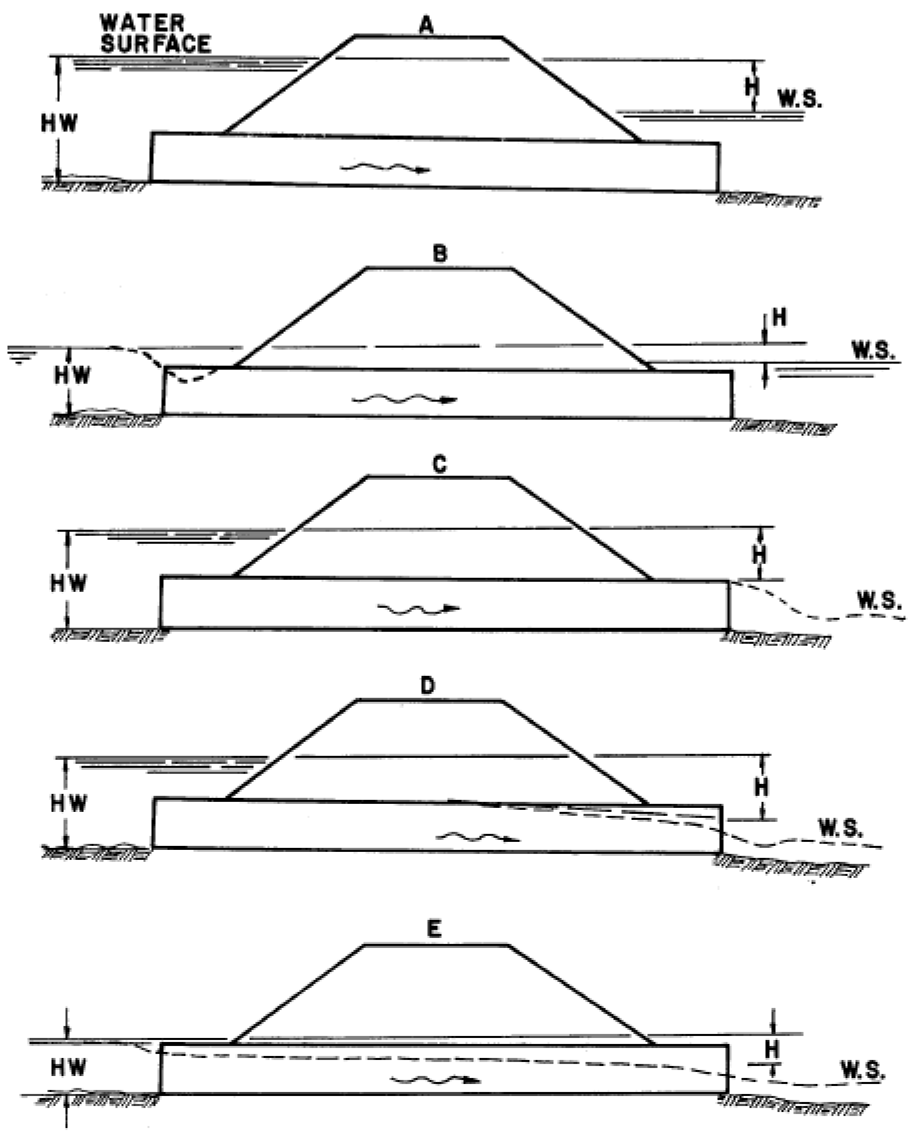


Figure 9.2C
Types of Outlet Control

Hydraulics of Outlet Control

Full flow in the culvert barrel as depicted in Figure 9.2C is the best type of flow for describing the outlet control hydraulics. Outlet control flow conditions can be calculated based on energy balance. The total energy (H_L) required to pass the flow through the culvert barrel is made up of the entrance loss (H_e), the friction Loss (H_f), and the exit Loss (H_o). Other losses, including band losses (H_b), losses at junctions (H_j), and losses at grates (H_g) should be included as appropriate.

$$H_L = H_e + H_f + H_o + H_b + H_j + H_g \quad (1)$$

The barrel velocity is calculated as follows:

$$V = Q/A \quad (2)$$

V is the average velocity in the culvert barrel, (ft/s)
Q is the flow rate (ft/s)
A is the full cross sectional area of the flow (ft^2)

The Velocity Head is:

$$H_V = V^2/2g \quad (3)$$

g is the acceleration due to gravity, 32.2 (ft/s/s)

The entrance loss is a function of the velocity head in the barrel, and can be expressed as a coefficient times the velocity head.

$$H_e = k_e(V^2/2g) \quad (4a)$$

Values of k_e based on various inlet configurations are given in Table 9.2C below.

The friction loss in the barrel is also a function of the velocity head. Based on the Manning equation, the friction loss is:

$$H_f = [29n^2L/R^{1.33}](V^2/2g) \quad (4b)$$

n is the Manning roughness coefficient
L is the length of the culvert barrel (ft)
R is the hydraulic radius of the full culvert barrel=A/p (ft)
A is the cross sectional area of the barrel (ft^2)
p is the perimeter of the barrel (ft)
V is the velocity in the barrel (ft/s)

The exit loss is a function of the change in velocity at the outlet of the culvert barrel. For a sudden expansion such as an end wall , the exit loss is:

$$Ho = [V^2/2g - V_d^2/2g] \quad (4c)$$

V_d is the channel velocity downstream of the culvert (ft/s)

The downstream velocity is usually neglected, in which case the exit loss is equal to the full flow velocity head in the barrel and the equation reduces to:

$$Ho = H_V = V^2/2g \quad (4d)$$

Table 9.2C
Entrance Loss Coefficients[E]

Outlet Control, Full or Partly Full Entrance Head Loss

$$H_e = K_e\left[\frac{V^2}{2g}\right]$$

Type of Structure and Design of Entrance	Coefficient K_e
• Pipe, Concrete	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, sq. cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end	0.2
Square-edge	0.5
Rounded (radius = D/12	0.2
Mitered to conform to fill slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° 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° to 75° 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° to 25° 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

*Note: "End Sections conforming to fill slope," made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections, incorporating a closed taper in their design have a superior hydraulic performance. These latter sections can be designed using the information given for the beveled inlet.

Bend losses, junction losses, grate losses and other losses are discussed in HDC-5 *Hydraulic Design of Highway Culvert.*

Inserting the above relationships for entrance loss, friction loss, and exit loss into Equation (1), the following equation for loss is obtained:

$$H=[1+k_e+(29n^2L/R^{1.33})]*V^2/2g \qquad (5)$$

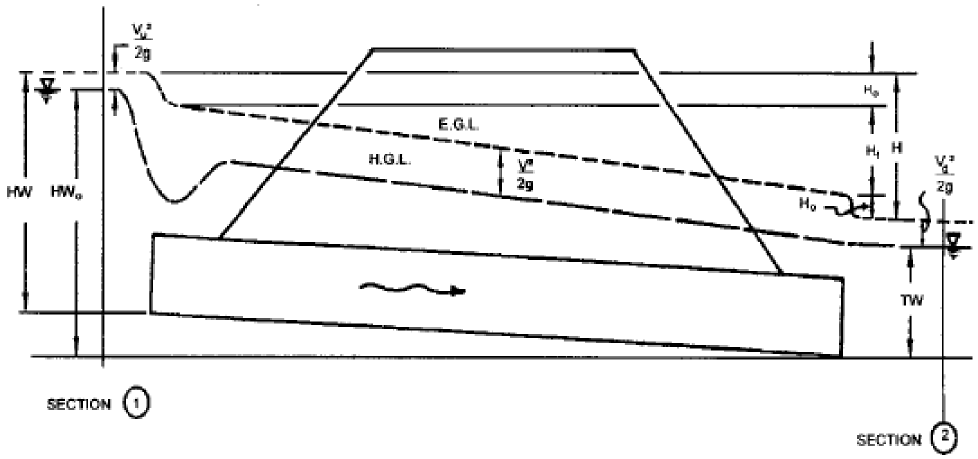


Figure 9.2D
Full Flow Energy and Hydraulic Grade Lines [E]

Figure 9.2D depicts the energy grade line and the hydraulic grade line for full flow in a culvert barrel. The energy grade line represents the total energy at any point along the culvert barrel. HW is the depth from the inlet invert to the energy grade line. The hydraulic grade line is the depth to which water would rise in vertical tubes connected to the side of the culvert barrel. In full flow, the energy grade line and the hydraulic grade line are parallel straight lines separated by the velocity head lines except in the vicinity of the inlet where the flow passes through a contraction.

The headwater and tailwater conditions as well as the entrance, friction, and exit losses are also shown in Figure 9.2D. Equaling the total energy at sections 1 and 2, upstream and downstream of the culvert barrel in Figure 9.2D, the following relationship results:

$$HW_O + V_U^2/2g = TW + V_d^2/2g + H_L \qquad (6)$$

HW_O is the headwater depth above the outlet invert (ft)
V_U is the approach velocity (ft/s)
TW is the tailwater depth above the outlet invert (ft)
V_d is the downstream velocity (ft/s)
H_L is the sum of all losses

Note: the total available upstream energy (*HW*) includes the depth of the upstream water surface above the outlet invert and the approach velocity head. In most instances, the approach velocity is low, and the approach velocity is neglected. However, it can be considered to be a part of the available headwater and used to convey the flow through the culvert.

Likewise, the velocity downstream of the culvert (V_d) is usually neglected. When both approach and downstream velocities are neglected, Equation 6 becomes:

$$HW_O = TW + H_L \qquad (7)$$

In this case, H_L is the difference in elevation between the water surface elevation at the outlet and the water surface elevation at the inlet. If it is desired to include the approach and/or downstream velocities, use Equation (4c) for exit losses and Equation (6) instead of Equation (7) to calculate the headwater.

Equations (1) through (7) were developed for full barrel flow. They also apply to the flow situations shown in Figure 9.2E-B and 9.2E-C, which are effectively full flow conditions. Backwater calculations may be required for the partly full flow conditions shown in Figure 9.2E-D and 9.2E-E. These calculations begin at the water surface at the downstream end of the culvert and proceed upstream to the entrance of the culvert. The downstream water surface is based on critical depth at the culvert outlet or on the tailwater depth whichever is higher. If the calculated backwater profile intersects the top of the barrel, as shown in Figure 9.2E-D, a straight full flow hydraulic grade line extends from that point upstream to the culvert entrance. From Equation (4b) , the full flow friction slope is:

$$S_n = H_f/L = (29\ n^2/R^{1.33})*(V^2/2g)$$

In order to avoid tedious backwater calculations, approximate methods have been developed to analyze partly full flow conditions. Based on numerous backwater calculations performed by the FHWA staff, it was found that a downstream extension of the full flow hydraulic grade line for the flow condition shown in Figure 9.2E pierces the plane of the culvert outlet at a point half-way between the critical depth and the top of the barrel. Therefore, it is possible to begin the hydraulic grade line at a depth of $(d_c+D)/2$ above the outlet invert and extend the straight, full flow hydraulic grade line. The inlet losses and the velocity head are added to the elevation of the hydraulic grade line at the inlet to obtain the headwater elevation

This approximate method works best when the barrel flows full over at least part of its length. When the barrel is partly full over its entire length, the method becomes increasingly inaccurate as the headwater falls further below the top of the barrel at the inlet. Adequate results are obtained down to a headwater of 0.75D. For lower headwaters, backwater calculations are required to obtain accurate headwater elevations.

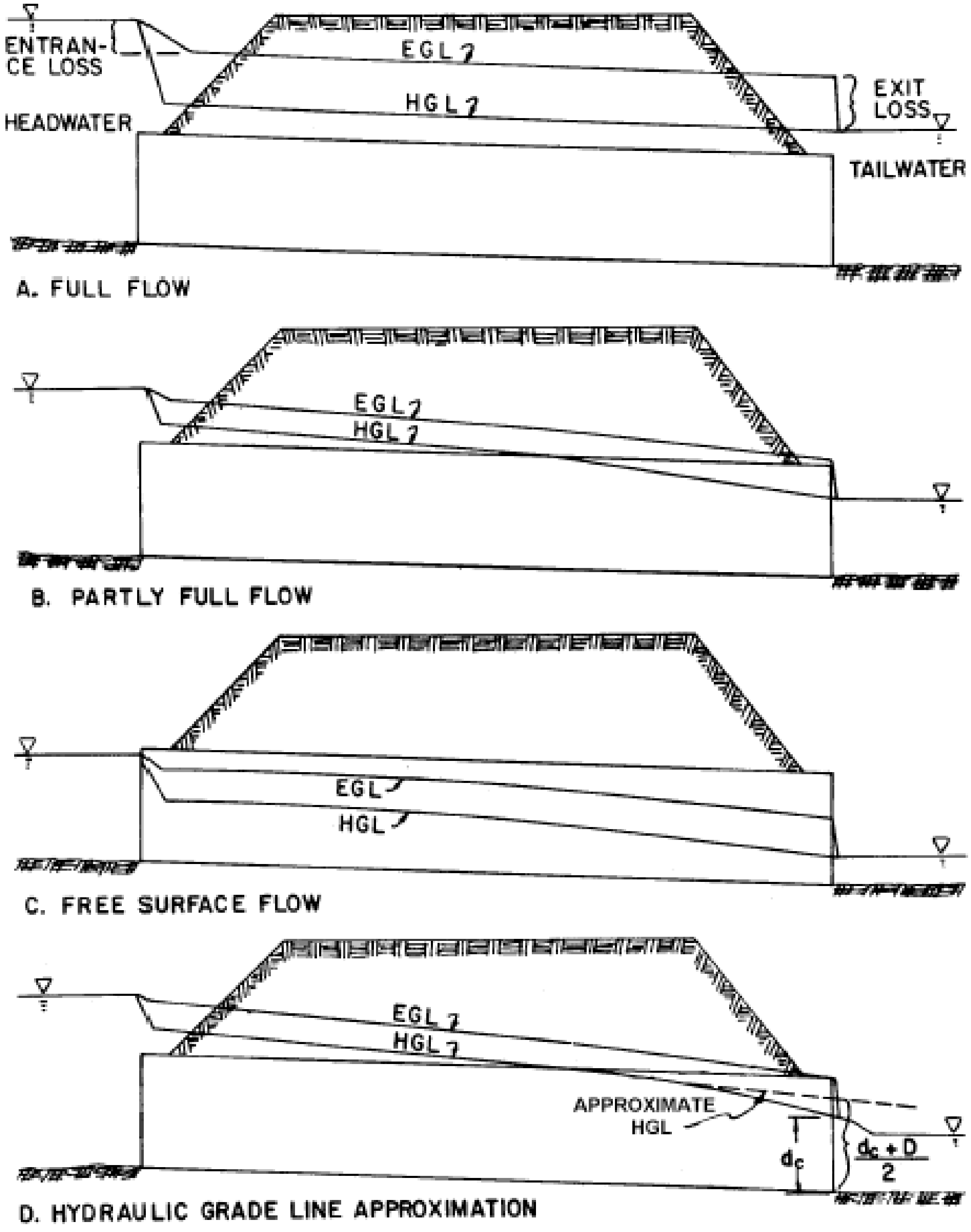


Figure 9.2E
Outlet Control Energy and Hydraulic Grade Lines

The outlet control nomographs in Appendix C provide solutions for Equation (5) for entrance, friction, and exit losses in full barrel flow. Using the approximate backwater method, the losses (*H*) obtained from the nomographs can be applied for partly full flow conditions. The losses are added to the elevation of the extended full flow hydraulic grade line at the barrel outlet in order to obtain the headwater elevation. The extended hydraulic grade line is set at the higher of $(d_c+D)/2$ or the tailwater elevation at the culvert outlet. Again, the approximation works best when the barrel flows full over at least part of its length.

Outlet control performance curves can be developed using Equations (1) through (7), or the nomographs in Appendix C. Flows bracketing the design flow are selected. For these flows, the total losses through the barrel are calculated or read from the outlet control nomographs. The losses are added to the elevation of the hydraulic grade line at the culvert outlet to obtain the headwater.

Design Process

Compare the headwater elevations calculated for the inlet and outlet control. The higher of the two is designated the controlling headwater elevation. The culvert can be expected to operate with that higher headwater for at least part of the time.

If outlet control governs and the headwater depth is less than 1.2D, it is possible that the barrel flows partly full through its entire length. In this case, caution should be used in applying the approximate method of setting the downstream elevation based on the greater of tailwater or $(d_c + D)/2$. If an accurate headwater is necessary, backwater calculations should be used to check the result from the approximate method. If the headwater depth falls below 0.75D, the approximate method should not be used.

Refer to Section 2.4, Drainage Design Computations, and the **Drainage Design Manual Volume 2 Appendix L**, *Computer Models and Information Tools – Computer Models and the iSWM 2006 Design Manual for Site Development* , Appendix G "Storm Water Computer Models and Information Tools", Part G.4 for accepted stormwater models and information tools.

Outlet Velocities

Culvert outlet velocities should be calculated to determine the need for erosion protection at the culvert exit. Culverts usually result in outlet velocities which are higher than the natural stream velocities. These outlet velocities may require flow readjustment or energy dissipation to prevent downstream erosion.

In inlet control, backwater (also called drawdown) calculations may be necessary to determine the outlet velocity. These calculations begin at the culvert entrance and proceed downstream to the exit. The flow velocity is obtained from the flow and the cross-sectional area at the exit (Equation (2)).

An approximation may be used to avoid backwater calculations in determining the outlet velocity for culverts operating in inlet control. The water surface profile converges toward normal depth as calculations proceed down the culvert barrel. Therefore, if the culvert is of adequate length, normal depth will exist at the culvert outlet. Even in short culverts, normal depth can be assumed and used to define the area of flow at the outlet and obtain the outlet velocity (Figure 9.2F). The velocity calculated in this manner may be slightly higher than the actual velocity at the outlet. Normal depth in common culvert shapes may be calculated using a trial and error solution of the Manning Equation. The known inputs are flow rate, barrel resistance, slope and geometry. Normal depths may also be obtained from HDS-2 *Highway Hydrology*.

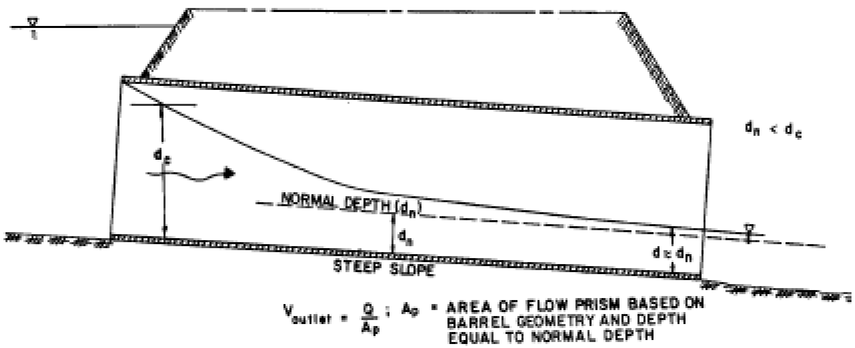


Figure 9.2F
Outlet Velocity – Inlet Control [E]

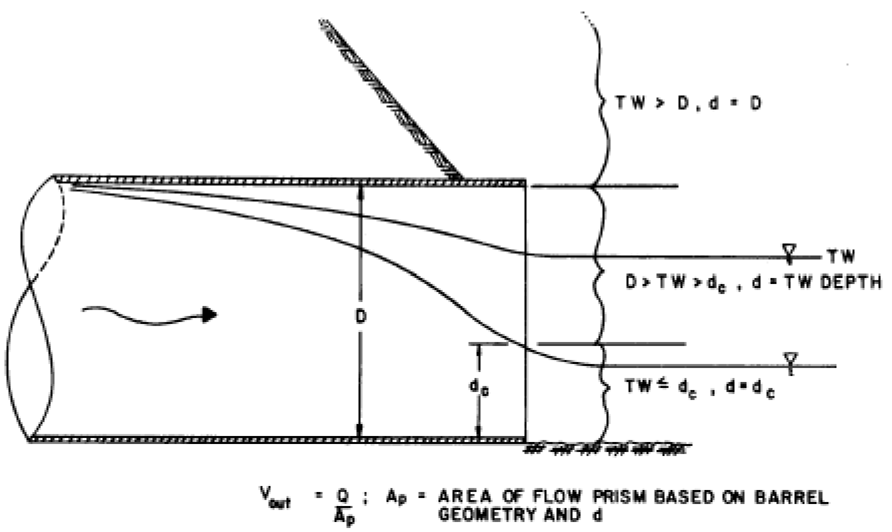


Figure 9.2G
Outlet Velocity – Outlet Control [E]

In outlet control, the cross sectional area of the flow is defined by the geometry of the outlet and either critical depth, tailwater depth, or the height of the conduit (Figure 9.2G).

Critical depth is used when the tailwater is less than critical depth and the tailwater depth is used when tailwater is greater than critical depth but below the top of the barrel. The total barrel area is used when the tailwater exceeds the top of the barrel.

10.0 DETENTION/RETENTION BASIN DESIGN

Detention Basins shall be required when downstream facilities are not adequately sized to convey a design storm based on current City criteria for hydraulic capacity. Detention basins shall not be required if downstream improvements will be constructed in conjunction with the project to safely convey the undetained flows from the project.

Calculated proposed stormwater discharge from a site shall not exceed the calculated discharges from existing conditions, unless sufficient downstream capacity above existing discharge conditions is available.

A properly designed detention basin may mitigate flood impacts caused by increased flows and may be employed to demonstrate no downstream impacts from the proposed project. When there are no downstream impacts and flows are not concentrated, the City Engineer may determine that the intent of UDC Articles 14 and 15 have been met. Detention facilities when required shall be designed such that peak discharges or velocities are not increased when compared to pre-project conditions for the 2- (50% annual chance), 10- (10% annual chance) and 100-year (1% annual chance) floods. Retention ponds may be constructed; however, they may not be considered in the reduction of flood flows except for available storage volume above the normal pool elevation. Dams shall meet TCEQ requirements and, dams subject to such requirements, shall meet or exceed US Army Corps of Engineers' design criteria. The criteria, technique and data to be used to analyze detention basins shall be as approved by the City Engineer. A complete set of all detail calculations shall be submitted to the City Engineer for approval prior to the completion of the plans for the drainage system.

These guidelines are limited to detention basins draining areas less than 500-acres. The city drainage engineer should be contacted for the technique and data to be employed for the design of detention basins draining areas greater than 500-acres. A complete set of all detailed calculations (including electronic copies of all models) are to be submitted to the city drainage engineer for approval prior to completion of construction plans for the drainage system.

The perimeter boundary of a detention/retention pond, or a portion thereof, that is situated within 120 feet of a street right-of-way designated on the Master Transportation Plan as a Collector or Arterial thoroughfare shall be fenced with a 4 foot high wrought iron type fence, equal in design to a Type 2 screening fence as specified in Section 8.26 of the Grand Prairie Unified Development Code. Any portion of said fence for pond that either directly adjoins or is situated within 15 feet of the designated street right-of-way shall contain brick columns. Said brick columns shall equal or exceed the height of the fence and be spaced a maximum 24 feet apart on center along the designated street right-of-way. Otherwise, no brick columns shall be required for fences that do not adjoin, or are situated more than 15 feet from, the designated street right-of-way.

The use of a chain link type fence as a substitute to the above requirement shall be considered by City staff if there are intervening structures or mature landscaping (existing or proposed) that would effectively screen the fence from view along the designated street right-of-way.

10.1 Applicable Design Criteria

A. All ponds and dams shall meet state and federal requirements, including TCEQ regulation 30 TAC Chapter 299, Dams and Reservoirs.

B. Ponds' draining areas greater than 25-acres must be designed with HEC-1 or HEC-HMS using Modified Puls routing, Curve Number (CN) loss rate, 3-hour rainfall depth-duration data, and Dimensionless Hydrograph Method (DHM).

C. Ponds draining areas of 25 acres or less may be designed using either the Modified Rational Method (MRM) or the HEC-1 or HEC-HMS routing method described above. Note that the MRM is not applicable to ponds in series. The Triangular Hydrograph Method (THM) is acceptable for preliminary estimates for ponds with drainage areas less than 200 acres. For the THM, the hydrograph peak shall be determined using the Rational method, the time to peak shall be the time of concentration, and the time of base shall be four times the time to peak. At two times the time of peak the hydrograph ordinate shall be 0.6 times the peak flow and at three times the time of peak the hydrograph ordinate shall be 0.3 times the peak flow. Modified Puls routing with HEC-HMS should be used for design with THM.

D. Note that for pre-developed conditions with an undeveloped site the minimum time of concentration is 20-minutes with a runoff coefficient of 0.30.

E. No increase in discharge from pre- to post-development for the 2- (50% annual chance), 10- (10% annual chance), and 100-year (1% annual chance) floods. Generally, to meet this condition a two-stage outlet, such as a pipe for low flow and Y-inlet for high flow, is required. An emergency spillway shall be provided at the 100-year maximum storage elevation with sufficient capacity to convey the 100-year (1% annual chance) storm assuming blockage of closed conduit portion of the outlet.

F. Drainage from any upstream detention systems shall bypass the pond or shall be fully analyzed using hydrograph techniques. A storm drain system shall be installed that is capable of conveying fully developed flows across the project site whether through an above ground or enclosed system, or a combination thereof.

G. Earthen dams and ponds shall have maximum engineered slopes of 4:1 (H:V). Erosion control blankets shall be placed on the complete pond and dam side-slopes after final grading and seeding. The crown width shall be a minimum of 4 feet for ponds draining areas less than 10 acres and 8 feet for ponds draining areas up to 500-acres. Type of soil, keyway dimensions, and compaction for dam shall be specified with frequency of testing on the construction plans.

H. Include on the plans plots depicting the stage-discharge-storage relationship for the pond and outfall structures with calculations, coefficients, and beginning WSEL assumptions. If a hydrograph method is used, include a plot of the pre-project and post-project inflow and outflow hydrographs for the design floods.

I. Ponds draining less than 10-acres shall have a minimum freeboard of six (6) inches over the 100-year (1% annual chance) flood being discharged over the spillway. Ponds draining 10 to 50 acres shall have a minimum freeboard of one (1) foot. Ponds draining 50 to 500 acres shall have a minimum freeboard of two (2) feet.

J. Erosion control shall be placed at the inlet and outlet in the pond. All storm drain inlets and outlets shall have headwalls. A concrete invert channel shall be installed from the inlet to the outlet in accordance with city standards to provide erosion protection and ensure proper drainage. The concrete flume invert shall have a width of at least 2 feet, an invert depth of at least 3 inches, a 12 inch to 18 inch wide 12 inch thick grouted rip rap on filter fabric border along the flume edges and at least 2 foot deep toe walls along the grouted rip rap edges. Additional flume width, grouted rip rap protection and toe wall depth may be required to accommodate outfall and channel velocities to provide erosion protection.

K. Pond outlet hydraulics shall be analyzed and it shall be determined whether the structures are outlet or inlet control. To determine the outlet hydraulics, it is necessary to analyze the downstream drainage system. If assumptions are required to avoid laborious calculations on the downstream drainage system, consult with the City Engineer.

L. The spillway shall be designed to discharge the 100-year (1% annual chance) flood (assuming the outlet structure is plugged). The spillway should include a downstream energy dissipater and erosion control. Design of energy dissipaters shall be based on the 100-year (1% annual chance) flood assuming the outlet structure is plugged. The discharge must be contained in an easement with a minimum of one (1) foot of freeboard.

M. The discharge from any pond may not be concentrated unless it discharges into a drainage easement or City owned stormwater management area (with City approval). When these downstream conditions do not exist, concentrated discharges are to be broken-up in a flow distribution structure. Such structures generally consist of a long gabion or concrete weir and downstream erosion control matching downstream grades. Typically, a maximum flow of 1.0 cfs per foot (1.0 cfs/ft) of discharge is allowed for the 100-year (1% annual chance) flood in determining the length of the weir. The length of the distribution structure (in the direction of flow) is to be based on a 1:1, length to spread of flow, ratio to ensure expansion of the flow when it reaches the weir to avoid short-circuiting of the flow distribution over the weir. This 1:1 ratio is to be maintained from the location at which the flow from the pond outfall reaches six (6) fps to the weir. Any energy dissipaters for the pond outlet works shall be self-contained to ensure proper operation and the distribution structure should begin at the outfall of the energy dissipater.

N. As a minimum, the area covered by the 100-year (1% annual chance) flood, as well as the dam, outlet structure, and discharge facilities shall be contained within a drainage easement. The plat shall have a note stating that the property owner is solely responsible for the design, operation, and maintenance of the pond and associated appurtenances.

O. Generally, a minimum four-foot (4’) chain link fence with a gate for maintenance shall be installed around the pond, outside the 100-year (1% annual chance) flood pool, for safety.

P. The engineer shall submit a maintenance plan for each pond. It shall include a procedure for removing sediment along with a measurable time interval or sediment depth to require attention. Pond functionality must be maintained while accumulation of sediment occurs. All ponds shall be maintained by the owner. The following note shall be placed on the plat and all applicable design sheets:

The City of Grand Prairie is not responsible for the design, construction, operation, maintenance, or use of any detention basin or underground detention facility and associated drainage easements, hereinafter referred to as “improvement,” to be developed, constructed or used by Owner or his successors, assigns or heirs. Owner shall indemnify, defend and hold harmless the City of Grand Prairie, its officers, employees, and agents from any direct or indirect loss, damage, liability, or expense and

attorneys' fees for any negligence whatsoever, arising out of the design, construction, operation, maintenance, condition, or use of the "improvement," including any non-performance of the foregoing. Owner shall require any successor, assigns or heirs in interest to accept full responsibility and liability for the "improvement." All of the above shall be covenants running with the land. It is expressly contemplated that the Owner shall impose these covenants upon all the lots of this plat abutting, adjacent, or served by the "improvement." It is also expressly contemplated that the Owner shall impose these covenants upon any successor, assigns or heirs in interest the full obligation and responsibility of maintaining and operating said "improvement." Owner shall require any successor, assigns or heirs in interest to accept full responsibility and liability for the "improvement." All of the above shall be covenants running with the land.

Q. All ponds shall be placed in a dedicated drainage easement.

R. All detention basins shall be grassed, landscaped and irrigated in accordance with City standards. Ponds with surface areas of two (2) acres or less must be sodded with grass. Ponds with surface areas greater than 2 acres may be seeded. Grassed areas shall be watered until dense grass is established.

S. A landscaping plan prepared in accordance with the UDC Article 8 shall be required for all stormwater controls. An irrigation plan and grassing (sod or seeding based on criteria in item R above) shall be provided in all stormwater control areas. Species of vegetation selected for landscape plans should be adapted to local climatic conditions and soils to be encountered on the site. Drought resistant vegetation is recommended for typical sites. Further landscaping information and guidance is provided in the City of Grand Prairie UDC Article 8. Appendix K of this manual contains suggested vegetation for stormwater control areas. Where conflicts exist, UDC Article 8 shall have precedence over Appendix K.

T. See Figure 10.1 for illustration of Dry Detention Basin.

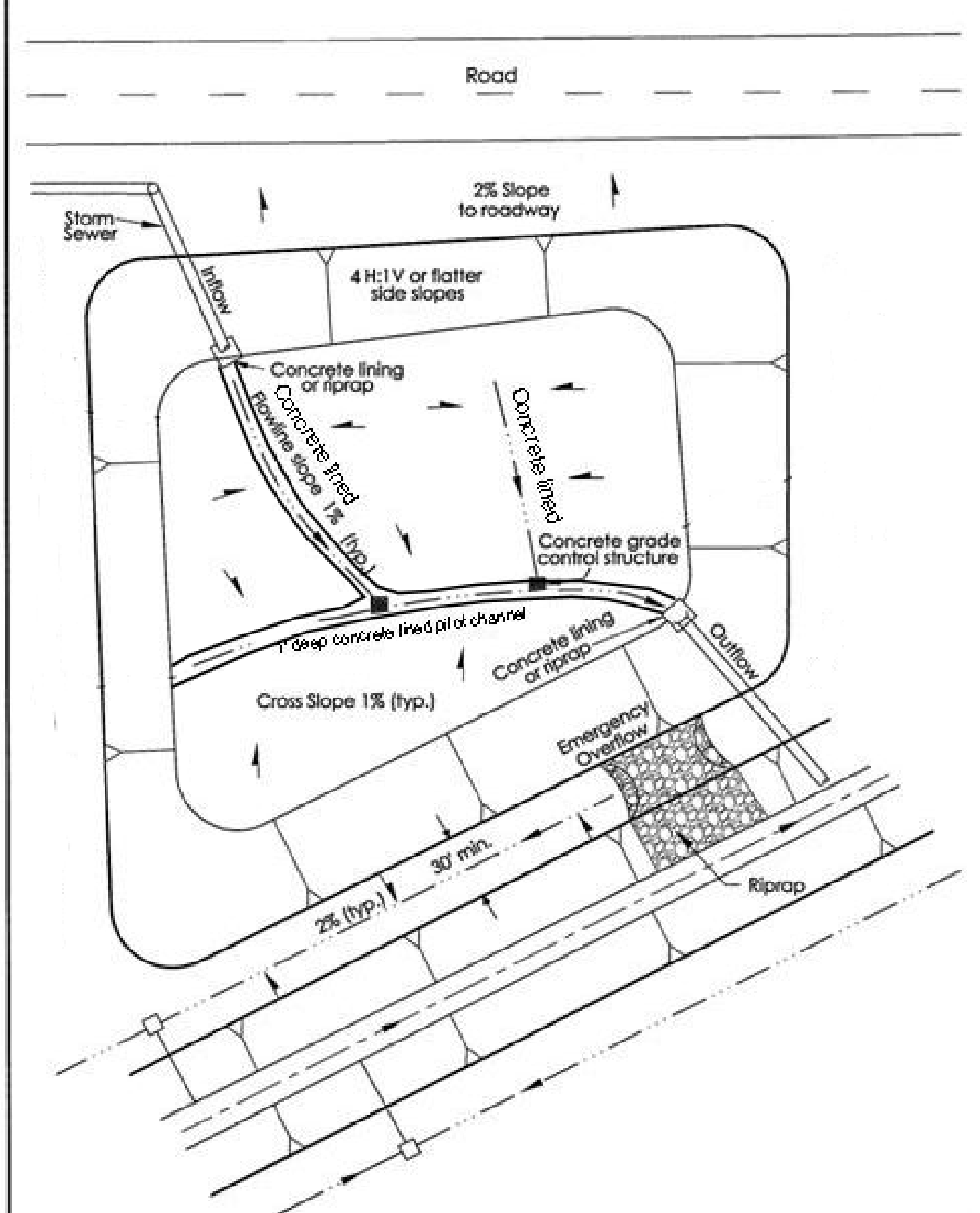


Figure 10.1
WELL-GRADED, "DRY" DETENTION BASIN

Note: All grassed channels shall have a concrete paved flume in accordance with city standards

10.2 Wet Detention Pond

Wet detention basins maintain a permanent pool with additional storage capacity to detain stormwater. The depth of a wet pond is generally seven (7) to ten (10) feet to prevent algal growth, although greater depths are possible with artificial mixing. The objective is to avoid thermal stratification that could result in odor problems or recycling of nutrients. Gentle artificial mixing may be needed in small ponds because they are effectively sheltered from the wind. If properly designed, constructed, and maintained, wet ponds will not only reduce peak stormwater flows, but also improve water quality and can be an attractive feature of a development.

Below are guidelines for wet detention basins in addition to those presented in Section 10.1.

A. If a three-foot (3') depth is not maintained during summer months the pond must be aerated.

B. Provisions shall be made to ensure normal water surface elevation is maintained through the use of ground wells or the City water supply.

C. The normal water surface elevation is clearly shown on the design plans.

D. Ten-foot (10') wide maintenance access shall be provided with a slope of 6:1 or flatter.

E. A mechanism for draining the pond for maintenance and emergency must be specified on plans. The pond shall be drained in less than 4 hours.

F. Anti-flotation calculations must be provided for a riser structure.

G. A debris filter must be provided for all outlet structures.

H. Design shall provide adequate capacity for trapped sediment for five (5) years.

I. A fore bay shall be provided for ponds draining more than three (3) acres.

J. Ten to 25 percent (10 – 25%) of the design storm surface area should be devoted to the fore bay. The fore bay can be distinguished from the rest of the pond by one of several means: differential pool depth, rock filled gabions or retaining wall, or a horizontal rock filter placed laterally across the pond.

K. Use a length to width ratio of at least 4:1, preferably 5:1 to minimize short-circuiting. The inlet and outlet should be placed at opposite ends of the pond where practical baffling shall be installed to direct the water to the opposite end before returning to the outlet. Dead space should be avoided.

L. To minimize water loss by infiltration through the bottom of the pond, an artificial liner, incorporating clay into the soil or compaction should be used. Natural material may be used if a geotechnical report is provided to assure it will not leach out the bottom or sides of the pond.

M. An anti-seep collar should be placed around the outlet pipe when earthen walls are used.

N. The outlet should incorporate an anti-vortex device if the facility serves more than ten (10) acres.

O. The permanent pool volume should be equal to the runoff volume of 1/3 of the two (2) year, 24 hour design storm.

P. The pond bottom should be relatively level to facilitate sedimentation. A mud slope of 0.25 percent (0.25%) should be provided for draining if a pump will not be used.

Q. See Figure 10.2 for illustration of Wet Detention Basin.

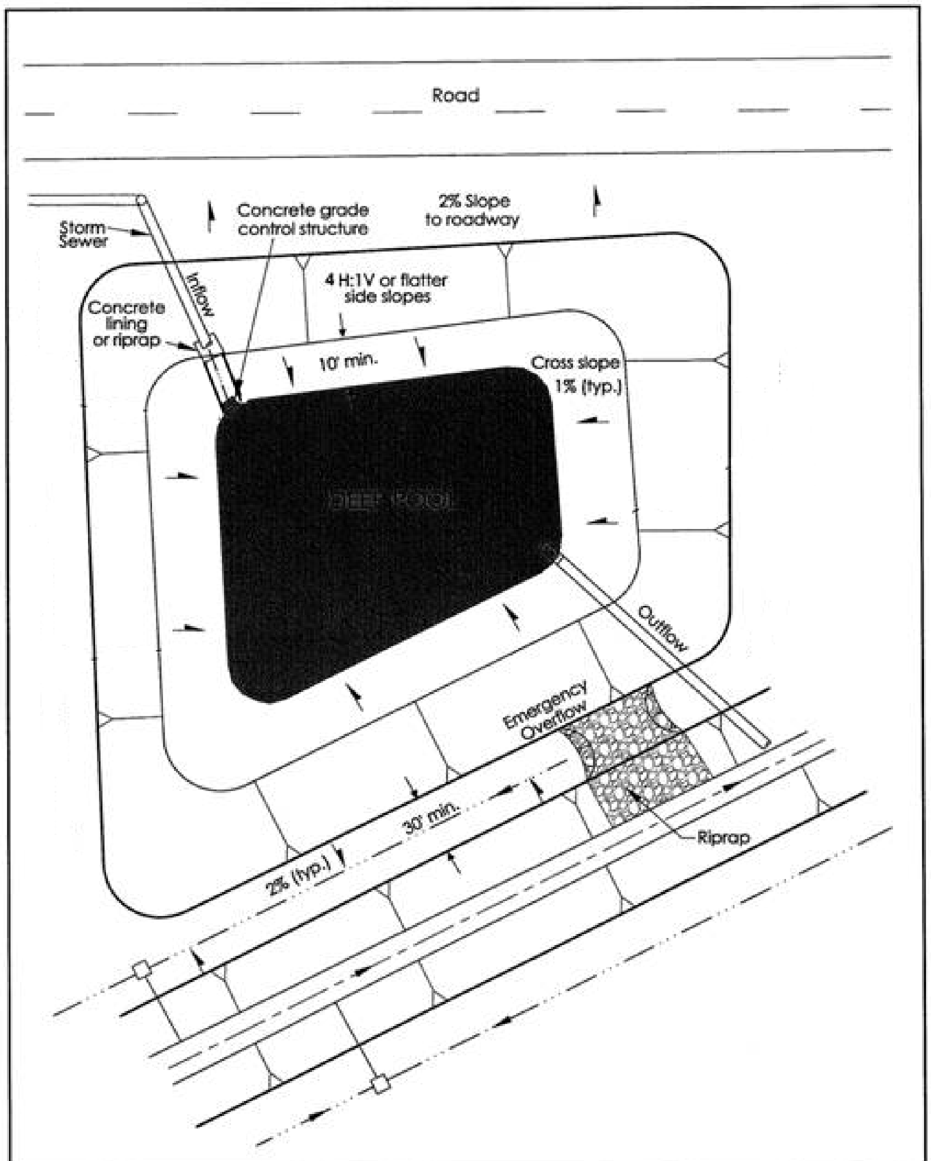


Figure 10.2
"WET" DETENTION BASIN

11.0 FLOODPLAIN/FLOODWAY DEVELOPMENT CRITERIA

All development within 200-feet of the 100-year (1% annual chance) floodplain or floodway shall be approved by the Floodplain Administrator and shall be in compliance with Article 15 "Floodplain Management".

APPENDIX A

CITY CHECKLISTS

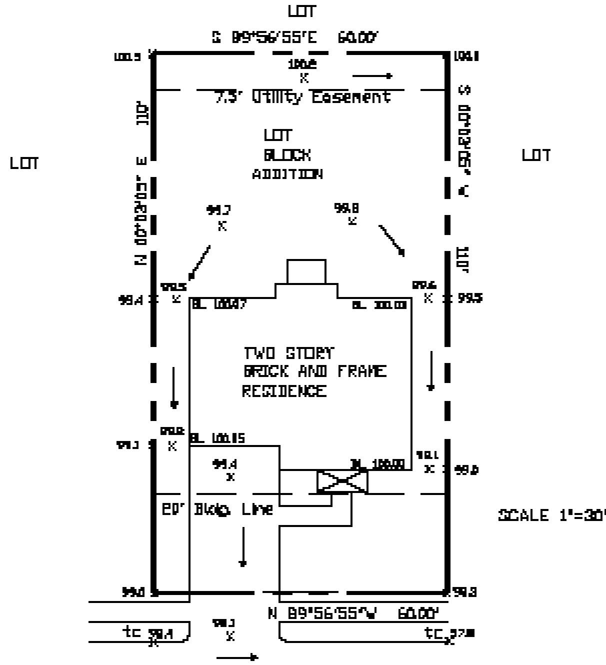
Please refer to the City of Grand Prairie website for the most current drainage-related checklists and applications:

- Private Development Plan Review Checklist
- Clearing/Grubbing/Earthwork Permit Application
- Floodplain Development Permit Application

APPENDIX A.1

PRECISE GRADING CERTIFICATE

PRECISE GRADE CERTIFICATE



STREET NAME

LEGEND

00.00 INDICATES ELEVATION
←—— INDICATES DRAINAGE FLOW DIRECTION
100.00 TOP OF FOUNDATION (RIGHT FRONT CORNER)

NOTES FOR NON-STANDARD STREETS

1. SHOW STREET ELEVATIONS AT CENTERLINE
2. SHOW FLOWLINE ELEVATION OF ALL ADJACENT SWALES

PROPERTY DESCRIPTION

LOT _ BLOCK _ OF____________________________, AN ADDITION TO
THE CITY OF GRAND PRAIRIE,________________ , COUNTY, TEXAS ACCORDING TO THE PLAT OF RECORD
CITY BUILDING PERMIT No:____________________
CITY APPROVED GRADING PLAN FILE NO (COPY ATTACHED)______________________________

I, THE UNDERSIGNED, HEREBY CERTIFY THAT THE ELEVATIONS AND DRAINAGE PATTERN SHOWN HEREON WERE DEVELOPED FROM A SURVEY ON THE GROUND OF THE ABOVE DESCRIBED PROPERTY. PERFORMED ON THE DATE SHOWN AND ACCURATELY AND TRULY DEPICT THE ELEVATIONS AS THEY EXIST ON THE DATE OF THE SURVEY. THAT THE RELATIVE ELEVATIONS AS DEPICTED HEREON ARE CORRECT TO THE NEAREST 1/10 OF A FOOT THAT BASED ON FIELD VERIFICATION THIS PROPERTY WILL DRAIN AS PER THE ORIGINAL GRADING PLAN GRADING PLAN SUBMITTED BY THE ENGINEER, AND AS HEREIN ATTACHED.

(SEAL)

LOTS WITHIN THE RECLAIMED FEMA REGULATORY FLOODPLAIN MUST SATISFY CITY OF GRAND PRAIRIE UNIFIED DEVELOPMENT CODE (15.52) SURVEY REQUIREMENTS BY INCLUDING THE FOLLOWING INFORMATION

1. ELEVATIONS TIED TO CITY OF GRAND PRAIRIE BENCH MARKS
2. ELEVATIONS FOR THE LOWEST FINISHED FLOOR INCLUDING ALL ATTENDANT UTILITIES INCLUDING AIR CONDITIONING AND EMERGENCY POWER GENERATOR PADS AS SHOWN ABOVE AND SATISFYING SAID REQUIREMENTS.

IT IS THE RESPONSIBILITY OF THE OWNER TO MAINTAIN GRADES AS SET BY THE BUILDER, AND AS CERTIFIED HEREIN BY THE SURVEYOR, TO ENSURE THAT PROPER DRAINAGE IN MAINTAINED.

BY: ____________________________

DATE: ____________________________

APPENDIX A.2

GUIDANCE FOR REVISION TO THE CITY OF GRAND PRAIRIE MASTER HYDROLOGY AND HYDRAULIC MODELS

GUIDANCE FOR REVISION TO THE CITY OF GRAND PRAIRIE MASTER HYDROLOGY AND HYDRAULIC MODELS

I. CONSIDERATIONS FOR SUBMITTAL OF REVISED HYDROLOGIC AND/OR HYDRAULIC ANALYSES

In general, detailed hydrologic analyses for the revision of the City of Grand Prairie Master Hydrology or Hydraulic Models (hereafter Master Models) can be initiated for any of the following reasons:

A. To reflect changes in time of concentration

B. To reflect changes in depth duration data

C. To reflect changes in the physical conditions of the watershed;

D. To take advantage of improved hydrologic and/or hydraulic analysis methods; or

E. To correct a demonstrable error in the Master Model.

Regardless of the reason for the revised analyses, the submitter shall provide detailed written documentation of the changes that have been made in the revised analyses and why flood discharges developed in the revision are more accurate than Master Model discharges.

If the reason for the revision analysis is an improved method, the submitter shall provide documentation as to why the alternative method is superior to the methods described in The City of Grand Prairie Drainage Design Manual (hereafter The Manual). The submitter shall obtain approval from The City Engineer to use the alternate method before submitting a revised analysis.

II. REASONABLENESS CRITERIA FOR REVIEW

Prior to detailed review of the proposed revision to a Master Model, the City Engineer shall review the submittal for "reasonableness". The purpose of the reasonableness criteria is to check for completeness, to determine the validity of methodologies selected, and to assess all documented sources of data used in the detailed study.

In determining whether to revise a Master Model, the City Engineer shall consider only such revisions yielding flood discharge values that differ significantly from the effective model, or flood discharges yielding significant differences in base flood elevations. At a minimum the submitter shall:

A. Compare the revised flood discharges to all available flood flow-frequency data that exist adjacent to the study area to ensure compatibility.

B. Document and resolve any discrepancies between proposed revisions and the flood discharges proposed in the City Master Models.

C. Provide a comparison of proposed flood discharges against the USGS regression equation for Dallas County as a good first screening tool. The regression equation does not replace the need for detailed hydrologic modeling

D. Complete the Hydraulic Submittal Checklist (attached)

Once the criteria for flood discharge reasonableness have been satisfied, a full review of the revised model will be conducted. Revision of a Master Model cannot be made if the calculations of the flood discharges are incorrect, even if they yield reasonable results.

If the proposed flood discharges are determined to be unreasonable, the options may include, but are not limited to the following:

A. Provide further justification or documentation that the proposed flood discharges should be used
B. Suggesting an alternative method; or
C. Refining the analysis to obtain more reasonable results.

III. SUBMITTAL CRITERIA FOR DETAILED ANALYSIS FOR REVISION OF THE CITY HYDROLOGIC MODEL

A. BASIN MODEL CRITERIA

In developing a rainfall-runoff model, the submitter performing the detailed hydrologic analysis shall consider the following factors:

1. Where the unit hydrograph method is used, sub watershed drainage areas shall be appropriately defined within the limit such that the unit hydrograph is able to reflect watershed response to changing conditions.
2. Loss rates may be varied when computing different frequency floods. Where HEC-HMS will calculate loss rates for blank fields it is recommended that the field is left blank.
3. Time of concentration or lag computations must reflect the effects of increases in velocities due to channel modifications and urbanization. City of Grand Prairie Drainage Design Manual Section 4.3 has criteria for time of concentration calculations.
4. Rainfall duration, at a minimum, must exceed the time of concentration for the watershed
5. The submitter performing the detailed hydrologic analysis shall determine flood discharges for existing and fully developed land-use conditions.

B. ROUTING CRITERIA

In watersheds with significant storage, hydrologic routing may be needed in estimating the flood discharges. When using hydrologic routing methods requiring a relationship between the water-surface elevation and the cross-sectional area, or the floodplain storage area between cross sections, a hydraulic model shall be submitted as part of the hydrologic analysis. The hydraulic model used to generate rating curves shall be provided by the submitter who performs the analysis along with the hydrologic model.

Where directed by The City Engineer, the submitter shall evaluate the impact of onsite detention basins on the watershed. Uncontrolled detention basins and natural depressions provide uncontrolled flood storage. Detention basins are typically used in developed areas for onsite storage, and these ponds limit post-development peak flow rates from a design storm to pre-development conditions. The ponds may also be used for regional detention based on a master plan for the watershed area of interest.

Usually, an ungated spillway and a low-level, ungated conduit comprise the detention basin outflow structure. The effectiveness of a detention basin in attenuating peak flow rates in the downstream reach depends on the pond’s location in the watershed and its storage and release characteristics. While an onsite detention basin may be effective for a single development site, it may not be as effective for a large urban watershed that has many onsite detention facilities that are not located and designed systematically.

The submitter may use both hydrologic and hydraulic routing methods to route the flow through ponds. Hydrologic routing methods are to be used when the outflow from the pond is not dependent on tail water. The submitter shall use hydraulic routing methods when outflow from the pond is dependent on tailwater conditions. For example, tailwater condition is a control factor where a series of interconnected detention basins are used for flood attenuation in a relatively flat watershed. The hydraulic routing for ponds is often performed with an unsteady-flow model

IV. Hydrologic Review Documentation

A. All criteria discussed under section, I. Considerations for Submittal of Revised Hydrologic Analyses, and, II. Reasonableness Criteria for Review shall be addressed in the detailed study. Depth of discussion is left to the discretion of the submitter, however brevity is desired.

B. The study shall include:

1. Soil maps with the watershed superimposed on it:
2. Detail of analysis of soil types shall be limited to the precision of CN values in the drainage manual.
3. Where soil types differ within a sub basin a weighted value average shall be used to determine CN.
4. All curve numbers shall be based on soil types and typical land usage.
5. Curve numbers which take into account impervious area shall not be used.

C. Percent impervious estimates shall be documented with an exhibit showing impervious area by zoning type for fully developed watershed. Existing condition watersheds require similar documentation. Acceptable documentation for existing condition basins includes city aerials, topos, appraisal district maps and basin reconnaissance. Where land uses differ within a sub basin a weighted value average shall be used to determine impervious area.

D. If the submitter chooses to calculate initial and uniform loss rates rather than leaving blank fields, thus enabling HMS to calculate the rates based on CN and return period, the submitter shall provide calculations as well as a table of comparison documenting the differences between HMS calculation and his.

E. When time of concentration values exceed the minimum specified in the manual documentation shall be provided in the detailed study.

V. Hydraulic Review Documentation

A. All criteria discussed under section, I. Considerations for Submittal of Revised Hydrologic Analyses, and, II. Reasonableness Criteria for Review shall be addressed in the detailed study.

B. The basic review will usually consist of two areas. One area is to satisfy NFIP regulations and FEMA mapping requirements for all analyses. The other area is to satisfy City of Grand Prairie requirements for issuance of a Floodplain Development Permit.

C. Completion of the "Hydraulic Submittal Checklist for Detailed Study for the City of Grand Prairie Flood Mitigation Master plan" is required. Where a checklist item does not apply write NA in the box next to it.

HYDRAULIC SUBMITTAL CHECKLIST FOR DETAILED HYDRAULIC STUDY FOR THE CITY OF GRAND PRAIRIE FLOOD MITIGATION MASTERPLAN		
I.	**Reasonableness Criteria Checklist**	
1	Proper documentation of the study as requested in the guidance document	
2	The most up-to-date topographic information is used in detailed study. If survey data is used to update model geometry then source of data must be cited. Updates to geometric files where vertical coordinates end in whole numbers, or geometry data suggest that a section has been scanned in from GIS must be signed and sealed by a register surveyor prior to approval.	
3	The hydraulic parameters for the submitted flooding sources are spot checked against topographic maps.	
4	Agreement of structures, distances, water-surface elevations, and regulatory floodway widths among the map, profile, and model.	
5	Water-surface profiles for different return period discharges do not cross each other.	
6	Flood discharges used as inputs in the new hydraulic modeling correlate with the hydrologic analysis being used (whether it is new hydrologic analysis or effective hydrologic analysis).	
7	Errors, messages and comments in RAS error box or HEC-2 output file addressed or include explanations why the messages are not applicable.	
8	All frequencies of flood events used to prepare the effective City model are included in the new model.	
9	The one percent (1%) annual-chance water-surface profile has been compared to the bottom slope. For long, straight channels, the water-surface profile shall be parallel to the bottom slope, because open channels tend toward the normal depth, and a problem likely exists if the profile and bottom slope are not parallel.	
10	The water-surface elevations at bridges or culvert sections have been compared to the top-of-roadway elevations. If a bridge or culvert is not designed to carry the one-percent (1%) annual-chance flood discharge, yet the one-percent (1%) annual-chance model shows low flow, a problem likely exists. On the other hand, almost all culverts and bridges are designed to pass the ten-percent (10%) annual-chance flood; if the ten-percent (10%) annual-chance water-surface elevation overtops the bridge or culvert, a problem may exist with the model or profile.	

II.	**Basic Hydraulic Model Criteria**	
1	Cross sections, Manning’s roughness coefficients, transition loss coefficients, and loss coefficients at structures are modeled in accordance with the user’s manual of the model (for detailed analyses), The City of Grand Prairie Drainage Design Manual, and/or the standards of the selected approximate-study method.	
2	Elevations in the new model must tie into the elevations of the effective model exactly or within 0.5 foot lower at the upstream end of the new model.	
3	Elevations in the new model must tie into the elevations of the effective model exactly at the downstream end of the new model.	
4	Floodplain widths at the upstream and downstream ends of the studied reach match those shown on the City effective hydraulic model.	
5	Starting water-surface conditions for the ten-, two-, one-, and 0.2-percent (10%, 2%, 1%, and 0.2%) annual-chance flood runs are appropriate and follow FEMA guidelines.	
6	A floodway run is included in the new model if the effective and/or Master Model included one.	
7	“With floodway” elevations at the downstream end of the new model match those in the effective model.	
8	“With floodway” elevations at the upstream end of a revised model and beyond do not create surcharge values greater than the allowable limits.	
9	Regulatory floodway widths at the downstream and upstream end of the new model match the effective model.	
10	The surcharge throughout the area of study is within acceptable limits.	
11	Starting water-surface conditions and encroachment methodology for the floodway run are appropriate and follow FEMA guidelines.	
12	The revised one-percent (1%) annual-chance water-surface elevation is not higher than the effective one-percent (1%) annual-chance water-surface elevation if the effective regulatory floodway is encroached.	

III.	**Mapping Standards for Detailed Hydraulic Study Submittal**	
1	The results of the new model match the work maps and revised Flood Profiles, including the distances between cross sections, water-surface elevations, regulatory floodway widths, and surcharges.	
2	Work map must show agreement of structures, distances, water-surface elevations, and regulatory floodway widths with the hydraulic model;	
3	All hydraulic structures in the model are reflected on the work maps and vice versa.	
4	The water-surface profiles of different flood frequencies do not cross one another.	
5	The water-surface profiles do not show draw downs (i.e., water-surface elevation at an upstream cross section is not lower than a water-surface elevation at a downstream cross section).	

APPENDIX B

DEFINITIONS OF TECHNICAL TERMS

APPENDIX B

DEFINITIONS OF TECHNICAL TERMS

Unless specifically defined below, words or phrases used in this Article shall be interpreted to give them the meaning they have in common usage and to give this Article its most reasonable application.

Area of Special Flood Hazard: An area having special flood or flood-related erosion hazards and shown on a Flood Insurance Rate Map (FIRM) Zone A, AO, A1-A30, AE, A99, AH, AR, AR/A, AR/AE, AR/AH, AR/AO, AR/A1-A30, V1-V30, VE or V.. Also called the Special Flood Hazard Area (SFHA).

Base Flood: A flood having a one percent (1%) chance of being equaled or exceeded in any given year. Said flood is sometimes known as the 100-year flood.

Corridor Development Certificate (CDC): The permit issued by the City prior to development within the Regulatory Zone of the Trinity River Corridor.

Corridor Development Certificate Manual: The manual developed by the North Central Texas Council of Governments that provides guidance for the CDC process.

Critical Facilities: Includes schools, hospitals, nursing homes, orphanages, penal institutions, police stations, fire stations, emergency ambulance service, emergency communication centers, water and sewage pumping stations.

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, excavating or drilling operations or storage of equipment or materials.

Equal Degree of Encroachment: A standard applied in determining the location of floodway limits so that floodplain lands on both sides of a stream are capable of conveying a proportionate share of flood flows. This is determined by considering the hydraulic conveyance of the floodplain along both sides of a stream for a significant reach.

Elevation Certificate: An administrative tool used by the National Flood Insurance Program (NFIP) to document the elevation of the lowest floor (including basement) of an existing, new or substantially improved building.

Flood or Flooding: A general and temporary condition of partial or complete inundation of 2 or more acres of normally dry land area of 2 or more properties are inundated by water from:

(1) The overflow of inland waters; or
(2) The unusual and rapid accumulation or runoff or surface waters from any source.

Flood Frequency: The average frequency statistically determined for which it is expected that a specific flood level or discharge may be equaled or exceeded.

Flood Insurance Rate Map (FIRM): Official map of a community on which FEMA has delineated the Special Flood Hazard Area, the base flood elevations, and the risk premium zones applicable to the community.

Flood Insurance Study: A compilation and presentation of flood risk data for specific watercourses, lakes, and coastal flood hazard areas within a community. When a flood study is completed for the NFIP, the information and maps are assembled into an FIS. The FIS report contains detailed flood elevation data in flood profiles and data tables.

Floodplain or Flood-Prone Area: Any land area susceptible to being inundated by floodwater from any source.

Floodproofing: Any combination of structural and non-structural additions, changes or adjustments to structures, which reduce or eliminate flood damage to real estate or improved real property, water and sanitary facilities or structures with their contents.

Floodway: The channel of a river or other watercourse and adjacent land areas that must be reserved in order to discharge the base flood without cumulatively increasing the water surface elevation more than a designated height.

Fully Developed Condition Models: Creek-specific studies authorized by the City that model the specific watershed after the watershed has under-gone ultimate development. These models should be used and modified as appropriate and submitted to the City.

Levee: A man-made structure, usually an earthen embankment, designed and constructed in accordance with sound engineering practices to contain, control, or diverts the flow of water in order to reduce the risk from temporary flooding.

Levee System: A flood protection system that consists of levees, floodwalls, and associated structures such as closure and drainage devices, which are constructed and operated in accordance with sound engineering practices.

Lowest Floor: The lowest floor of the lowest enclosed area (including a basement) of a structure. An unfinished or flood resistant enclosure, usable solely for parking of vehicles, building access or storage in an area other than a basement area is not considered a building's lowest floor; provided that such enclosure is not built so as to render the structure in violation of requirements .

Lowest Floor Elevation (LFE): The minimum elevation of the lowest floor of a structure which must comply with the City Ordinance. The LFE must be two (2) or more feet above the base flood elevation.

Manufactured Home: A structure built on a permanent chassis, transported to its site in 1 or more sections and affixed to a permanent foundation. The term "manufactured (mobile) home" recreational vehicles.

Mean Sea Level: The North American Vertical Datum (NAVD) of 1988 (or other datum, where specified), to which base flood elevations shown on a community's Flood Insurance Rate Map are referenced.

New Construction: Structures for which the "start of construction" commenced on or after the effective date of a floodplain management regulation adopted by the City and includes any subsequent improvements to such structures.

Reach: A hydraulic engineering term to describe longitudinal segments of a stream or river.

Standard Project Flood (SPF): The flood having a 0.3 to 0.08 percent (0.3 – 0.08%) chance of being equaled or exceeded in any given year. The SPF generally has a volume discharge of approximately double the 100-year (1% annual chance) storm and water surface elevation of four (4) to seven (7) feet higher that the 100-year (1% annual chance) flood.

Structure: A walled and roofed building, including a gas or liquid storage tank, which is principally above ground, as well as a manufactured home. The terms "structure" and "building" are interchangeable in the NFIP.

Subdivision: The division of any lot, tract, or parcel of land into two (2) or more lots or sites for the purpose of sale or building development, whether immediate or future. Said term also includes the re-subdivision of any lot, tract, or parcel of land.

Substantial Improvement: Any repair, reconstruction, rehabilitation, addition, or other improvement of a structure, the cost of which equals or exceed 50 percent (50%) of the market value of the structure before the first alteration of any wall, ceiling, floor, or other structural part of the building commences, whether or not that alteration affects the external dimensions of the structure. The term does not include any project for improvement of a building to correct existing state or local code violations or any alteration to a "historic building", provided that the alteration will not preclude the building's continued designation as a "historic building".

Trinity River Corridor: The area defined by the bed and banks of the Trinity River and the adjacent river floodplain within the City of Grand Prairie. Also referred to as Corridor.

UDC: City of Grand Prairie Unified Development Code

Variance: A grant of relief by the City from the terms of its floodplain management regulations. (For full requirements, see Section 60.6 of the National Flood Insurance Program regulations.)

Violation: The failure of a structure or other development to be fully compliant with the City of Grand Prairie's floodplain management regulations. A structure or other development without the elevation certificate, other certifications, or other evidence of compliance required in the National Flood Insurance Program Section 60.3 (b)(5), (c)(4), (c)(10), (d)(3), (e)(2), (e)(4), or (e)(5) is presumed to be in violation until such time as that documentation is provided.

Water Surface Elevation: The height, in relation to the North American Vertical Datum (NAVD) of 1988 (or other datum, where specified), of floods of various magnitudes and frequencies in the floodplains of riverine areas.

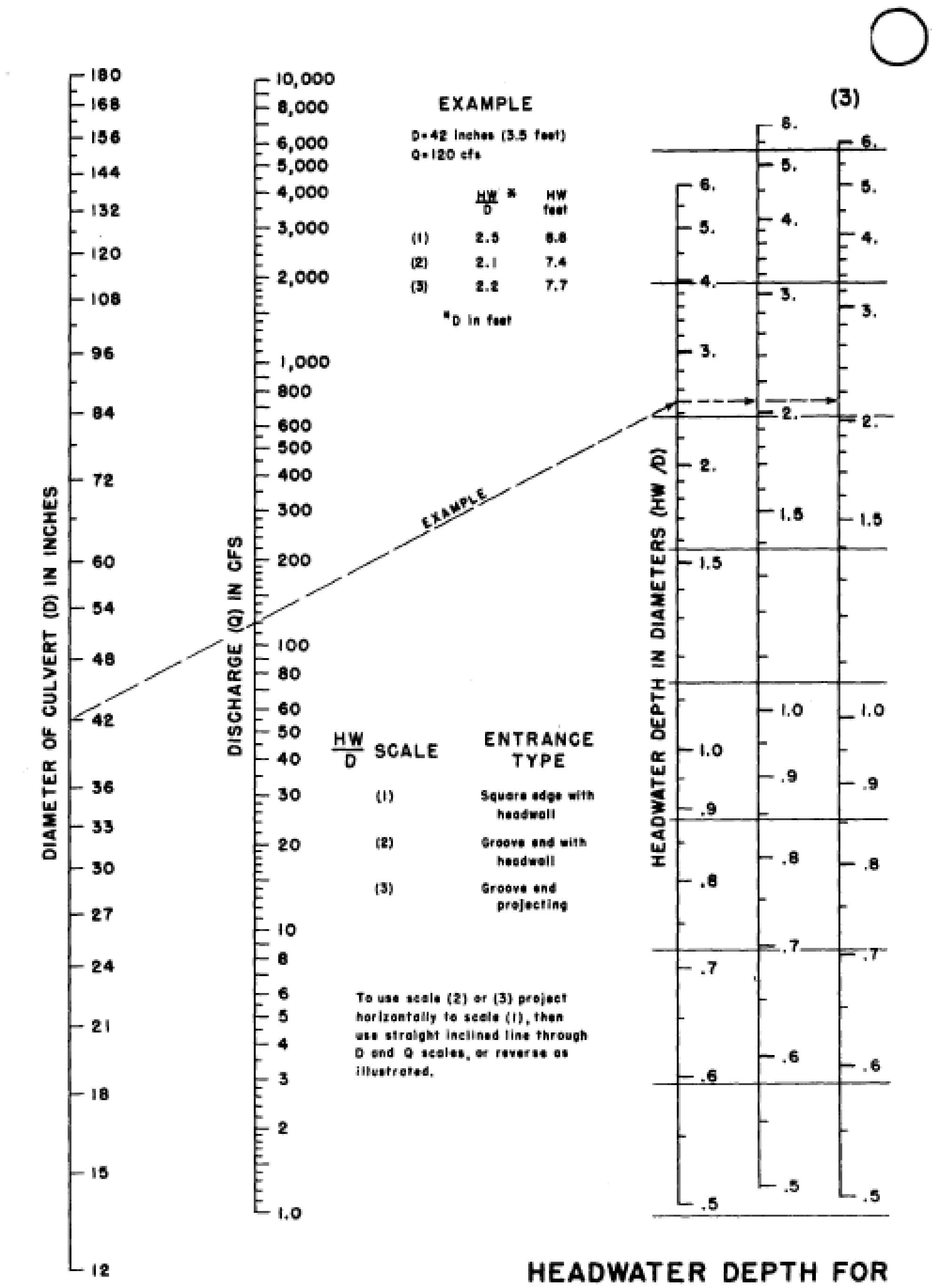
APPENDIX C

DESIGN CHARTS FOR CULVERTS

APPENDIX C

DESIGN CHARTS FOR CULVERTS [E]

CHART 1B

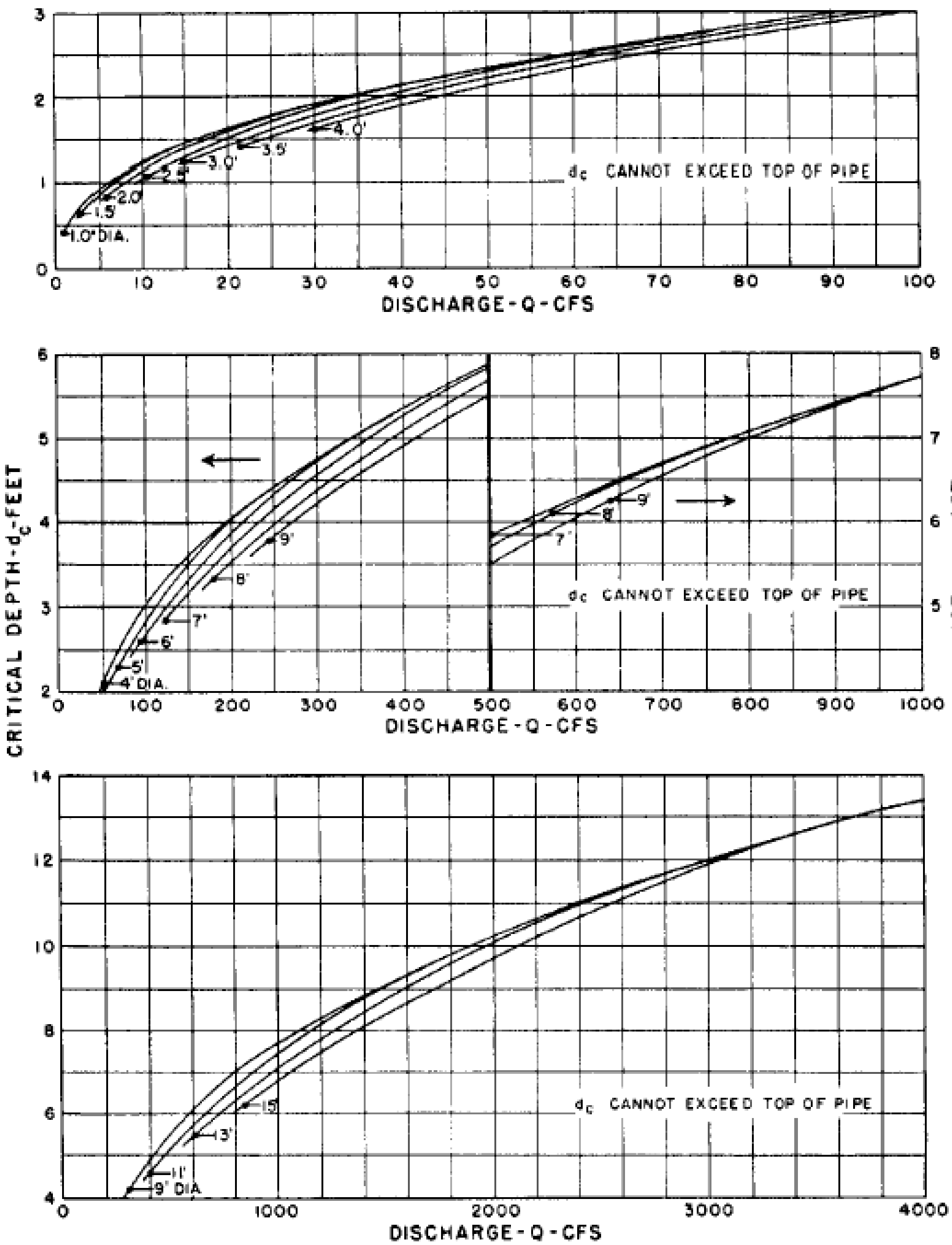


HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL

HEADWATER SCALES 2&3 REVISED MAY 1964

BUREAU OF PUBLIC ROADS JAN. 1963

CHART 4B

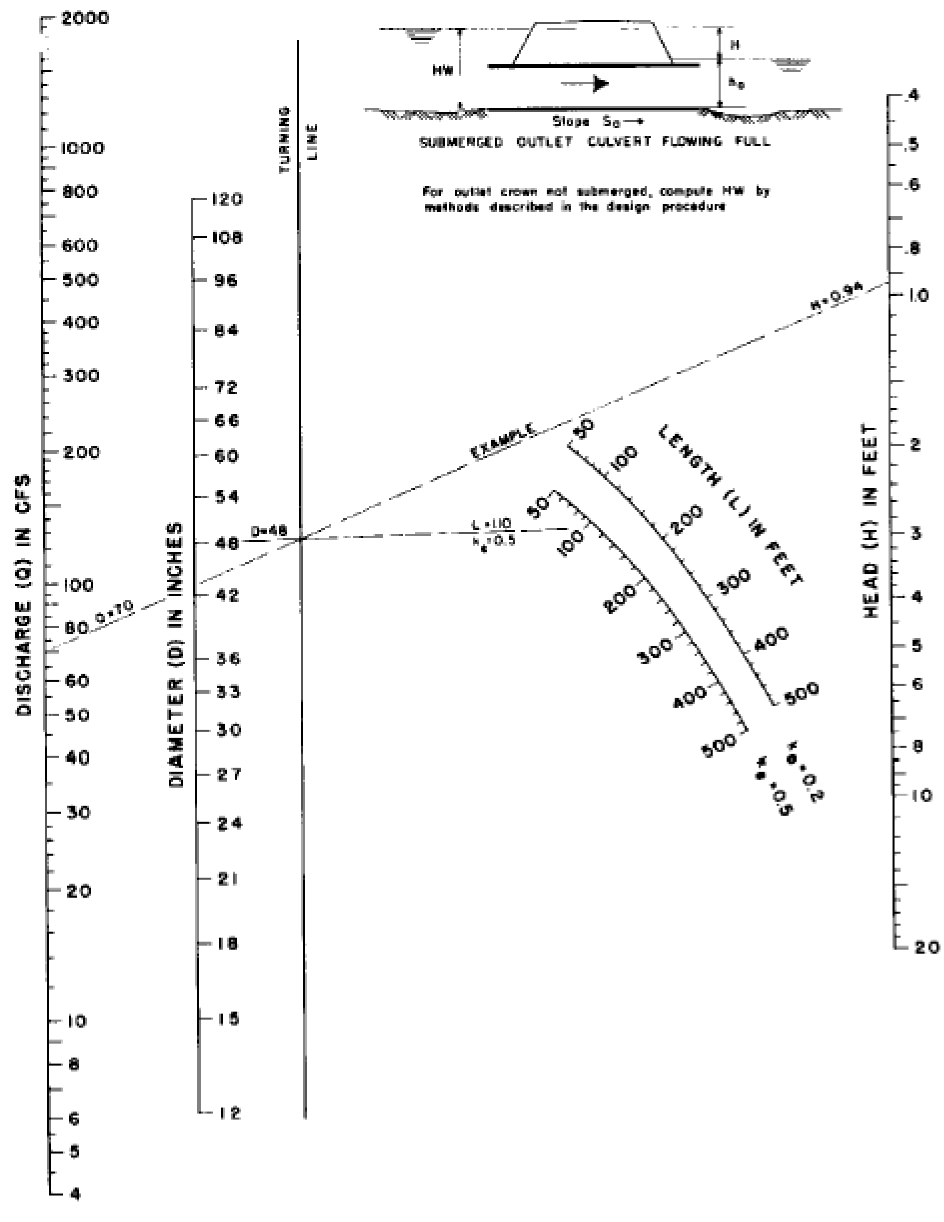


BUREAU OF PUBLIC ROADS

JAN. 1964

CRITICAL DEPTH
CIRCULAR PIPE

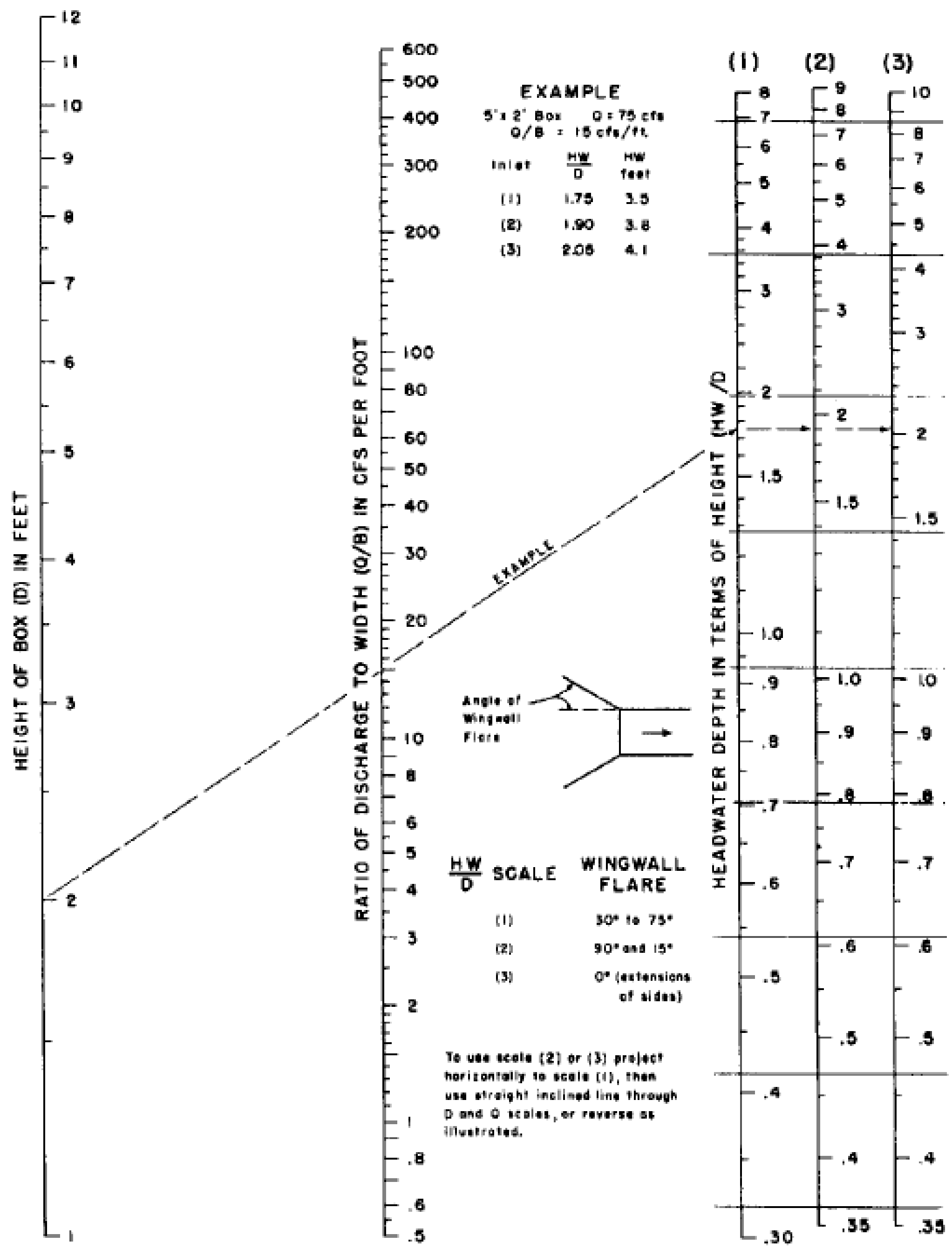
CHART 5B



HEAD FOR
CONCRETE PIPE CULVERTS
FLOWING FULL
n = 0.012

BUREAU OF PUBLIC ROADS JAN. 1963

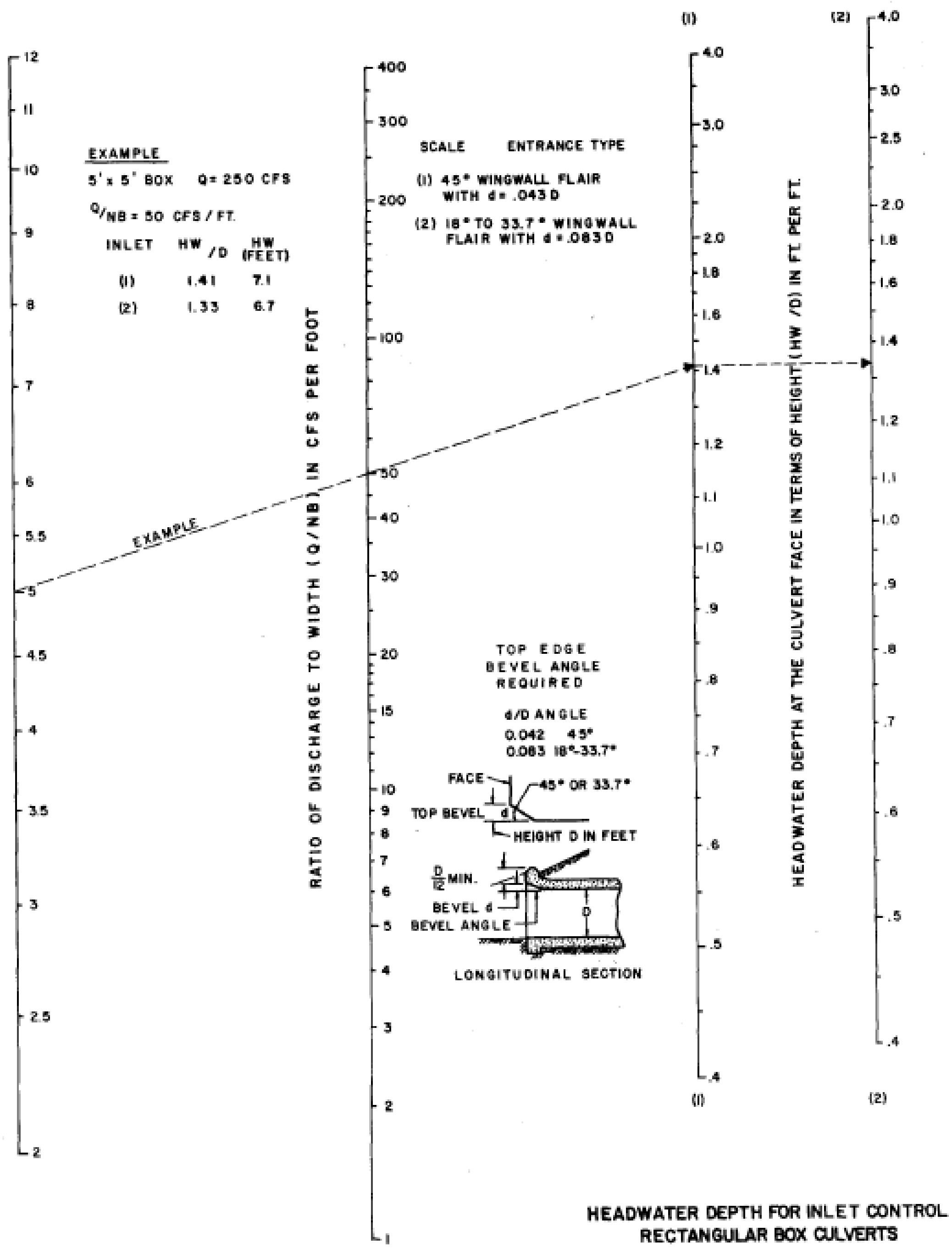
CHART 8B



HEADWATER DEPTH
FOR BOX CULVERTS
WITH INLET CONTROL

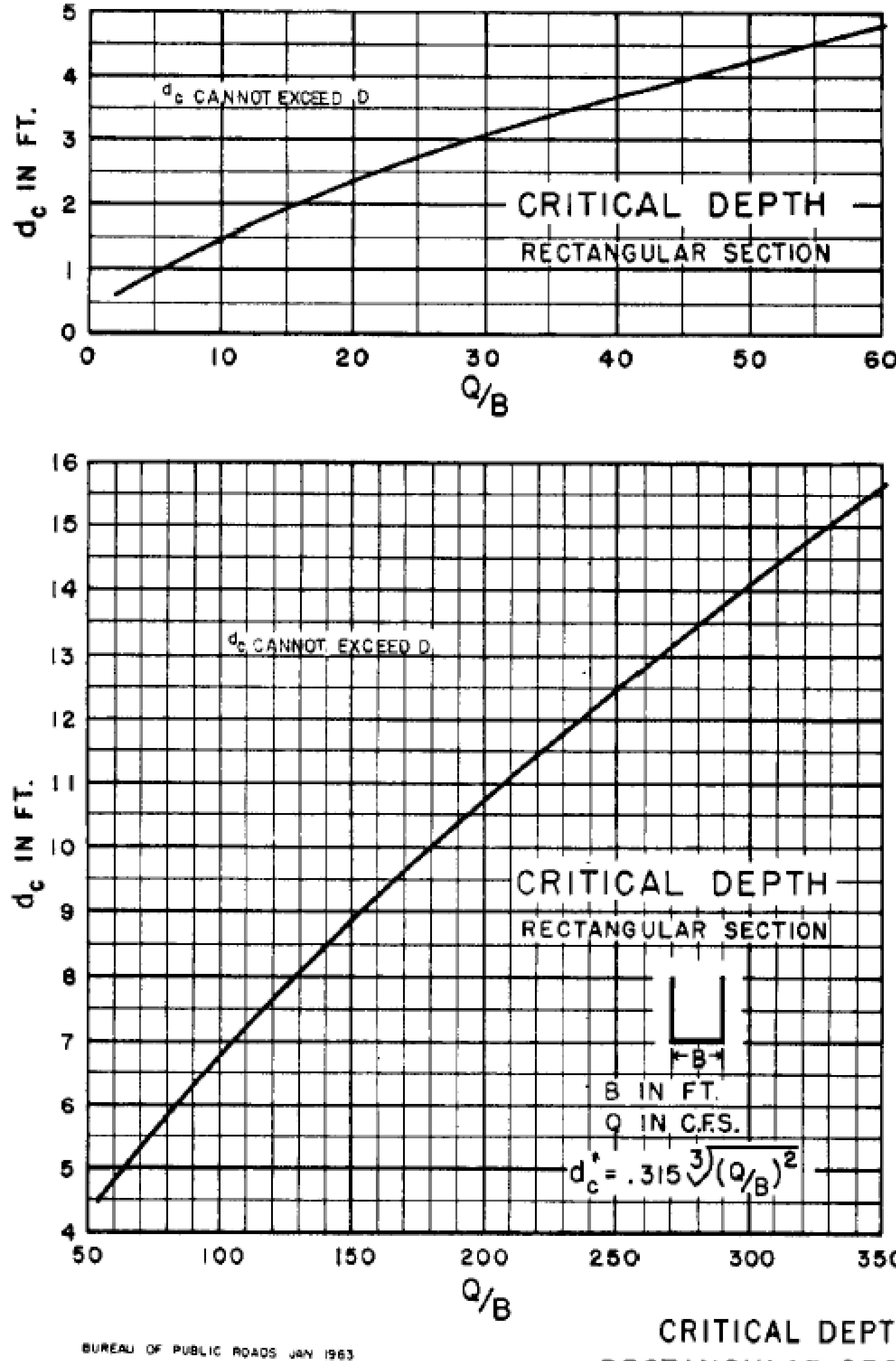
BUREAU OF PUBLIC ROADS JAN. 1963

CHART 9B



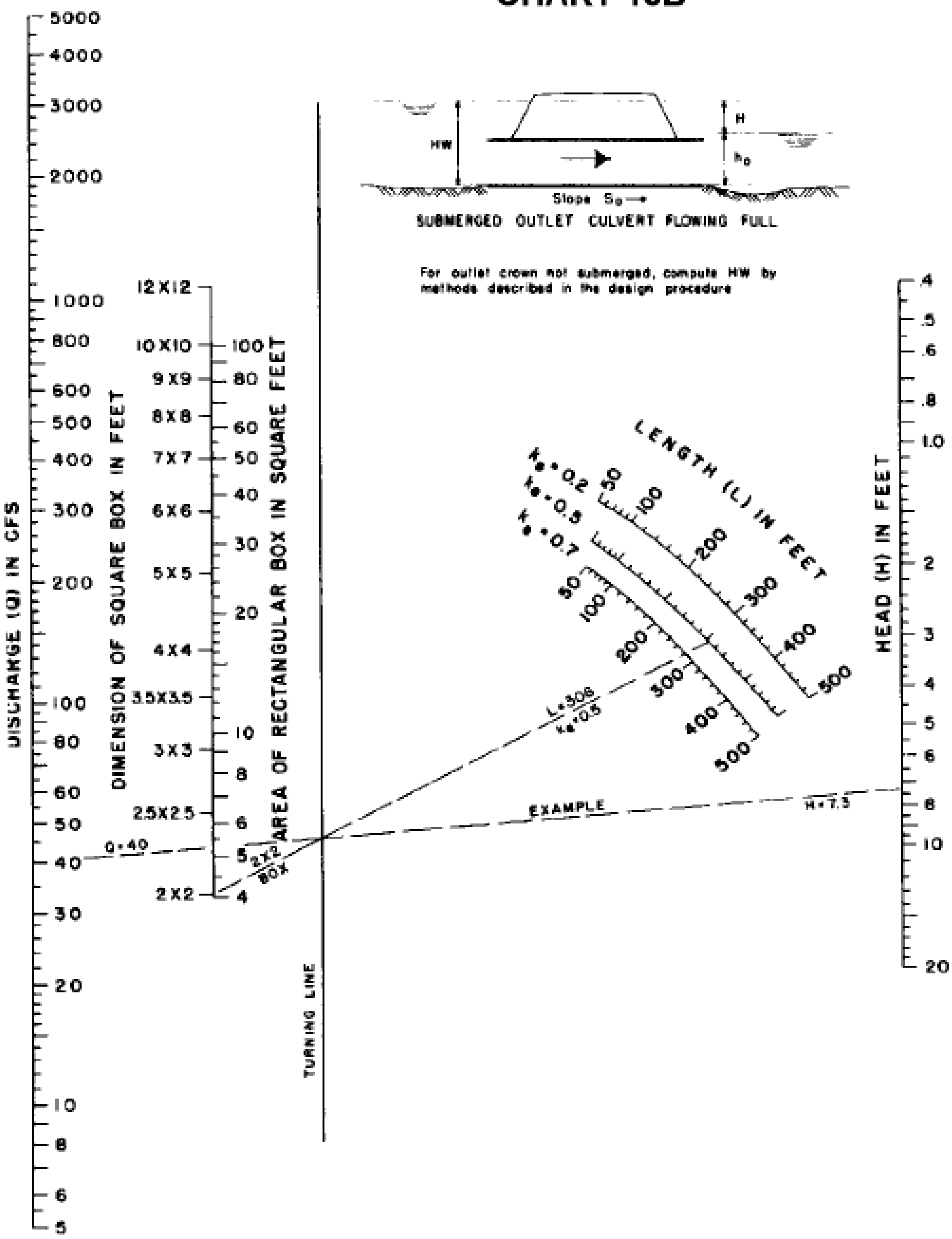
HEADWATER DEPTH FOR INLET CONTROL
RECTANGULAR BOX CULVERTS
FLARED WINGWALLS 18° TO 33.7° & 45°
WITH BEVELED EDGE AT TOP OF INLET

CHART 14B



CRITICAL DEPTH
RECTANGULAR SECTION

CHART 15B



HEAD FOR
CONCRETE BOX CULVERTS
FLOWING FULL
$n = 0.012$

AU OF PUBLIC ROADS JAN. 1963

APPENDIX D

STANDARD HYDRAULICS TABLES AND EXAMPLES

APPENDIX D.1

DRAINAGE RUNOFF CALCULATIONS FOR DRAINAGE AREA TABLE

DRAINAGE AREA COMPUTATIONS

Design Point ID	Runoff Coef. "C"	Area "A" (Acres)	Total "CA"	Time of Concentration (min)	Intensity I 2yr (in/hr)	Discharge Q 2yr (cfs)	Intensity I 10yr (in/hr)	Discharge Q 10yr (cfs)	Intensity I 100yr (in/hr)	Discharge Q 100yr (cfs)	Comments
1	2	3	4	5	6	7	8	9	10	11	12

APPENDIX D.2

HYDRAULIC GRADE LINE CALCULATIONS TABLE

HYDRAULIC COMPUTATIONS FOR STORM DRAIN LINE ____________

				Drainage Area					Rainfall Intensity					Design Flow			Design Conduit					Friction Loss		Hydraulic Grade Line			Velocity		Minor Loss					Ground/HGL Elev				
Design Point ID	Upstream Location (Design Point)	Downstream Location	Distance	Drainage Area	Total Drainage Area "A"	Runoff Coefficient "C"	Incremental "CA"	Total "CA"	Design Flood	Inlet Time	Travel Time In Conduit	Time Of Concentration	Rainfall Intensity "I"	Design Runoff "Q"	Inlet Bypass "Q_{co}"	Pipe Discharge "Q_p"	No. Of Conduits	Span (Box Culvert)	Pipe Diameter (Culvert Rise)	Slope of Conduit	Flow Depth in Conduit	Friction Slope "Sf"	Friction Loss	Upstream HGL Elevation	Downstream HGL Elevation	Design Point Hgl Elevation	Upstream Velocity(V_1)	Downstream Velocity (V_2)	Upstream Velocity Head $V_1^2/2g$	Downstream Velocity Head $V_2^2/2g$	Minor Loss Coefficient K	$kV_1^2/2g$	Total Minor Loss	Upstream Ground Elev (Top of Curb)	Elev. Diff Ground-HGL	Upstream Pipe Flowline	Downstream Pipe Flowline	Comments
	sta	sta	ft	acres	acres				yrs	min	min	min	in/hr	cfs	cfs	cfs		ft	in (ft)	ft/ft	ft	ft/ft	ft	ft	ft	ft	ft/sec	ft/sec	ft	ft		ft	ft	ft	ft	ft	ft	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39
STORM DRAIN LINE																																						
STORM DRAIN LATERAL																																						
STORM DRAIN LATERAL																																						

APPENDIX D.3

INLET DRAINAGE FLOW INTERCEPTION CALCULATION TABLE

INLET CALCULATIONS (10-YR)

INLET			STORM	DRAINAGE AREA CHARACTERISTICS			FLOW					SAG INLET		GRADE	STREET TYPE		STRAIGHT CROSS SLOPE STREETS			PARABOLIC CROWN STREETS					EQUIVALENT CROSS SLOPE				INLET LENGTH					
DESIGN POINT ID	STATION	TYPE	DESIGN FLOOD YRS	RUN COEF C	INTENSITY I IN/HR	AREA A ACRES	STREET Q_S CFS	PIPE Q_P CFS	CARRYOVER FLOW TO INLET Q_{CO} CFS	TOTAL FLOW Q_T TO INLET CFS	GUTTER OR ROW CAPACITY CFS	WEIR (W) ORIFICE (O) FLOW	SAG DEPTH FT	LONGITUDINAL S_L FT/FT	SECTION WIDTH (B-B) FT	SECTION PARABOLIC (P) STRAIGHT (S)	CROSS SLOPE S_X FT/FT	TOP WIDTH T FT	DEPTH d FT	CROWN HEIGHT FT	CONVEYANCE K CFS	TOP WIDTH T FT	CALC CROSS SLOPE Sx FT/FT	DEPTH d FT	E_O	W FT	a FT	S_E FT/FT	L_R FT	L_A FT	INLET BY PASS FLOW/CARRYOVER Q_{CO} CFS	FLOW INTERCEPT BY INLET Q_I CFS	INLET FLOW BY PASS TO DESIGN POINT ID	COMMENTS
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35

INLET CALCULATIONS (100-YR)

INLET			STORM	DRAINAGE AREA CHARACTERISTICS			FLOW					SAG INLET		GRADE	STREET TYPE		STRAIGHT CROSS SLOPE STREETS			PARABOLIC CROWN STREETS					EQUIVALENT CROSS SLOPE				INLET LENGTH					
DESIGN POINT ID	STATION	TYPE	DESIGN FLOOD YRS	RUN COEF C	INTENSITY I IN/HR	AREA A ACRES	STREET Q_S CFS	PIPE Q_P CFS	CARRYOVER FLOW TO INLET Q_{CO} CFS	TOTAL FLOW Q_T TO INLET CFS	GUTTER OR ROW CAPACITY CFS	WEIR (W) ORIFICE (O) FLOW	SAG DEPTH FT	LONGITUDINAL S_L FT/FT	SECTION WIDTH (B-B) FT	SECTION PARABOLIC (P) STRAIGHT (S)	CROSS SLOPE S_X FT/FT	TOP WIDTH T FT	DEPTH d FT	CROWN HEIGHT FT	CONVEYANCE K CFS	TOP WIDTH T FT	CALC CROSS SLOPE Sx FT/FT	DEPTH d FT	E_O	W FT	a FT	S_E FT/FT	L_R FT	L_A FT	INLET BY PASS FLOW/CARRYOVER Q_{CO} CFS	FLOW INTERCEPT BY INLET Q_I CFS	INLET FLOW BY PASS TO DESIGN POINT ID	COMMENTS
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35

APPENDIX D.4

EXAMPLE DESIGN PLANS

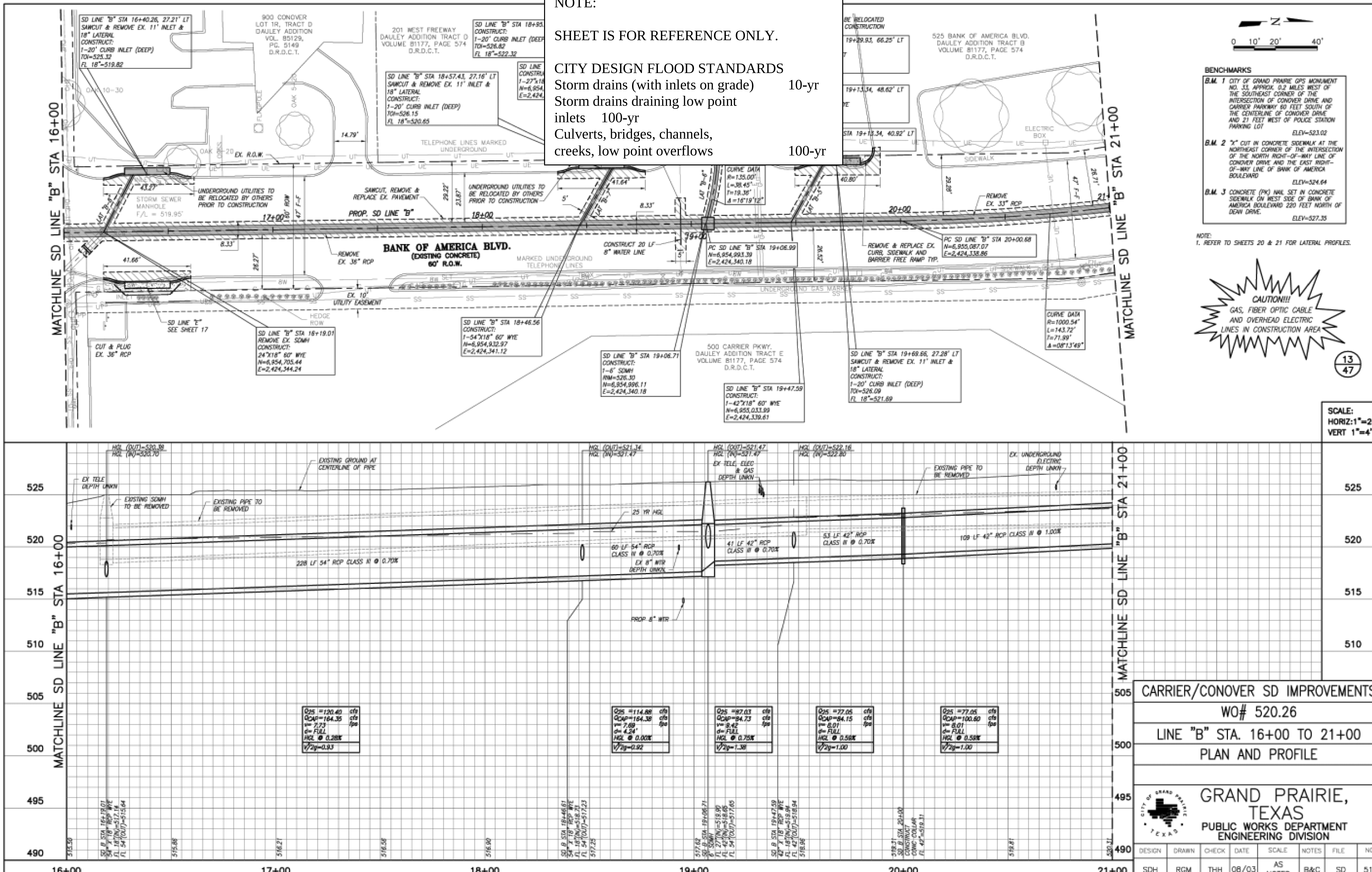

NOTE:
ALL EXISTING STORM DRAIN PIPES TO BE VERIFIED BASED ON THE CITY'S EXISTING STORM DRAIN MAPS.
N
0 50' 100' 200'
BENCHMARKS
B.M. 1 CITY OF GRAND PRAIRIE GPS MONUMENT NO. 33, APPROX. 0.2 MILES WEST OF THE SOUTHEAST CORNER OF THE INTERSECTION OF CONOVER DRIVE AND CARRIER PARKWAY 60 FEET SOUTH OF THE CENTERLINE OF CONOVER DRIVE AND 21 FEET WEST OF POLICE STATION PARKING LOT
ELEV=523.02
B.M. 2 "X" CUT IN CONCRETE SIDEWALK AT THE NORTHEAST CORNER OF THE INTERSECTION OF THE NORTH RIGHT-OF-WAY LINE OF CONOVER DRIVE AND THE EAST RIGHT-OF-WAY LINE OF BANK OF AMERICA BOULEVARD
ELEV=524.64
B.M. 3 CONCRETE (PK) NAIL SET IN CONCRETE SIDEWALK ON THE WEST SIDE OF BANK OF AMERICA BLVD. 220 FEET NORTH OF DEAN DRIVE.
ELEV=527.35
LEGEND
DRAINAGE AREA BOUNDARY
EXISTING CONTOURS
DIRECTION OF RUNOFF
DRAINAGE AREA CALLOUTS/AREAS
DESIGN POINT
EXISTING STORM DRAIN
PROPOSED STORM DRAIN
WEST JEFFERSON STREET
WEST FREEWAY (S)
WEST FREEWAY (N)
DEAN DRIVE
CONOVER DRIVE
BANK OF AMERICA BLVD.
SOUTH CARRIER PARKWAY
SHAWNEE TERRACE
SD LINE "A"
SD LINE "B"
SD LINE "C"
SD LINE "E"
CARRIER/CONOVER SD IMPROVEMENTS
WO# 520.26
DRAINAGE AREA MAP
GRAND PRAIRIE, TEXAS
PUBLIC WORKS DEPARTMENT
ENGINEERING DIVISION
DESIGN SDH
DRAWN RGM
CHECK THH
DATE 08/03
SCALE AS NOTED
NOTES B&C
FILE SD
NO. 513

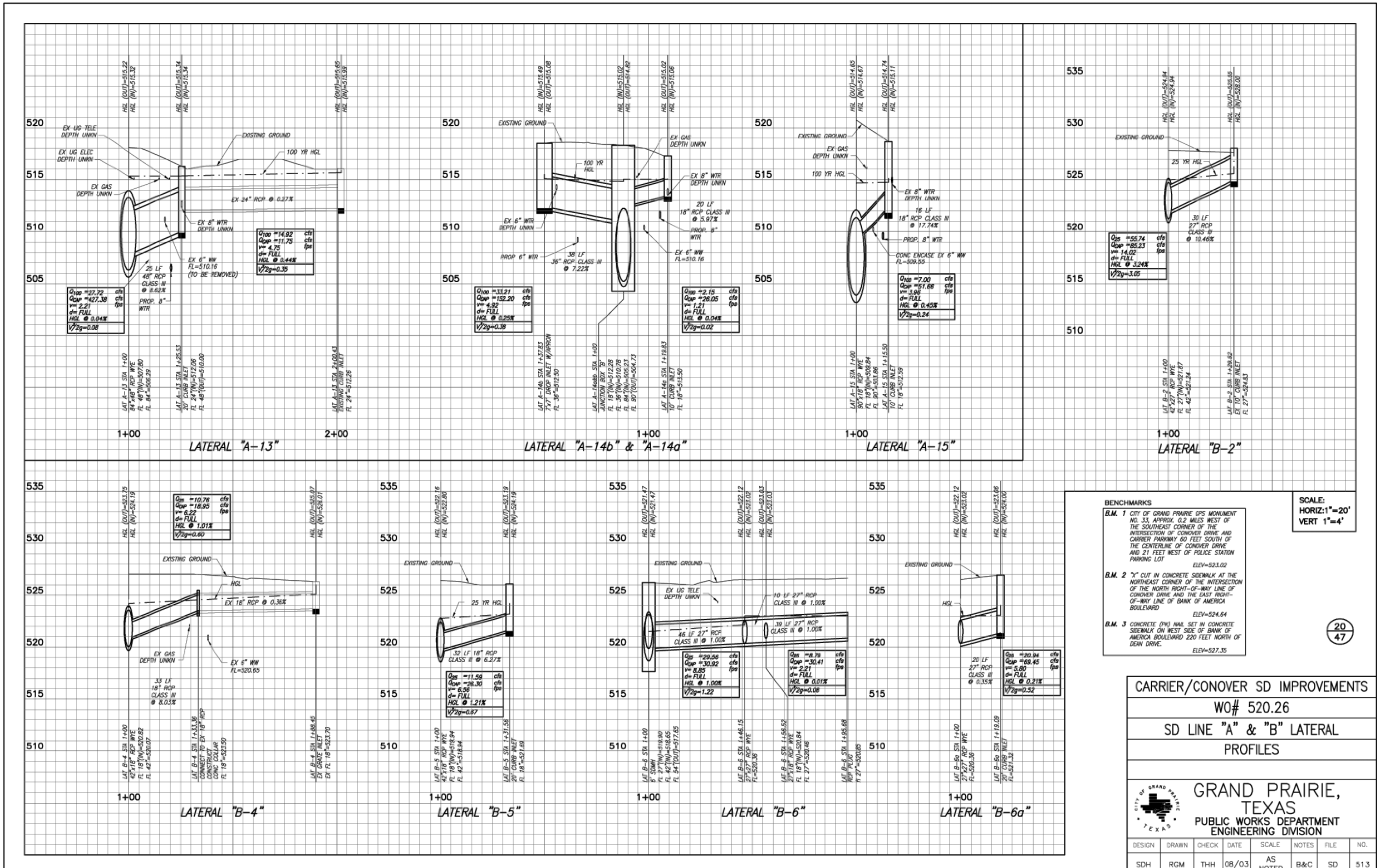
NOTE:

SHEET IS FOR REFERENCE ONLY.

CITY DESIGN FLOOD STANDARDS

Storm drains (with inlets on grade)	10-yr
Storm drains draining low point inlets	100-yr
Culverts, bridges, channels, creeks, low point overflows	100-yr




LATERAL "A-13"
LATERAL "A-14b" & "A-14a"
LATERAL "A-15"
LATERAL "B-2"
LATERAL "B-4"
LATERAL "B-5"
LATERAL "B-6"
LATERAL "B-6a"
BENCHMARKS
SCALE:
HORIZ:1"=20'
VERT 1"=4'
CARRIER/CONOVER SD IMPROVEMENTS
WO# 520.26
SD LINE "A" & "B" LATERAL
PROFILES
GRAND PRAIRIE, TEXAS
PUBLIC WORKS DEPARTMENT
ENGINEERING DIVISION
DESIGN SDH
DRAWN RGM
CHECK THH
DATE 08/03
SCALE AS NOTED
NOTES B&C
FILE SD
NO. 513

APPENDIX D.5

EXAMPLE DESIGN PLAN HYDRAULIC CALCULATION TABLES

DRAINAGE DESIGN MANUAL APPENDIX E.1 TABLE - STORM DRAIN DRAINAGE AREA COMPUTATIONS FOR SANDRA LANE - 6/27/2012 BID PLAN SUBMITTAL

Inlet ID #	Area Designation	Runoff Coef. "C"	Area "A" (Acres)	Total "CA"	Time of Concentration (min)	Intensity I 2yr (in/hr)	Discharge Q 2yr (cfs)	Intensity I 10yr (in/hr)	Discharge Q 10yr (cfs)	Intensity I 100yr (in/hr)	Discharge Q 100yr (cfs)	Comments
1	2	3	4	5	6	7	8	9	10	11	12	13
1S	S1	0.4	1.990	0.796	20	3.837	3.080	5.745	4.611	8.299	6.661	Collects drainage subarea S1; Type E opening/grate inlet line termination
2S	S2	0.4	2.180	0.872	20	3.837	3.374	5.745	5.052	8.299	7.297	Collects drainage subarea S2; Type E opening/grate inlet line termination
3S	S3	0.4	2.050	0.82	20	3.837	3.173	5.745	4.750	8.299	6.862	Collects drainage subarea S3; Type E opening/grate inlet line termination
4S	S4	0.4	2.190	0.876	20	3.837	3.390	5.745	5.075	8.299	7.331	Collects drainage subarea S4; Type E opening/grate inlet line termination
5S	S5	0.4	2.140	0.856	20	3.837	3.312	5.745	4.959	8.299	7.163	Collects drainage subarea S5; Type E opening/grate inlet line termination
6S	S6	0.4	2.280	0.912	20	3.837	3.529	5.745	5.283	8.299	7.632	Collects drainage subarea S6; Type E opening/grate inlet line termination
7S	S7	0.4	2.210	0.884	20	3.837	3.420	5.745	5.121	8.299	7.398	Collects drainage subarea S7; Type E opening/grate inlet line termination
8S	S8	0.4	2.080	0.832	20	3.837	3.219	5.745	4.820	8.299	6.963	Collects drainage subarea S8; Type E opening/grate inlet line termination
9S	S9	0.4	2.240	0.896	20	3.837	3.467	5.745	5.191	8.299	7.498	Collects drainage subarea S9; Type E opening/grate inlet line termination
10S	S10	0.4	2.360	0.944	20	3.837	3.653	5.745	5.469	8.299	7.900	Collects drainage subarea S10; Type E opening/grate inlet line termination
	S11	0.4	2.850	1.14	15	4.475	5.144	6.667	7.663	9.625	11.063	Includes drainage to Sandra Lane drainage sytem and Prairie Lane drainage system.
11S	S11S	0.4	1.830	0.732	15	4.475	3.303	6.667	4.921	9.625	7.104	Future collects drainage subarea S11S portion of subarea S11; assumed Type E opening/grate inlet line termination
	S12	0.4	2.280	0.912	15	4.475	4.115	6.667	6.131	9.625	8.851	Includes drainage to Sandra Lane drainage sytem and Prairie Lane drainage system.
12S	S12S	0.4	1.140	0.456	15	4.475	2.058	6.667	3.065	9.625	4.425	Future collects drainage subarea S12S portion of subarea S12; assumed Type E opening/grate inlet line termination

No.	Description	Date
5		
4		
3		
2		
1		

Revisions

STORM DRAIN, WATER, AND WASTEWATER REPLACEMENT SANDRA LN.

SANDRA LANE STORM DRAINAGE AREA MAP

WORK ORDER NO. 610.17

Grand Prairie TEXAS

ENGINEERING DIVISION

DESIGN	DRAWN	CHECK	DATE	SCALE	NOTES	FILE	NO.
KJ	JP	GF	2/10	1"=100'	DH	SD	573

DRAINAGE DESIGN MANUAL APPENDIX E.3 TABLE - STORM DRAIN INLET CALCULATIONS FOR SANDRA LANE - 6/27/2012 BID PLAN SUBMITTAL

INLET			STORM	DRAINAGE AREA			FLOW					SAG INLET		GRADE	STREET TYPE		STRAIGHT CROSS SLOPE STREETS			PARABOLIC CROWN STREETS					EQUIVALENT CROSS SLOPE				INLET LENGTH					COMMENTS
DESIGN POINT/INLET ID #	STATION	TYPE	DESIGN FLOOD YRS	RUN COEF C	INTENSITY I IN/HR	AREA A ACRES	STREET Q_S CFS	PIPE Q_P CFS	CARRYOVER FLOW TO INLET Q_{CO} CFS	TOTAL FLOW Q_T to Inlet CFS	GUTTER OR ROW CAPACITY CFS	WEIR (W) ORIFICE (O) FLOW	SAG DEPTH FT	Longitudinal S_L FT/FT	STREET WIDTH (B-B) FT	SECTION PARABOLIC (P) STRAIGHT (S)	CROSS SLOPE S_X FT/FT	TOP WIDTH T FT	DEPTH d FT	CROWN HEIGHT FT	CONVEYANCE K CFS	TOP WIDTH T FT	CALC CROSS SLOPE Sx FT/FT	DEPTH d FT	E_O	W FT	a FT	S_E FT/FT	L_R FT	L_A FT	INLET BY PASS FLOW/CARRYOVER Q_{CO} CFS	FLOW INTERCEPT BY INLET Q_I CFS	INLET FLOW BY PASS TO DESIGN POINT/INLET ID	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	19	15	17	18	20	21	16	22	23	24	25	26	27	28	29	30	31	32	33	34	35
1S	0+30	S	100	0.40	8.30	1.99	6.66	6.66	0	6.66	6.66	W	0.44	0.0060	20	P	-			0.33	86.00	10.00	0.04	0.30		3.00			7.52	7.52	0.00	6.66	4S	Collects drainage subarea S1; Type E opening/grate inlet line termination
2S	0+30	S	100	0.40	8.30	2.18	7.30	7.3	0	7.30	7.30	W	0.47	0.0060	20	P	-			0.33	94.21	10.00	0.05	0.30		3.00			7.52	7.52	0.00	7.30	3S	Collects drainage subarea S2; Type E opening/grate inlet line termination
3S	0+30	S	100	0.40	8.30	2.05	6.86	6.86	0	6.86	6.86	W	0.45	0.0060	20	P	-			0.33	88.59	10.00	0.05	0.30		3.00			7.52	7.52	0.00	6.86	5S	Collects drainage subarea S3; Type E opening/grate inlet line termination
4S	0+30	S	100	0.40	8.30	2.19	7.33	7.33	0	7.33	7.33	W	0.47	0.0060	20	P	-			0.33	94.64	10.00	0.05	0.30		3.00			7.52	7.52	0.00	7.33	6S	Collects drainage subarea S4; Type E opening/grate inlet line termination
5S	0+30	S	100	0.40	8.30	2.14	7.16	7.16	0	7.16	7.16	W	0.47	0.0060	20	P	-			0.33	92.48	10.00	0.05	0.30		3.00			7.52	7.52	0.00	7.16	8S	Collects drainage subarea S5; Type E opening/grate inlet line termination
6S	0+30	S	100	0.40	8.30	2.28	7.63	7.63	0	7.63	7.63	W	0.49	0.0060	20	P	-			0.33	98.53	10.00	0.05	0.31		3.00			7.52	7.52	0.00	7.63	7S	Collects drainage subarea S6; Type E opening/grate inlet line termination
7S	0+30	S	100	0.40	8.30	2.21	7.40	7.4	0	7.40	7.40	W	0.48	0.0060	20	P	-			0.33	95.58	10.00	0.05	0.31		3.00			7.52	7.52	0.00	7.40		Collects drainage subarea S7; Type E opening/grate inlet line termination
8S	0+30	S	100	0.40	8.30	2.08	6.96	6.96	0	6.96	6.96	W	0.46	0.0060	20	P	-			0.33	89.89	10.00	0.05	0.30		3.00			7.52	7.52	0.00	6.96		Collects drainage subarea S8; Type E opening/grate inlet line termination
9S	0+30	S	100	0.40	8.30	2.24	7.50	7.5	0	7.50	7.50	W	0.48	0.0060	20	P	-			0.33	96.80	10.00	0.05	0.31		3.00			7.52	7.52	0.00	7.50		Collects drainage subarea S9; Type E opening/grate inlet line termination
10S	0+30	S	100	0.40	8.30	2.36	7.90	7.9	0	7.90	7.90	W	0.50	0.0060	20	P	-			0.33	101.99	10.00	0.05	0.31		3.00			7.52	7.52	0.00	7.90		Collects drainage subarea S10; Type E opening/grate inlet line termination
11S	0+30	S	100	0.40	9.62	1.83	7.10	7.1	0	7.10	7.10	W	0.46	0.0060	20	P	-			0.33	91.71	10.00	0.05	0.30		3.00			7.52	7.52	0.00	7.10		Future collects drainage subarea S11S; assumed Type E opening/grate inlet line termination
12S	0+30	S	100	0.40	9.62	1.14	4.43	4.43	0	4.43	4.43	W	0.34	0.0060	20	P	-			0.33	57.13	10.00	0.03	0.32		3.00			7.52	7.52	0.00	4.43		Future collects drainage subarea S12S; assumed Type E opening/grate inlet line termination

DRAINAGE DESIGN MANUAL APPENDIX E.2 TABLE - STORM DRAIN HYDRAULIC COMPUTATIONS FOR SANDRA LANE - 6/27/2012 BID PLAN SUBMITTAL

				Drainage Area					Rainfall Intensity					Design Flow			Design Conduit					Friction Loss		Hydraulic Grade Line			Velocity		Minor Loss					Ground/HGL Elev				
Design Point ID	Upstream Location (Design Point) sta	Downstream Location sta	Distance ft	Incremental Drainage Area acres	Total Drainage Area "A" acres	Runoff Coefficient "C"	Incremental "CA"	Total "CA"	Design Flood yrs	Inlet Time min	Travel Time In Conduit min	Time Of Concentration min	Rainfall Intensity "I" in/hr	Total Discharge (Without bypass inflows) "Q" cfs	Pipe Discharge cfs	Total Bypass Flow (Street or ditch) cfs	No. Of Conduits	Span (Box Culvert) ft	Pipe Diameter (Culvert Rise) in (ft)	Slope of Conduit ft/ft	Flow Depth in Conduit ft	Friction Slope "Sf" ft/ft	Friction Loss ft	Upstream HGL Elevation ft	Downstream HGL Elevation ft	Design Point Hgl Elevation ft	Downstream Velocity (V) ft/sec	Upstream Velocity (V) ft/sec	Downstream Velocity Head $V^2/2g$ ft	Upstream Velocity Head $V^2/2g$ ft	Minor Loss Coefficient K	$kV^2/2g$ ft	Total Minor Loss ft	Upstream Ground Elev (Top of Curb) ft	Elev. Diff Ground-HGL ft	Upstream Pipe Flowline ft	Downstream Pipe Flowline ft	Comments
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39
LINE = Main Lateral Line C																																						
C1	20+18.90	20+15.80	3.10				0.00	0.00	100		0.00	0.00	21.133	0.00		0.0	1	-	24	0.0010	2.00	0.0000	0.00	534.79	534.79	534.79	0.00	0.00	0.00	0.00	1.00	0.00	0.00	537.00	2.21	528.75	528.75	End conduit and plug
C2	20+15.80	20+05.02	10.78	1.99	1.99	0.40	0.80	0.80	100	0.00	0.09	20.33	8.227	6.60	6.60	0.0	1	-	24	0.0010	2.00	0.0009	0.01	534.72	534.71	534.79	2.10	0.00	0.07	0.00	0.75	0.00	0.07	537.00	2.21	528.75	528.74	Junction incoming line C1; 45 degree lateral connection; collects subarea S1 flows
C3	20+05.02	20+02.02	3.00		1.99		0.00	0.80	100		0.05	20.38	8.215	6.59	6.59	0.0	1	-	36	0.0010	3.00	0.0001	0.00	534.71	534.71	534.71	0.93	2.10	0.01	0.07	0.72	0.05	0.00	537.00	2.29	528.74	528.73	Pipe size increase from 24" RCP to 36" RCP
C4	20+02.02	24+55.09	346.93	2.18	4.17	0.40	0.87	1.67	100	0.00	3.21	23.59	7.578	12.75	12.75	0.0	1	-	36	0.0010	3.00	0.0004	0.13	534.67	534.55	534.71	1.80	0.93	0.05	0.01	0.75	0.01	0.04	537.00	2.29	528.73	528.39	Junction incoming line C2; 45 degree lateral connection; collects subarea S2 flows
C5	24+55.09	24+49.09	6.00	2.05	6.22	0.40	0.82	2.49	100	0.00	0.04	23.62	7.571	18.99	18.99	0.0	1	-	36	0.0010	3.00	0.0008	0.00	534.47	534.47	534.55	2.69	1.80	0.11	0.05	0.75	0.04	0.07	536.20	1.65	528.39	528.38	Junction incoming line C3; 45 degree lateral connection; collects subarea S3 flows
C6	24+49.09	22+50.00	199.09	2.19	8.41	0.40	0.88	3.36	100	0.00	0.93	24.56	7.407	25.12	25.12	0.0	1	-	36	0.0010	3.00	0.0014	0.28	534.35	534.07	534.47	3.55	2.69	0.20	0.11	0.75	0.08	0.11	536.20	1.73	528.38	528.18	Junction incoming line C4; 45 degree lateral connection; collects subarea S4 flows
C7	22+50.00	21+68.05	81.95		8.41		0.00	3.36	100		0.53	25.09	7.317	24.82	24.82	0.0	1	-	42	0.0010	3.50	0.0006	0.05	534.07	534.02	534.07	2.58	3.55	0.10	0.20	0.75	0.15	0.00	536.00	1.93	528.18	528.10	Junction box with no incoming laterals; conduit size increase from 36" RCP to 42" RCP
C8	21+68.05	21+62.05	6.00	2.14	10.55	0.40	0.86	4.22	100	0.00	0.03	25.12	7.312	31.11	31.11	0.0	1	-	42	0.0010	3.50	0.0010	0.01	533.94	533.93	534.02	3.23	2.58	0.16	0.10	0.75	0.08	0.08	536.00	1.98	528.10	528.09	Junction incoming line C5; 45 degree lateral connection; collects subarea S5 flows
C9	21+62.05	19+00.00	262.05	2.28	12.83	0.40	0.91	5.13	100	0.00	1.14	26.26	7.128	36.88	36.88	0.0	1	-	42	0.0010	3.50	0.0013	0.35	533.83	533.47	533.93	3.83	3.23	0.23	0.16	0.75	0.12	0.11	536.00	2.07	528.09	527.83	Junction incoming line C6; 45 degree lateral connection; collects subarea S6 flows
C10	19+00.00	19+00.00	0.00		12.83		0.00	5.13	100		0.00	26.26	7.128	36.88	36.88	0.0	1	-	42	0.0000	3.50	0.0013	0.00	533.47	533.47	533.47	3.83	3.83	0.23	0.23	1.00	0.23	0.00	535.60	2.13	527.83	527.83	Internal Point (No Energy Loss)
C11	19+00.00	18+11.49	88.51		12.83		0.00	5.13	100		0.51	26.77	7.049	36.48	36.48	0.0	1	-	48	0.0010	4.00	0.0006	0.06	533.47	533.42	533.47	2.90	3.83	0.13	0.23	0.75	0.17	0.00	535.60	2.13	527.83	527.74	Junction box with no incoming laterals; conduit size increase from 42" RCP to 48" RCP
C12	18+11.49	17+77.50	33.99	2.21	15.04	0.40	0.88	6.02	100	0.00	0.17	26.93	7.023	42.60	42.60	0.0	1	-	48	0.0010	4.00	0.0009	0.03	533.34	533.31	533.42	3.39	2.90	0.18	0.13	0.75	0.10	0.08	534.60	1.18	527.74	527.71	Junction incoming line C7; 45 degree lateral connection; collects subarea S7 flows
C13	17+77.50	15+50.00	227.50	2.08	17.12	0.40	0.83	6.85	100	0.00	1.00	27.94	6.874	47.47	47.47	0.0	1	-	48	0.0010	4.00	0.0011	0.25	533.22	532.97	533.31	3.78	3.39	0.22	0.18	0.75	0.13	0.09	534.60	1.29	527.71	527.48	Junction incoming line C8; 45 degree lateral connection; collects subarea S8 flows
C14	15+50.00	14+61.15	88.85		17.12		0.00	6.85	100		0.40	28.33	6.818	47.08	47.08	0.0	1	-	48	0.0010	4.00	0.0011	0.10	532.92	532.82	532.97	3.75	3.78	0.22	0.22	0.75	0.17	0.05	535.15	2.18	527.48	527.39	Junction box with no incoming laterals; no conduit size change
C15	14+61.15	14+55.15	6.00	2.24	19.36	0.40	0.90	7.74	100	0.00	0.02	28.36	6.815	53.21	53.21	0.0	1	-	48	0.0010	4.00	0.0014	0.01	532.71	532.70	532.82	4.23	3.75	0.28	0.22	0.75	0.16	0.11	535.00	2.30	527.39	527.39	Junction incoming line C9; 45 degree lateral connection; collects subarea S9 flows
C16	14+55.15	12+00.00	255.15	2.36	21.72	0.40	0.94	8.69	100	0.00	0.93	29.27	6.688	58.59	58.59	0.0	1	-	48	0.0010	4.00	0.0017	0.42	532.57	532.15	532.70	4.66	4.23	0.34	0.28	0.75	0.21	0.13	535.00	2.30	527.39	527.13	Junction incoming line C10; 45 degree lateral connection; collects subarea S10 flows
C17	12+00.00	11+15.35	84.65		21.72		0.00	8.69	100		0.49	29.75	6.623	58.02	58.02	0.0	1	5	4	0.0010	4.00	0.0006	0.05	532.15	532.10	532.15	2.90	4.66	0.13	0.34	0.75	0.25	0.00	534.80	2.65	527.13	527.06	Junction box with no incoming laterals; conduit size increase from 48" RCP to 5' x 4' RCB
C18	11+15.35	11+09.53	5.82	1.14	22.86	0.40	0.46	9.14	100	0.00	0.03	29.79	6.619	60.51	60.51	0.0	1	5	4	0.0010	4.00	0.0006	0.00	532.08	532.08	532.10	3.05	2.90	0.14	0.13	1.00	0.13	0.01	534.80	2.70	527.05	527.04	Junction incoming line C12; 45 degree lateral connection; collects subarea S12 flows
C19	11+09.53	10+00.00	109.53	1.83	24.69	0.40	0.73	9.88	100	0.00	0.56	30.35	6.545	65.18	65.18	0.0	1	5	4	0.0010	4.00	0.0007	0.08	532.02	531.95	532.08	3.26	3.05	0.16	0.14	0.75	0.11	0.06	534.80	2.72	527.04	526.93	Junction incoming line C11; 45 degree lateral connection; collects subarea S11 flows
C20	10+00.00	10+00.00	0.00		24.69		0.00	9.88	100		0.00	30.35	6.545	65.18	65.18	0.0	1	5	4	0.0000	4.00	0.0004	0.00	531.95	531.95	531.95	2.72	3.26	0.12	0.16	0.95	0.16	0.00	534.70	2.75	526.93	526.93	Conduit size increase from 5' x 4' RCB to 6' x 4' RCB
C21	10+00.00	10+00.00	0.00		24.69		0.00	9.88	100		0.00	30.35	6.545	65.57	65.57	0.0	1	6	4	0.0000	4.00	0.0004	0.00	531.95	531.95	531.95	2.23	2.72	0.12	0.12	1.00	0.12	0.00	534.20	2.25	526.93	526.93	Junction Main Line A
LINE = Lateral Line C1 to Main Lateral Line C																																						
C1.1	0+30.44	0+00.30	30.44	1.99	1.99	0.40	0.80	0.80	100	20.00	0.24	20.24	8.246	6.62	6.62	0.0	1	-	24	0.0050	2.00	0.0009	0.03	534.78	534.76	534.89	2.11	0.00	0.07	0.00	1.50	0.00	0.10	534.50	-0.39	528.90	528.75	Collects drainage subarea S1; Type E opening/grate inlet line termination
C1.2	0+00.30	0+00.00	0.30		1.99		0.00	0.80	100		0.00	20.24	8.246	6.60	6.60	0.0	1	-	24	0.0000	2.00	0.0009	0.00	534.72	534.72	534.76	2.10	2.11	0.07	0.07	0.50	0.03	0.03	537.00	2.24	528.75	528.75	Junction Main Lateral Line C
LINE = Lateral Line C2 to Main Lateral Line C																																						
C2.1	0+30.41	0+00.30	30.4	2.18	2.18	0.40	0.87	0.87	100	20.00	0.22	20.22	8.251	7.25	7.25	0.0	1	-	24	0.0050	2.00	0.0010	0.03	534.71	534.68	534.84	2.31	0.00	0.08	0.00	1.50	0.00	0.12	534.75	-0.09	529.20	529.22	Collects drainage subarea S2; Type E opening/grate inlet line termination
C2.2	0+00.30	0+00.00	0.0		2.18		0.00	0.87	100		0.00	20.22	8.251	7.25	7.25	0.0	1	-	36	0.0000	3.00	0.0001	0.00	534.67	534.67	534.68	1.03	2.31	0.02	0.08	0.50	0.04	0.01	537.00	2.33	528.73	528.73	Junction Main Lateral Line C
LINE = Lateral Line C3 to Main Lateral Line C																																						
C3.1	0+30.44	0+00.30	30.4	2.05	2.05	0.40	0.82	0.82	100	20.00	0.23	20.23	8.248	6.82	6.82	0.0	1	-	24	0.0050	2.00	0.0009	0.03	534.57	534.55	534.63	2.17	0.00	0.07	0.00	1.50	0.00	0.11	533.20	-1.38	529.04	528.09	Collects drainage subarea S3; Type E opening/grate inlet line termination
C3.2	0+00.30	0+00.00	0.0		2.05		0.00	0.82	100		0.00	20.23	8.248	6.82	6.82	0.0	1	-	36	0.0000	3.00	0.0000	0.00	534.47	534.47	534.55	2.60	2.17	0.11	0.07	0.50	0.04	0.04	536.20	1.60	528.39	528.39	Junction Main Lateral Line C

SUBMITTED FOR REVIEW
BY CHRISTIAN Y. AGNEW, P.E. 50352
DATE Sep. 19, 12
NOT FOR CONSTRUCTION, BIDDING, OR PERMITTING PURPOSES

16
43

No.	Description	Date
5		
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3		
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Revisions

STORM DRAIN, WATER, AND WASTEWATER REPLACEMENT SANDRA LN.

SANDRA LANE STORM HYDRAULIC CALCULATIONS

WORK ORDER NO. 610.17

Grand Prairie TEXAS
ENGINEERING DIVISION

DESIGN	DRAWN	CHECK	DATE	SCALE	NOTES	FILE	NO.
KJ	JP	GF	2/10		DH	SD	573

DRAINAGE DESIGN MANUAL APPENDIX E.2 TABLE - STORM DRAIN HYDRAULIC COMPUTATIONS FOR SANDRA LANE - 6/27/2012 BID PLAN SUBMITTAL

Design Point ID	Upstream Location (Design Point) sta	Downstream Location sta	Distance ft	Drainage Area: Incremental Drainage Area acres	Drainage Area: Total Drainage Area "A" acres	Drainage Area: Runoff Coefficient "C"	Drainage Area: Incremental "CA"	Drainage Area: Total "CA"	Rainfall Intensity: Design Flood yrs	Rainfall Intensity: Inlet Time min	Rainfall Intensity: Travel Time in Conduit min	Rainfall Intensity: Time of Concentration min	Rainfall Intensity: Rainfall Intensity "I" in/hr	Design Flow: Total Discharge (Without bypass inflows) "Q" cfs	Design Flow: Pipe Discharge cfs	Design Flow: Total Bypass Flow (Street or Ditch) cfs	Design Conduit: No. Of Conduits	Design Conduit: Span (Box Culvert) ft	Design Conduit: Pipe Diameter (Culvert Rise) in (ft)
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
LINE = Lateral Line C4 to Main Lateral Line C																			
C4,1	0+30.44	0+00.00	30.4	2.19	2.19	0.40	0.88	0.88	100	20.00	0.22	20.22	8.251	7.29	7.29	0.0	1	-	24
C4,2	0+00.00	0+00.00	0.0		2.19		0.00	0.88	100		0.00	20.22	8.251	25.12	25.12	0.0	1	-	36
LINE = Lateral Line C5 to Main Lateral Line C																			
C5,1	0+30.44	0+00.00	30.4	2.14	2.14	0.40	0.86	0.86	100	20.00	0.22	20.22	8.250	7.12	7.12	0.0	1	-	24
C5,2	0+00.00	0+00.00	0.0		2.14		0.00	0.86	100		0.00	20.22	8.250	31.11	31.11	0.0	1	-	42
LINE = Lateral Line C6 to Main Lateral Line C																			
C6,1	0+30.44	0+00.00	30.4	2.28	2.28	0.40	0.91	0.91	100	20.00	0.21	20.21	8.253	7.59	7.59	0.0	1	-	24
C6,2	0+00.00	0+00.00	0.0		2.28		0.00	0.91	100		0.00	20.21	8.253	36.88	36.88	0.0	1	-	42
LINE = Lateral Line C7 to Main Lateral Line C																			
C7,1	0+30.44	0+00.00	30.4	2.21	2.21	0.40	0.88	0.88	100	20.00	0.22	20.22	8.251	7.35	7.35	0.0	1	-	24
C7,2	0+00.00	0+00.00	0.0		2.21		0.00	0.88	100		0.00	20.22	8.251	42.60	42.60	0.0	1	-	48
LINE = Lateral Line C8 to Main Lateral Line C																			
C8,1	0+30.44	0+00.00	30.4	2.08	2.08	0.40	0.83	0.83	100	20.00	0.23	20.23	8.248	6.92	6.92	0.0	1	-	24
C8,2	0+00.00	0+00.00	0.0		2.08		0.00	0.83	100		0.00	20.23	8.248	47.47	47.47	0.0	1	-	48
LINE = Lateral Line C9 to Main Lateral Line C																			
C9,1	0+30.44	0+00.00	30.4	2.24	2.24	0.40	0.90	0.90	100	20.00	0.21	20.21	8.252	7.46	7.46	0.0	1	-	24
C9,2	0+00.00	0+00.00	0.0		2.24		0.00	0.90	100		0.00	20.21	8.252	53.21	53.21	0.0	1	-	48
LINE = Lateral Line C10 to Main Lateral Line C																			
C10,1	0+30.44	0+00.00	30.4	2.36	2.36	0.40	0.94	0.94	100	20.00	0.20	20.20	8.254	7.86	7.86	0.0	1	-	24
C10,2	0+00.00	0+00.00	0.0		2.36		0.00	0.94	100		0.00	20.20	8.254	58.59	58.59	0.0	1	-	48
LINE = Lateral Line C11 to Main Lateral Line C																			
C11,1	0+30.44	0+00.00	30.4	1.83	1.83	0.40	0.73	0.73	100	15.00	0.23	15.23	9.554	7.05	7.05	0.0	1	-	24
C11,2	0+00.00	0+00.00	0.0		1.83		0.00	0.73	100		0.00	15.23	9.554	65.18	65.18	0.0	1	5	4
LINE = Lateral Line C12 to Main Lateral Line C																			
C12,1	0+30.44	0+00.00	30.4	1.14	1.14	0.40	0.46	0.46	100	15.00	0.36	15.36	9.511	4.37	4.37	0.0	1	-	24
C12,2	0+00.00	0+00.00	0.0		1.14		0.00	0.46	100		0.00	15.36	9.511	61.03	61.03	0.0	1	5	4

Design Point ID	Design Conduit: Slope of Conduit ft/ft	Design Conduit: Flow Depth in Conduit ft	Friction Loss: Friction Slope "Sf" ft/ft	Friction Loss: Friction Loss ft	Hydraulic Grade Line: Upstream HGL Elevation ft	Hydraulic Grade Line: Downstream HGL Elevation ft	Hydraulic Grade Line: Design Point Hgl Elevation ft	Velocity: Downstream Velocity (Vo) ft/sec	Velocity: Upstream Velocity (Vi) ft/sec	Minor Loss: Downstream Velocity Head $V_o^2/2g$ ft	Minor Loss: Upstream Velocity Head $V_i^2/2g$ ft	Minor Loss: Minor Loss Coefficient K	Minor Loss: $kV^2/2g$ ft	Minor Loss: Total Minor Loss ft	Ground/HGL Elev: Upstream Ground Elev (Top of Curb) ft	Ground/HGL Elev: Elev. Diff Ground-HGL ft	Ground/HGL Elev: Upstream Pipe Flowline ft	Ground/HGL Elev: Downstream Pipe Flowline ft	Comments
1	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39
LINE = Lateral Line C4 to Main Lateral Line C																			
C4,1	0.0050	2.00	0.0010	0.03	534.54	534.51	534.67	2.32	0.00	0.08	0.00	1.50	0.00	0.13	533.10	-1.57	529.03	528.88	Collects drainage subarea S4; Type E opening/grate inlet line termination
C4,2	0.0000	3.00	0.0014	0.00	534.35	534.35	534.51	3.55	2.32	0.20	0.08	0.50	0.01	0.15	536.20	1.69	528.88	528.38	Junction Main Lateral Line C
LINE = Lateral Line C5 to Main Lateral Line C																			
C5,1	0.0050	2.00	0.0010	0.03	534.09	534.06	534.21	2.27	0.00	0.08	0.00	1.50	0.00	0.12	533.05	-1.16	529.00	528.85	Collects drainage subarea S5; Type E opening/grate inlet line termination
C5,2	0.0000	3.50	0.0010	0.00	533.94	533.94	534.06	3.23	2.27	0.16	0.08	0.50	0.04	0.12	536.00	1.94	528.85	528.10	Junction Main Lateral Line C
LINE = Lateral Line C6 to Main Lateral Line C																			
C6,1	0.0050	2.00	0.0011	0.03	534.04	534.01	534.18	2.42	0.00	0.09	0.00	1.50	0.00	0.14	533.10	-1.08	528.99	528.84	Collects drainage subarea S6; Type E opening/grate inlet line termination
C6,2	0.0000	3.50	0.0013	0.00	533.83	533.83	534.01	3.83	2.42	0.23	0.09	0.50	0.05	0.18	536.00	1.99	528.84	528.09	Junction Main Lateral Line C
LINE = Lateral Line C7 to Main Lateral Line C																			
C7,1	0.0050	2.00	0.0011	0.03	533.50	533.47	533.63	2.34	0.00	0.09	0.00	1.50	0.00	0.13	532.60	-1.03	528.89	528.74	Collects drainage subarea S7; Type E opening/grate inlet line termination
C7,2	0.0000	4.00	0.0009	0.00	533.34	533.34	533.47	3.39	2.34	0.18	0.09	0.50	0.04	0.14	534.60	1.13	528.74	527.74	Junction Main Lateral Line C
LINE = Lateral Line C8 to Main Lateral Line C																			
C8,1	0.0050	2.00	0.0009	0.03	533.43	533.40	533.54	2.20	0.00	0.08	0.00	1.50	0.00	0.11	532.90	-0.64	528.86	528.71	Collects drainage subarea S8; Type E opening/grate inlet line termination
C8,2	0.0000	4.00	0.0011	0.00	533.22	533.22	533.40	3.78	2.20	0.22	0.08	0.50	0.04	0.10	534.60	1.20	528.71	527.71	Junction Main Lateral Line C
LINE = Lateral Line C9 to Main Lateral Line C																			
C9,1	0.0050	2.00	0.0011	0.03	532.98	532.94	533.11	2.37	0.00	0.09	0.00	1.50	0.00	0.13	532.00	-1.11	528.54	528.39	Collects drainage subarea S9; Type E opening/grate inlet line termination
C9,2	0.0000	4.00	0.0014	0.00	532.71	532.71	532.94	4.23	2.37	0.28	0.09	0.50	0.04	0.23	535.00	2.06	528.39	527.39	Junction Main Lateral Line C
LINE = Lateral Line C10 to Main Lateral Line C																			
C10,1	0.0050	2.00	0.0012	0.04	532.90	532.86	533.04	2.50	0.00	0.10	0.00	1.50	0.00	0.15	532.55	-0.49	528.54	528.39	Collects drainage subarea S10; Type E opening/grate inlet line termination
C10,2	0.0000	4.00	0.0017	0.00	532.57	532.57	532.86	4.66	2.50	0.34	0.10	0.50	0.05	0.29	535.00	2.14	528.39	527.39	Junction Main Lateral Line C
LINE = Lateral Line C11 to Main Lateral Line C																			
C11,1	0.0050	2.00	0.0010	0.03	532.18	532.15	532.30	2.24	0.00	0.08	0.00	1.50	0.00	0.12	532.65	0.35	528.19	528.04	Future collects drainage subarea S11S; assumed Type E opening/grate inlet line termination
C11,2	0.0000	4.00	0.0007	0.00	532.02	532.02	532.15	3.26	2.24	0.16	0.08	0.50	0.04	0.13	534.80	2.65	528.04	527.04	Junction Main Lateral Line C
LINE = Lateral Line C12 to Main Lateral Line C																			
C12,1	0.0050	2.00	0.0004	0.01	532.23	532.21	532.27	1.39	0.00	0.03	0.00	1.50	0.00	0.05	532.33	0.06	528.20	528.05	Future collects drainage subarea S12S; assumed Type E opening/grate inlet line termination
C12,2	0.0000	4.00	0.0006	0.00	532.08	532.08	532.21	3.05	1.39	0.14	0.03	0.50	0.02	0.13	534.80	2.59	528.05	527.05	Junction Main Lateral Line C

SUBMITTED FOR REVIEW
BY CHRISTIAN Y. AGNEW, P.E. 50352
DATE Sep. 19, 12
NOT FOR CONSTRUCTION, BIDDING, OR PERMITTING PURPOSES

17/43

No.	Description	Date
5		
4		
3		
2		
1		

Revisions

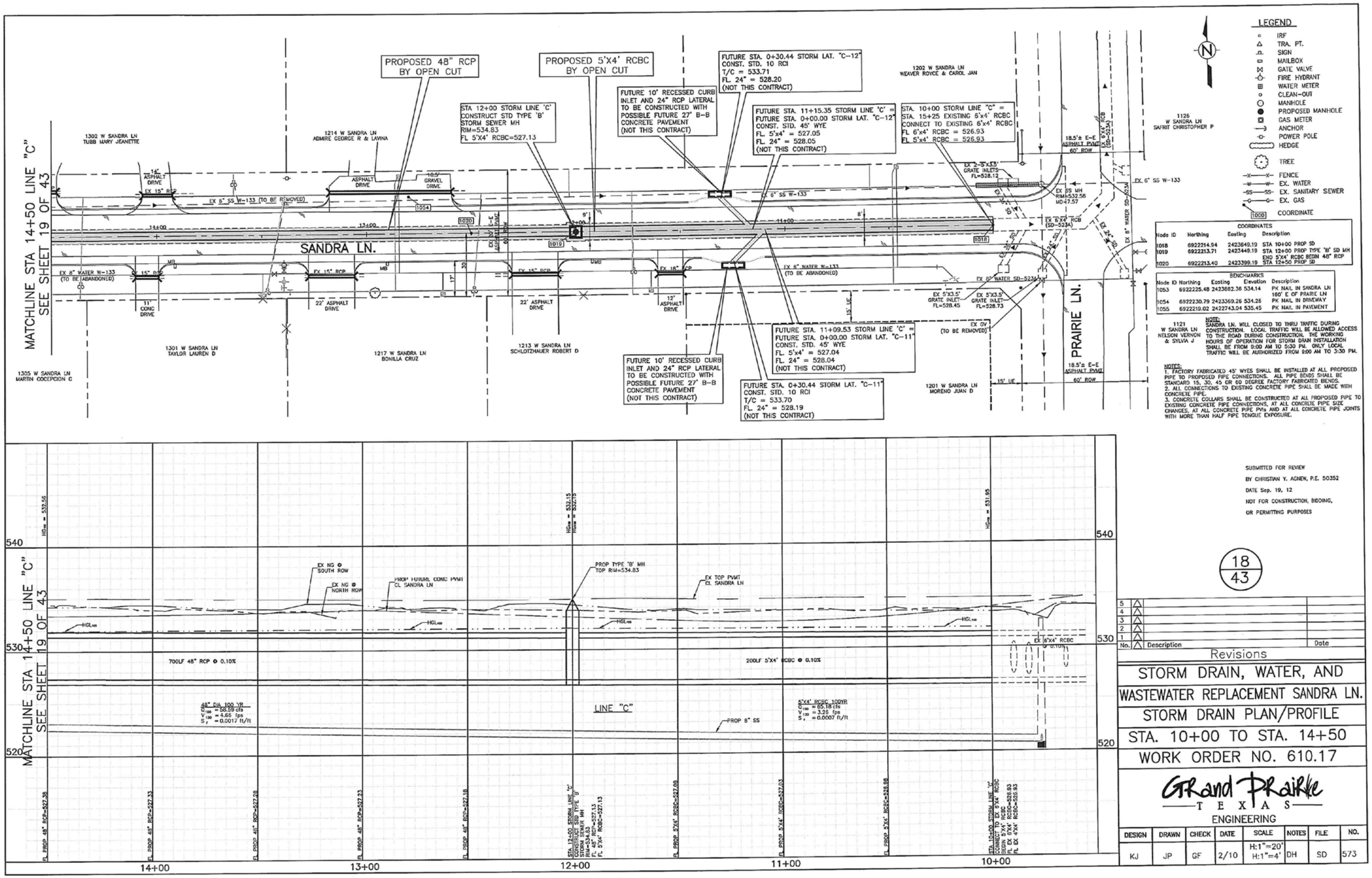
STORM DRAIN, WATER, AND WASTEWATER REPLACEMENT SANDRA LN.

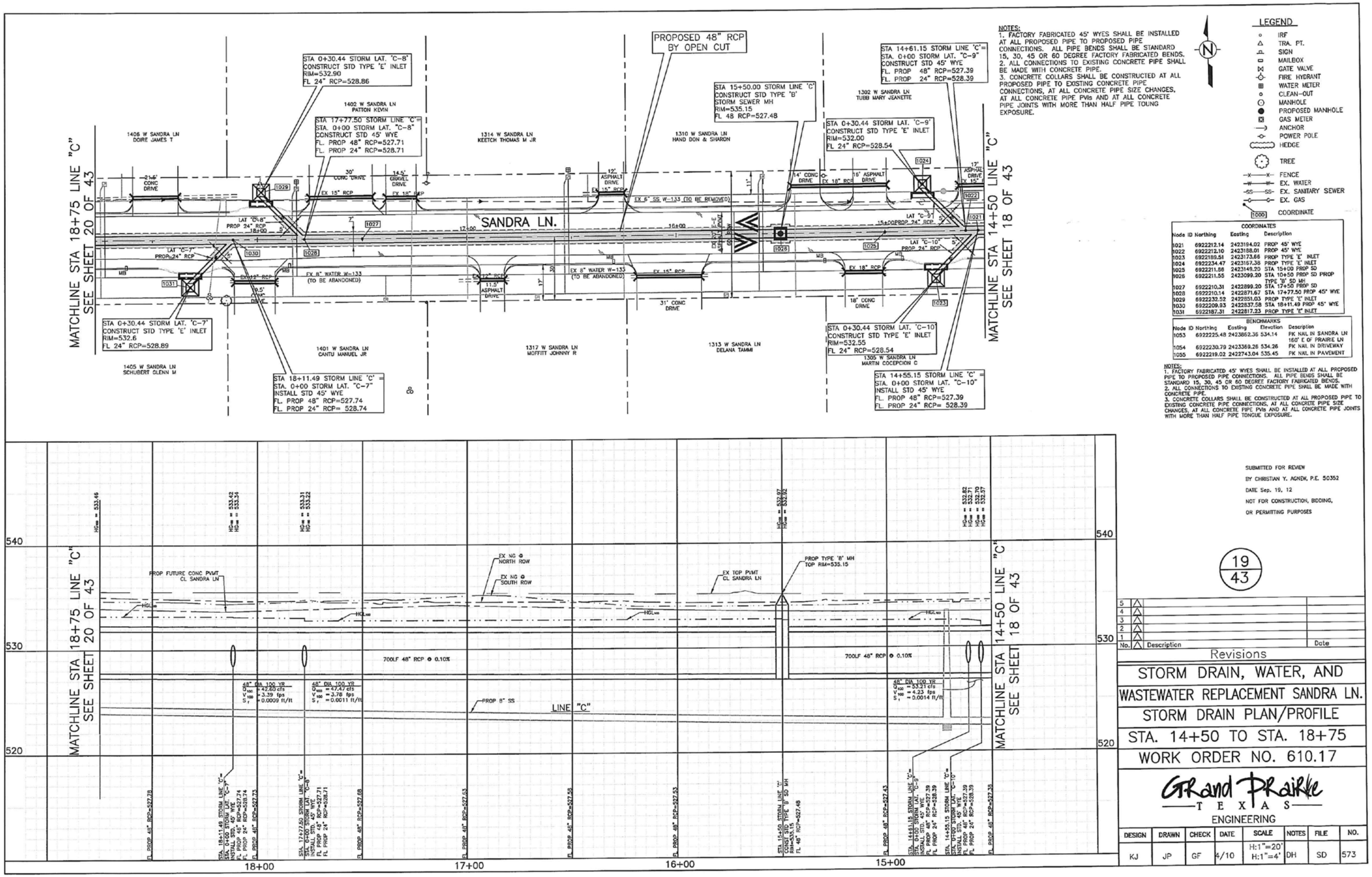
SANDRA LANE STORM HYDRAULIC CALCULATIONS

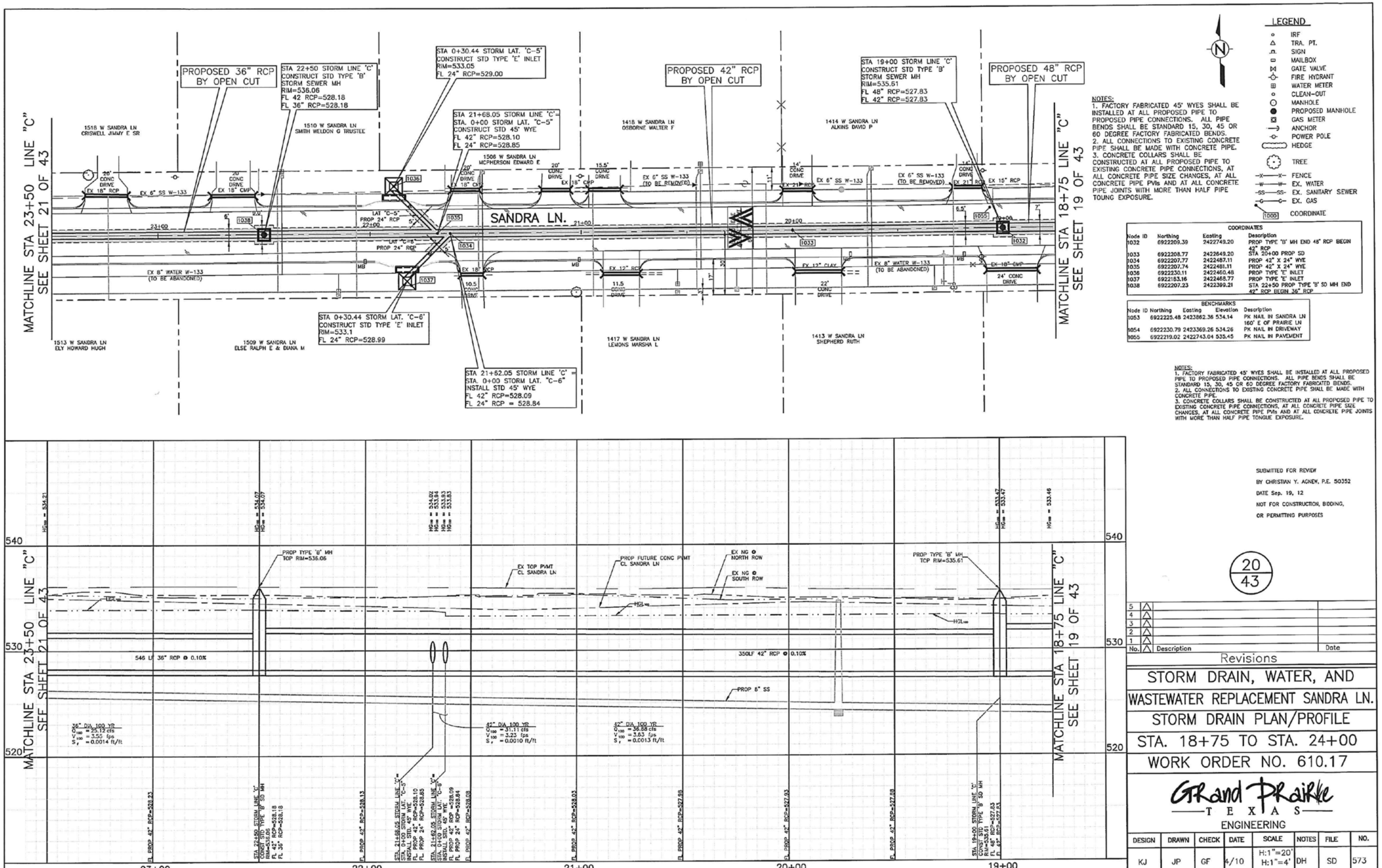
WORK ORDER NO. 610.17

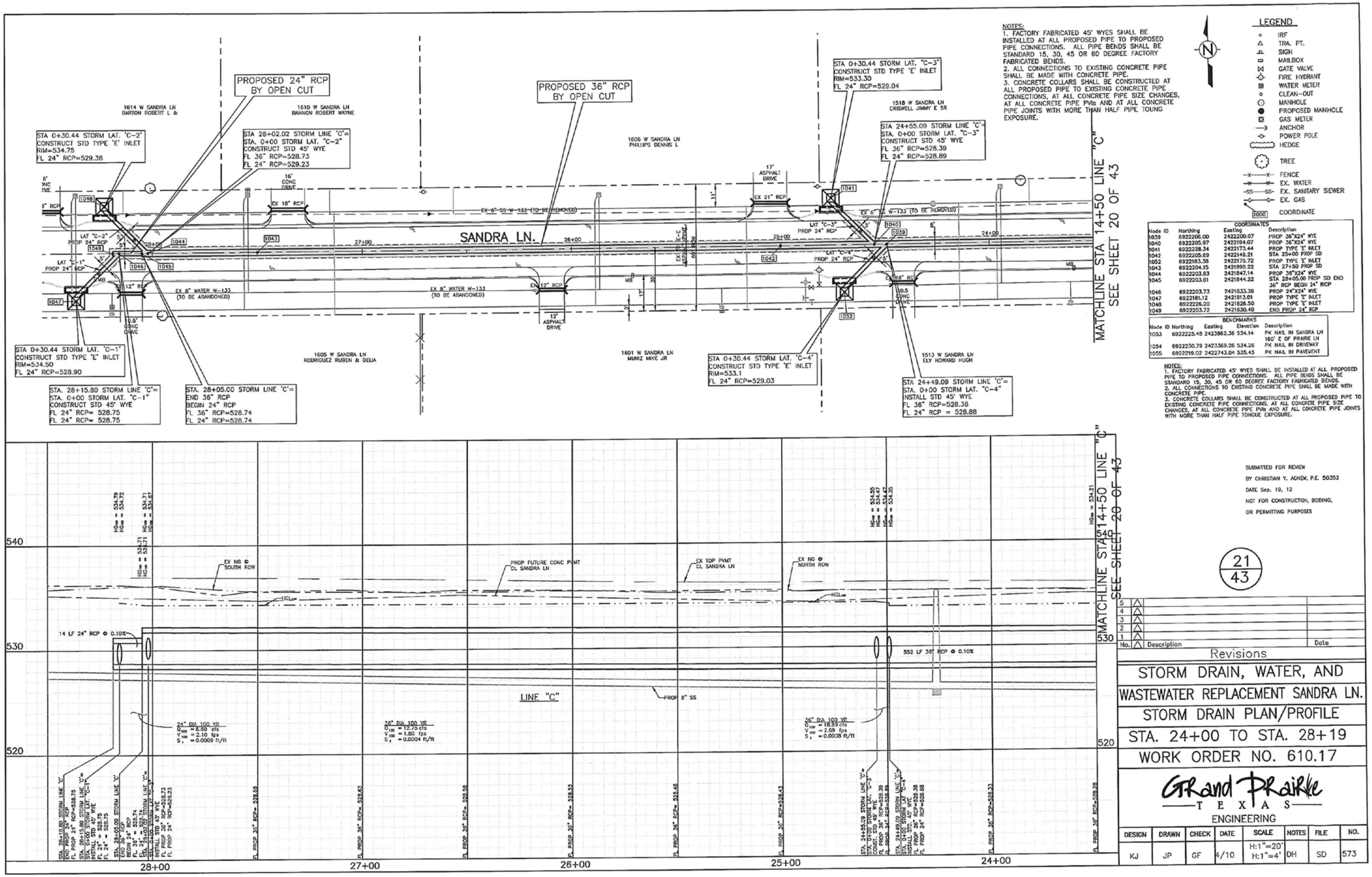
Grand Prairie TEXAS
ENGINEERING DIVISION

DESIGN	DRAWN	CHECK	DATE	SCALE	NOTES	FILE	NO.
KJ	JP	GF	2/10		DH	SD	573

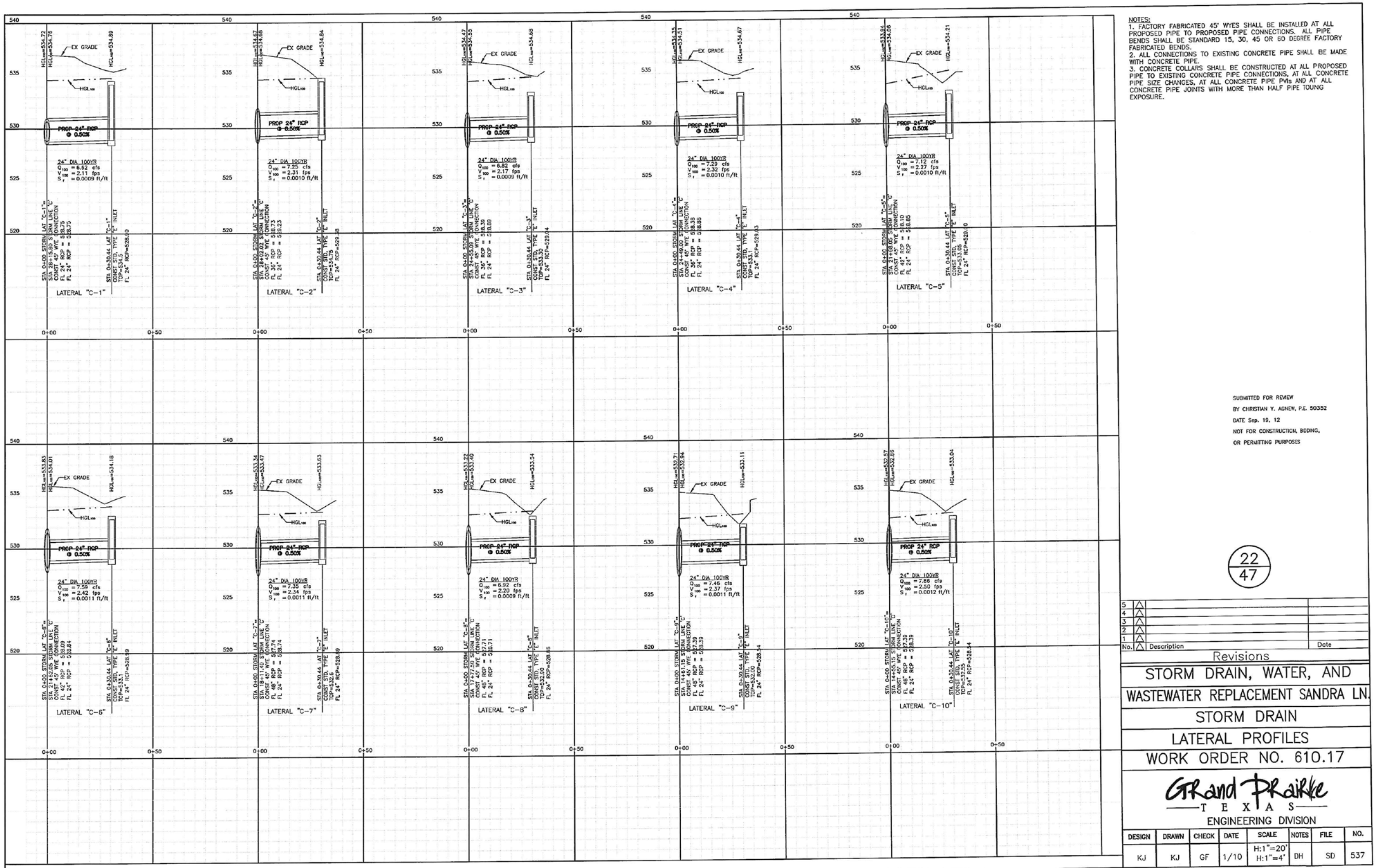

PROPOSED 48" RCP
BY OPEN CUT
PROPOSED 5'X4' RCBC
BY OPEN CUT
FUTURE STA. 0+30.44 STORM LAT. "C-12"
CONST. STD. 10 RCI
T/C = 533.71
FL 24" = 528.20
(NOT THIS CONTRACT)
FUTURE 10' RECESSED CURB INLET AND 24" RCP LATERAL TO BE CONSTRUCTED WITH POSSIBLE FUTURE 27' B-B CONCRETE PAVEMENT (NOT THIS CONTRACT)
STA 12+00 STORM LINE 'C'
CONSTRUCT STD TYPE 'B'
STORM SEWER MH
RIM=534.83
FL 5'X4' RCBC=527.13
FUTURE STA. 11+15.35 STORM LINE 'C' =
FUTURE STA. 0+00.00 STORM LAT. "C-12"
CONST. STD. 45° WYE
FL 5'x4' = 527.05
FL 24" = 528.05
(NOT THIS CONTRACT)
STA. 10+00 STORM LINE "C" =
STA. 15+25 EXISTING 6'x4' RCBC
CONNECT TO EXISTING 6'x4' RCBC
FL 6'x4' RCBC = 526.93
FL 5'x4' RCBC = 526.93
SANDRA LN.
PRAIRIE LN.
MATCHLINE STA 14+50 LINE "C"
SEE SHEET 19 OF 43
FUTURE STA. 11+09.53 STORM LINE 'C' =
FUTURE STA. 0+00.00 STORM LAT. "C-11"
CONST. STD. 45° WYE
FL 5'x4' = 527.04
FL 24" = 528.04
(NOT THIS CONTRACT)
FUTURE STA. 0+30.44 STORM LAT. "C-11"
CONST. STD. 10 RCI
T/C = 533.70
FL 24" = 528.19
(NOT THIS CONTRACT)
LEGEND
IRF
TRA. PT.
SIGN
MAILBOX
GATE VALVE
FIRE HYDRANT
WATER METER
CLEAN-OUT
MANHOLE
PROPOSED MANHOLE
GAS METER
ANCHOR
POWER POLE
HEDGE
TREE
FENCE
EX. WATER
EX. SANITARY SEWER
EX. GAS
COORDINATE
COORDINATES
BENCHMARKS
700LF 48" RCP @ 0.10%
200LF 5'X4' RCBC @ 0.10%
LINE "C"
18
43
Revisions
No.
Description
Date
STORM DRAIN, WATER, AND
WASTEWATER REPLACEMENT SANDRA LN.
STORM DRAIN PLAN/PROFILE
STA. 10+00 TO STA. 14+50
WORK ORDER NO. 610.17
Grand Prairie
TEXAS
ENGINEERING
DESIGN
DRAWN
CHECK
DATE
SCALE
NOTES
FILE
NO.
KJ
JP
GF
2/10
H:1"=20'
H:1"=4'
DH
SD
573
14+00
13+00
12+00
11+00
10+00
540
530
520

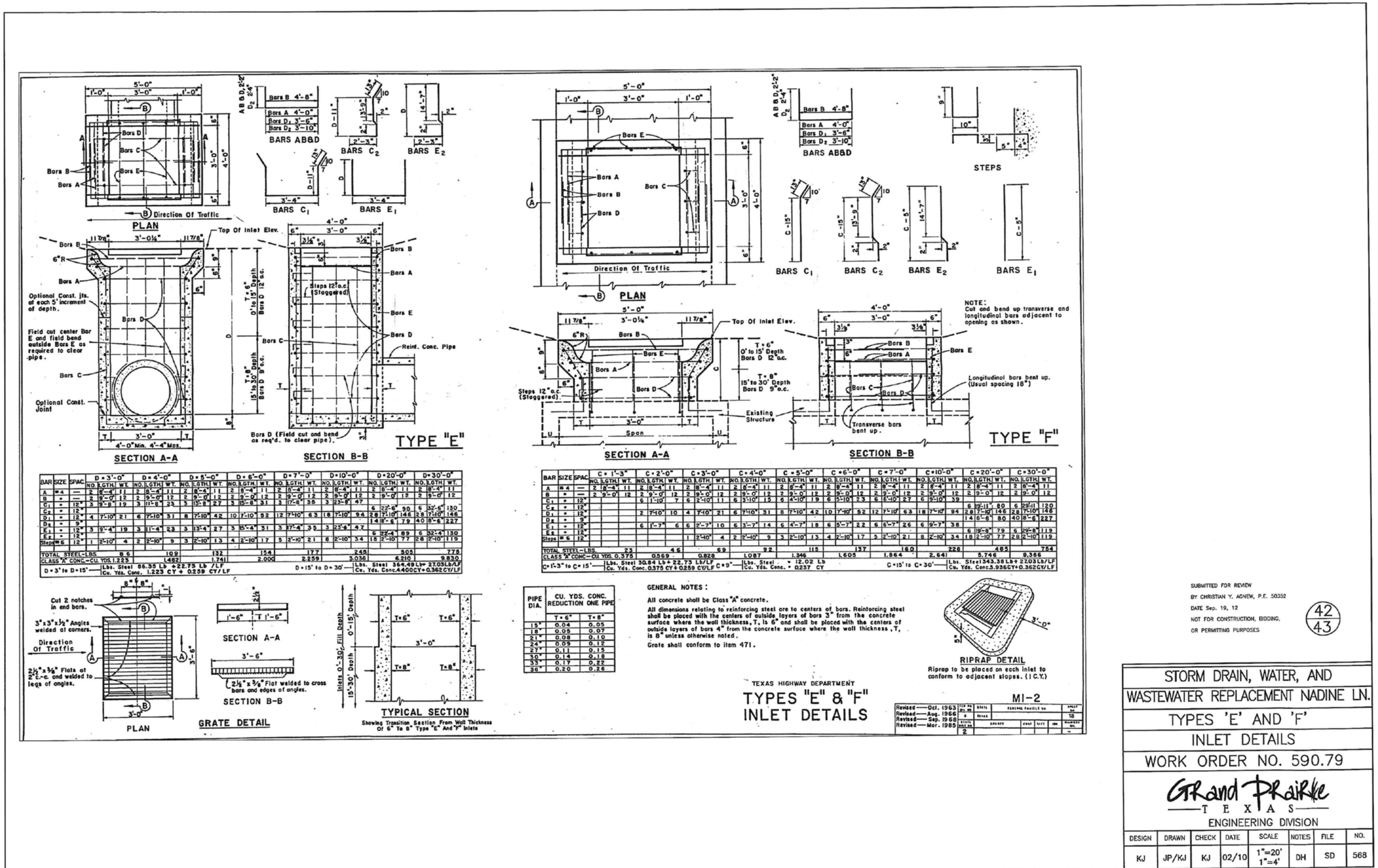

SANDRA LN.
PROPOSED 48" RCP BY OPEN CUT
MATCHLINE STA 18+75 LINE "C" SEE SHEET 20 OF 43
MATCHLINE STA 14+50 LINE "C" SEE SHEET 18 OF 43
LEGEND
COORDINATES
BENCHMARKS
NOTES:
SUBMITTED FOR REVIEW
19
43
Revisions
STORM DRAIN, WATER, AND
WASTEWATER REPLACEMENT SANDRA LN.
STORM DRAIN PLAN/PROFILE
STA. 14+50 TO STA. 18+75
WORK ORDER NO. 610.17
Grand Prairie
TEXAS
ENGINEERING
DESIGN DRAWN CHECK DATE SCALE NOTES FILE NO.
KJ JP GF 4/10 H:1"=20' H:1"=4' DH SD 573
LINE "C"
18+00
17+00
16+00
15+00
540
530
520

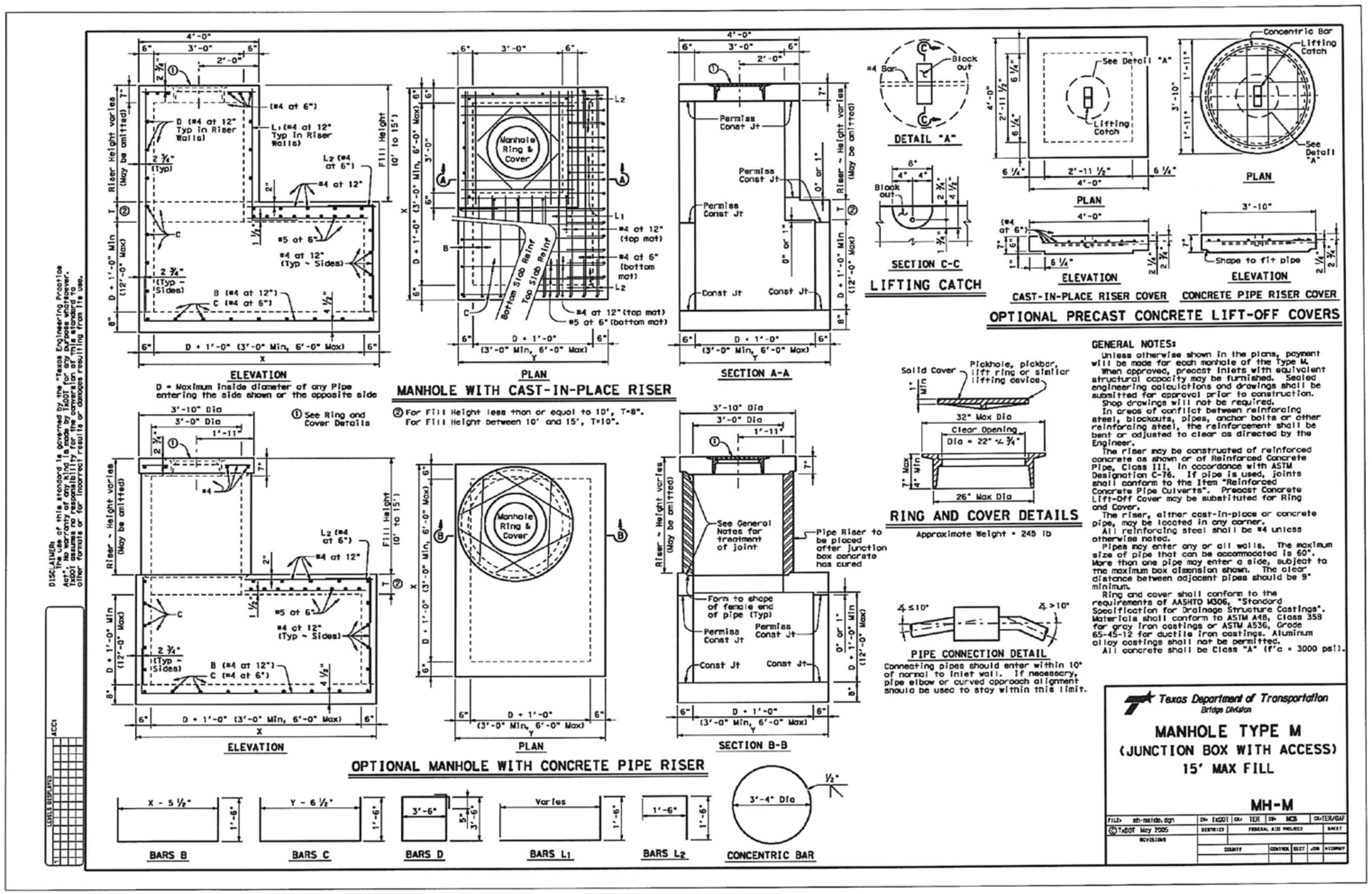

PROPOSED 36" RCP BY OPEN CUT
PROPOSED 42" RCP BY OPEN CUT
PROPOSED 48" RCP BY OPEN CUT
SANDRA LN.
MATCHLINE STA 23+50 LINE "C" SEE SHEET 21 OF 43
MATCHLINE STA 18+75 LINE "C" SEE SHEET 19 OF 43
LEGEND
NOTES:
COORDINATES
BENCHMARKS
20
43
Revisions
STORM DRAIN, WATER, AND
WASTEWATER REPLACEMENT SANDRA LN.
STORM DRAIN PLAN/PROFILE
STA. 18+75 TO STA. 24+00
WORK ORDER NO. 610.17
ENGINEERING
23+00
22+00
21+00
20+00
19+00



STORM DRAIN, WATER, AND WASTEWATER REPLACEMENT SANDRA LN.
STORM DRAIN PLAN/PROFILE
STA. 24+00 TO STA. 28+19
WORK ORDER NO. 610.17
Grand Prairie TEXAS ENGINEERING
SANDRA LN.
PROPOSED 24" RCP BY OPEN CUT
PROPOSED 36" RCP BY OPEN CUT
MATCHLINE STA 14+50 LINE "C" SEE SHEET 20 OF 43
LINE "C"
LEGEND
NOTES:
SUBMITTED FOR REVIEW BY CHRISTIAN Y. AGNEW, P.E. 50352 DATE Sep. 19, 12 NOT FOR CONSTRUCTION, BIDDING, OR PERMITTING PURPOSES
Revisions
28+00
27+00
26+00
25+00
24+00


NOTES:
1. FACTORY FABRICATED 45' WYES SHALL BE INSTALLED AT ALL PROPOSED PIPE TO PROPOSED PIPE CONNECTIONS. ALL PIPE BENDS SHALL BE STANDARD 15, 30, 45 OR 60 DEGREE FACTORY FABRICATED BENDS.
2. ALL CONNECTIONS TO EXISTING CONCRETE PIPE SHALL BE MADE WITH CONCRETE PIPE.
3. CONCRETE COLLARS SHALL BE CONSTRUCTED AT ALL PROPOSED PIPE TO EXISTING CONCRETE PIPE CONNECTIONS, AT ALL CONCRETE PIPE SIZE CHANGES, AT ALL CONCRETE PIPE PVIs AND AT ALL CONCRETE PIPE JOINTS WITH MORE THAN HALF PIPE TOUNG EXPOSURE.
LATERAL "C-1"
LATERAL "C-2"
LATERAL "C-3"
LATERAL "C-4"
LATERAL "C-5"
LATERAL "C-6"
LATERAL "C-7"
LATERAL "C-8"
LATERAL "C-9"
LATERAL "C-10"
SUBMITTED FOR REVIEW
BY CHRISTIAN Y. AGNEW, P.E. 50352
DATE Sep. 19, 12
NOT FOR CONSTRUCTION, BIDDING,
OR PERMITTING PURPOSES
22
47
No. Description Date
Revisions
STORM DRAIN, WATER, AND
WASTEWATER REPLACEMENT SANDRA LN.
STORM DRAIN
LATERAL PROFILES
WORK ORDER NO. 610.17
Grand Prairie
TEXAS
ENGINEERING DIVISION
DESIGN DRAWN CHECK DATE SCALE NOTES FILE NO.
KJ KJ GF 1/10 H:1"=20' H:1"=4' DH SD 537


TYPE "E"
TYPE "F"
PLAN
SECTION A-A
SECTION B-B
BARS AB&D
BARS C1
BARS C2
BARS E1
BARS E2
STEPS
GRATE DETAIL
TYPICAL SECTION
Showing Transition Section From Wall Thickness Of 6" To 8" Type "E" And "F" Inlets
RIPRAP DETAIL
Riprap to be placed on each inlet to conform to adjacent slopes. (1 C.Y.)
GENERAL NOTES:
All concrete shall be Class "A" concrete.
All dimensions relating to reinforcing steel are to centers of bars. Reinforcing steel shall be placed with the centers of outside layers of bars 3" from the concrete surface where the wall thickness, T, is 6" and shall be placed with the centers of outside layers of bars 4" from the concrete surface where the wall thickness, T, is 8" unless otherwise noted.
Grate shall conform to item 471.
NOTE: Cut and bend up transverse and longitudinal bars adjacent to opening as shown.
TEXAS HIGHWAY DEPARTMENT
TYPES "E" & "F" INLET DETAILS
MI-2
SUBMITTED FOR REVIEW
BY CHRISTIAN Y. AGNEW, P.E. 50352
DATE Sep. 19, 12
NOT FOR CONSTRUCTION, BIDDING, OR PERMITTING PURPOSES
42
43
STORM DRAIN, WATER, AND
WASTEWATER REPLACEMENT NADINE LN.
TYPES 'E' AND 'F'
INLET DETAILS
WORK ORDER NO. 590.79
Grand Prairie
TEXAS
ENGINEERING DIVISION
DESIGN KJ
DRAWN JP/KJ
CHECK KJ
DATE 02/10
SCALE 1"=20' 1"=4'
NOTES DH
FILE SD
NO. 568



STORM DRAIN, WATER, AND							
WASTEWATER REPLACEMENT NADINE LN.							
MANHOLE TYPE M							
WORK ORDER NO. 590.79							
Grand Prairie TEXAS ENGINEERING DIVISION							
DESIGN	DRAWN	CHECK	DATE	SCALE	NOTES	FILE	NO.
KJ	JP/KJ	KJ	02/10	1"=20' 1"=4'	DH	SD	568

APPENDIX E

DETENTION BASIN EXAMPLE CALCULATIONS

STAGE-STORAGE-DISCHARGE DATA

Elevation	Storage	Primary Outlet								Secondary Outlet (Emergency Spillway)		Total Outlet Discharge	Outlet Pipe Discharge	Total Discharge
		Stage 1				Stage 2								
		Weir[1]		Orifice[2]		Weir[1]		Orifice[2]		Weir[1]				
		H	Q	H	Q	H	Q	H	Q	H	Q			
(ft)	(acre-ft)	(ft)	(cfs)	(ft)	(cfs)	(ft)	(cfs)	(ft)	(cfs)	(ft)	(cfs)	(cfs)	(cfs)	(cfs)
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
												0.0		0.0
												0.0		0.0
												0.0		0.0
												0.0		0.0
												0.0		0.0
												0.0		0.0
												0.0		0.0
												0.0		0.0
												0.0		0.0
												0.0		0.0
												0.0		0.0
												0.0		0.0
												0.0		0.0
												0.0		0.0

[1] The Weir Equation ($Q = Cd*L*H^{1.5}$)

[2] The Orifice Equation ($Q=Cd(A)(2*g*(H-d/2))^{0.5}$)

Modified Rational Method
Detention Basin Storage

Example

Given: A 5-acre site currently undeveloped is proposed to be a commercial development.

Determine: Maximum peak discharge and required basin storage to restrict outflows to pre-development conditions.

Predevelopment Maximum Peak Discharges for Standard Frequency Storms

T_c = 20.00 in minutes   **Q = C_{dis} C I A** in cfs

C_{dis} = 1.00833

	2-Year	10-Year	100-Year
C =	0.30	0.30	0.30
I =	3.83734361	5.74540131	8.29935996
A =	5.00	5.00	5.00
Q =	5.80	8.69	12.55

Proposed Development Maximum Peak Discharges for Standard Frequency Storms

T_c = 10.00 in minutes   Q = 1.00833 C I A in cfs

	2-Year	10-Year	100-Year
C =	0.80	0.80	0.80
I =	5.417450451	8.01444909	11.5695944
A =	5.00	5.00	5.00
Q =	21.85	32.32	46.66

MRM Equations

Q = 1.00833 C I A   **$I_i = 60\ Q_i\ T_d$**

$V = I_i\ O_m$ **$O_m = 0.5 \times 60\ Q_m\ (T_c + T_d)$**

Q = Peak discharge using rational method for frequency storm with duration T_c in cfs in accordance with Section 5.3

Q_m = Proposed conditions frequency storm peak outflow discharge in cfs

Q_i = Peak discharge for frequency storm with corresponding duration T_d in cfs

C = Runoff coefficient in accordance with Section 5.3A

I = Rainfall intensity in in/hr as defined by Section 4.0

A = Watershed drainage area in acres

T_c = Watershed drainage area time of concentration in minutes in accordance with Section 5.3B

T_d = Trial storm duration in minutes

I_i = Estimated volume of inflow runoff into basin for trial storm duration T_d in cubic feet

O_m = Estimated volume of outflow runoff from the basin for frequency storm with trial duration T_d in cubic feet

V = Estimated volume of storage in basin necessary to limit the peak outflow discharge from the basin to Q_m in cubic feet

Calculated Basin Volumes

2 Year Frequency Storm

Storm Duration "T_d"(min)	Storm Frequency (Yr)	I(in/hr)	Inflow Peak Discharge Q_i (cfs)	Proposed Outflow Peak Discharge Q_m (cfs)	Estimated Inflow Volume I_i (ft^3)	Estimated Outflow Volume O_m (ft^3)	Estimated Required Storage Volume "V"
10	2	5.417450451	21.85031125	5.80	13110.18675	1741.188907	11368.9978
15	2	4.475228618	18.05002909	5.80	16245.02618	2611.783361	13633.2428
20	2	3.83734361	15.47723473	5.80	18572.68168	3482.377814	15090.3039
25	2	3.373963431	13.60827418	5.80	20412.41128	4352.972268	16059.439
30	2	3.020532616	12.18277461	5.80	21928.9943	5223.566722	16705.4276
35	2	2.741143892	11.05591048	5.80	23217.41202	6094.161175	17123.2508
40	2	2.514152282	10.14038068	5.80	24336.91364	6964.755629	17372.158
45	2	2.325693813	9.380267369	5.80	25326.7219	7835.350082	17491.3718
50	2	2.166456299	8.73801152	5.80	26214.03456	8705.944536	17508.09
55	2	2.029941046	8.187401819	5.80	27018.426	9576.53899	17441.887
60	2	1.911467786	7.70956125	5.80	27754.4205	10447.13344	17307.2871
65	2	1.80757579	7.290531585	5.80	28433.07318	11317.7279	17115.3453
70	2	1.71564807	6.919757674	5.80	29062.98223	12188.32235	16874.6599
75	2	1.633667103	6.589102199	5.80	29650.95989	13058.9168	16592.0431
80	2	1.560051169	6.292185581	5.80	30202.49079	13929.51126	16272.9795
85	2	1.493541793	6.023931983	5.80	30722.05311	14800.10571	15921.9474
90	2	1.433124522	5.780249797	5.80	31213.3489	15670.70016	15542.6487
95	2	1.37797204	5.557802188	5.80	31679.47247	16541.29462	15138.1779
100	2	1.327402559	5.35383929	5.80	32123.03574	17411.88907	14711.1467
105	2	1.280848904	5.166073503	5.80	32546.26307	18282.48353	14263.7795

10 Year Frequency Storm

Storm Duration "T_d"(min)	Storm Frequency (Yr)	I(in/hr)	Inflow Peak Q_i (cfs)	Proposed Outflow Peak Q_m (cfs)	Estimated Inflow Volume I_i (ft^3)	Estimated Outflow Volume O_m (ft^3)	Estimated Required Storage Volume "V"
10	10	8.014449089	32.3248378	8.69	19394.90268	2606.967225	16787.9355
15	10	6.666764863	26.88919606	8.69	24200.27645	3910.450838	20289.8256
20	10	5.745401308	23.173042	8.69	27807.6504	5213.934451	22593.716
25	10	5.071380188	20.45449914	8.69	30681.74871	6517.418064	24164.3306
30	10	4.554525971	18.36986069	8.69	33065.74924	7820.901676	25244.8476
35	10	4.14419828	16.71487781	8.69	35101.2434	9124.385289	25976.8581
40	10	3.809645805	15.36552062	8.69	36877.24949	10427.8689	26449.3806
45	10	3.531055817	14.24187805	8.69	38453.07073	11731.35251	26721.7182
50	10	3.295055986	13.29001521	8.69	39870.04563	13034.83613	26835.2095
55	10	3.0922761	12.47213904	8.69	41158.05883	14338.31974	26819.7391
60	10	2.91594426	11.7609363	8.69	42339.3707	15641.80335	26697.5673
65	10	2.761037878	11.13614929	8.69	43430.98224	16945.28697	26485.6953
70	10	2.623748695	10.58241809	8.69	44446.15597	18248.77058	26197.3854
75	10	2.50113378	10.0878729	8.69	45395.42804	19552.25419	25843.1739
80	10	2.390880941	9.643187918	8.69	46287.30201	20855.7378	25431.5642
85	10	2.291146927	9.240928723	8.69	47128.73649	22159.22142	24969.5151
90	10	2.200443263	8.875091822	8.69	47925.49584	23462.70503	24462.7908

10 Year Frequency Storm (continued from above)

Storm Duration "T_d"(min)	Storm Frequency (Yr)	I(in/hr)	Inflow Peak Q_i (cfs)	Proposed Outflow Peak Q_m (cfs)	Estimated Inflow Volume I_i (ft^3)	Estimated Outflow Volume O_m (ft^3)	Estimated Required Storage Volume "V"
95	10	2.117554072	8.540773191	8.69	48682.40719	24766.18864	23916.2185
100	10	2.041475851	8.233925379	8.69	49403.55228	26069.67225	23333.88
105	10	1.971372625	7.951176638	8.69	50092.41282	27373.15587	22719.257

100 Year Frequency Storm

Storm Duration "T_d"(min)	Storm Frequency (Yr)	I(in/hr)	Inflow Peak Q_i (cfs)	Proposed Outflow Peak Q_m (cfs)	Estimated Inflow Volume I_i (ft^3)	Estimated Outflow Volume O_m (ft^3)	Estimated Required Storage Volume "V"
10	100	11.56959442	46.66387658	12.55	27998.32595	3765.822134	24232.5038
15	100	9.624554885	38.81890971	12.55	34937.01874	5648.733201	29288.2855
20	100	8.299359963	33.47397453	12.55	40168.76943	7531.644269	32637.1252
25	100	7.331677602	29.5710019	12.55	44356.50286	9414.555336	34941.9475
30	100	6.590348783	26.58098555	12.55	47845.774	11297.4664	36548.3076
35	100	6.002084264	24.2083265	12.55	50837.48566	13180.37747	37657.1082
40	100	5.522530863	22.27413418	12.55	53457.92203	15063.28854	38394.6335
45	100	5.123181382	20.66342993	12.55	55791.26082	16946.1996	38845.0612
50	100	4.784829054	19.29874672	12.55	57896.24017	18829.11067	39067.1295
55	100	4.494032804	18.12587239	12.55	59815.37889	20712.02174	39103.3571
60	100	4.24108805	17.10566526	12.55	61580.39492	22594.93281	38985.4621
65	100	4.018802249	16.20911549	12.55	63215.55039	24477.84387	38737.7065
70	100	3.82172473	15.41423879	12.55	64739.8029	26360.75494	38379.048
75	100	3.645644982	14.70405282	12.55	66168.23769	28243.66601	37924.5717
80	100	3.487255831	14.06521869	12.55	67513.04971	30126.57707	37386.4726
85	100	3.343921339	13.48710482	12.55	68784.23456	32009.48814	36774.7464
90	100	3.213513174	12.96112696	12.55	69990.08556	33892.39921	36097.6863
95	100	3.094292896	12.48027342	12.55	71137.55851	35775.31028	35362.2482
100	100	2.984825741	12.03875736	12.55	72232.54413	37658.22134	34574.3228
105	100	2.883916448	11.63175789	12.55	73280.07469	39541.13241	33738.9423

Required Minimum Basin Storage Above Normal Pool Level

2 Year Frequency Storm	V =	17508.09	cubic feet	at Q_m =	5.80	cfs
10 Year Frequency Storm	V =	26835.21	cubic feet	at Q_m =	8.69	cfs
100 Year Frequency Storm	V =	39103.36	cubic feet	at Q_m =	12.55	cfs

TRIANGULAR HYDROGRAPH METHOD
Detention Basin Storage

Example

Given: A 5-acre site currently undeveloped is proposed to be an industrial development.

Determine: Maximum peak discharge and required pond storage to restrict outflow to peak pre-development conditions.

Predevelopment Maximum Peak Discharges

T_c = 20.00 in minutes **Q = C_{dis} C I A** in cfs

C_{dis} = 1.00833

The calculations below are based on the 2005 Drainage Design Manual requirements. Current applications will require the use of the above standard formula with the standard frequency storms of 2-year, 10-year and 100-year storms and the HEC-HMS hydrology program provided by the US Armay Corps of Engineers.

	2-Year	25-Year	100-Year
C =	0.30	0.30	0.30
I =	3.8	7.4	8.4
A =	5.00	5.00	5.00
Q =	5.7	11.1	12.6

Inflow Hydrographs

T_c = 10.00 in minutes

Peak Flows in cfs

	2-Year	25-Year	100-Year
C =	0.85	0.85	0.85
I =	5.4	10.2	11.4
A =	5.00	5.00	5.00
Q =	23.0	43.4	48.5

Hydrograph Ordinates in cfs

Time (min)	2-Year	25-Year	100-Year
0.0	0.0	0.0	0.0
10.0	23.0	43.4	48.5
20.0	13.8	26.0	29.1
30.0	6.9	13.0	14.5
40.0	0.0	0.0	0.0

Stage-Discharge-Storage

Stage	Pipe Discharge	Y-inlet Discharge	Spillway Discharge	Total Discharge	Storage (cubic feet)	Storage (acre-feet)
460.0	0.0	0.0	0.0	0.0	0	0.000
462.0	5.7	0.0	0.0	5.7	17400	0.399
463.0	11.1	0.0	0.0	11.1	35900	0.824
463.2	11.2	1.4	0.0	12.6	38400	0.882
464.0	11.4	6.0	8.6	26.0	52000	1.194

```
1*****************************************
***************************************
 *                                            *                                                         *
*
 *   FLOOD HYDROGRAPH PACKAGE  (HEC-1)   *                                                      *    U.S.
ARMY CORPS OF ENGINEERS     *
 *                 JUN   1998             *                                                         *
HYDROLOGIC ENGINEERING CENTER    *
 *              VERSION 4.1               *                                                         *
609 SECOND STREET         *
 *                                            *                                                         *
DAVIS, CALIFORNIA 95616       *
 *  RUN DATE   12DEC01  TIME  13:21:24   *                                                          *
(916) 756-1104          *
 *                                            *                                                         *
*
 *****************************************
***************************************




                                                  X     X  XXXXXXX   XXXXX          X
                                                  X     X  X        X     X        XX
                                                  X     X  X        X               X
                                                  XXXXXXX  XXXX     X       XXXXX   X
                                                  X     X  X        X               X
                                                  X     X  X        X     X         X
                                                  X     X  XXXXXXX   XXXXX         XXX



            THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS,
HEC1DB, AND HEC1KW.

            THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE
1973-STYLE INPUT STRUCTURE.
            THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS
THE FORTRAN77 VERSION
            NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE , SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE
STAGE FREQUENCY,
            DSS:READ  TIME  SERIES  AT  DESIRED  CALCULATION  INTERVAL      LOSS  RATE:GREEN  AND  AMPT
INFILTRATION
            KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM


1                                                                                                HEC-1  INPUT
PAGE  1

           LINE
ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

              1           ID        TRIANGULAR HYDROGRAPH METHOD
              2           ID        EXAMPLE - CITY OF GRAND PRAIRIE
              3              ID          2-, 25-, & 100-YEAR FLOODS                            FILE =
EXAMPLE1.IH1
              4           IT       1 01NOV90    1600      41
              5           IN      10
              6           IO       5       1

              7           KK   INFLO
              8           KM        2-YEAR FLOOD
              9           KM        POND INFLOW
             10           BA   .0078
             11           QI      .1    23.0    13.8     6.9      .1

             12           KK  OUTFLO
             13           KM        2-YEAR FLOOD
             14           KM        POND DISCHARGE
             15           RS       1    ELEV     460
             16           SV       0    .399    .824    .882   1.194
             17           SE     460     462     463   463.2     464
             18           SQ       0     5.7    11.1    12.6    26.0

             19           KK   INFLO
             20           KM        25-YEAR FLOOD
             21           KM        POND INFLOW
             22           BA   .0078
```

```
23          QI      .1    43.4    26.0    13.0      .1

24          KK  OUTFLO
25          KM        25-YEAR FLOOD
26          KM        POND DISCHARGE
27          RS       1    ELEV     460
28          SV       0    .399    .824    .882   1.194
29          SE     460     462     463   463.2     464
30          SQ       0     5.7    11.1    12.6    26.0

31          KK   INFLO
32          KM        100-YEAR FLOOD
33          KM        POND INFLOW
34          BA   .0078
35          QI      .1    48.5    29.1    14.5      .1

36          KK  OUTFLO
37          KM        100-YR FLOOD
38          KM        POND DISCHARGE
39          RS       1    ELEV     460
40          SV       0    .399    .824    .882   1.194
41          SE     460     462     463   463.2     464
42          SQ       0     5.7    11.1    12.6    26.0
43          ZZ
```

```
1*****************************************
***************************************
 *                                          *                                                       *
*
 *   FLOOD HYDROGRAPH PACKAGE  (HEC-1)   *                                                       *    U.S.
ARMY CORPS OF ENGINEERS     *
 *                 JUN   1998               *                                                       *
HYDROLOGIC ENGINEERING CENTER    *
 *              VERSION 4.1                 *                                                       *
609 SECOND STREET         *
 *                                          *                                                       *
DAVIS, CALIFORNIA 95616       *
 *  RUN DATE   12DEC01  TIME  13:21:24   *                                                          *
(916) 756-1104          *
 *                                          *                                                       *
*
 *****************************************
***************************************



                                 TRIANGULAR HYDROGRAPH METHOD
                                 EXAMPLE - CITY OF GRAND PRAIRIE
                                 2-, 25-, & 100-YEAR FLOODS                      FILE = EXAMPLE1.IH1

    6 IO          OUTPUT CONTROL VARIABLES
                        IPRNT           5  PRINT CONTROL
                        IPLOT           1  PLOT CONTROL
                        QSCAL          0.  HYDROGRAPH PLOT SCALE

      IT          HYDROGRAPH TIME DATA
                         NMIN           1  MINUTES IN COMPUTATION INTERVAL
                        IDATE      1NOV90  STARTING DATE
                        ITIME        1600  STARTING TIME
                           NQ          41  NUMBER OF HYDROGRAPH ORDINATES
                       NDDATE      1NOV90  ENDING DATE
                       NDTIME        1640  ENDING TIME
                       ICENT           19  CENTURY MARK

                    COMPUTATION INTERVAL     .02 HOURS
                         TOTAL TIME BASE     .67 HOURS

           ENGLISH UNITS
                DRAINAGE AREA         SQUARE MILES
                PRECIPITATION DEPTH   INCHES
                LENGTH, ELEVATION     FEET
                FLOW                  CUBIC FEET PER SECOND
                STORAGE VOLUME        ACRE-FEET
                SURFACE AREA          ACRES
                TEMPERATURE           DEGREES FAHRENHEIT
1
                                                           RUNOFF SUMMARY
                                                   FLOW IN CUBIC FEET PER SECOND
                                                TIME IN HOURS,  AREA IN SQUARE MILES

                                          PEAK   TIME OF      AVERAGE FLOW FOR MAXIMUM PERIOD       BASIN
MAXIMUM     TIME OF
          OPERATION       STATION      FLOW      PEAK                                                AREA
STAGE     MAX STAGE
+                                                            6-HOUR     24-HOUR     72-HOUR

          HYDROGRAPH AT
+                          INFLO        23.      .17          11.         11.         11.         .01

          ROUTED TO
+                         OUTFLO         6.      .52           4.          4.          4.         .01
+
462.04         .52

          HYDROGRAPH AT
+                          INFLO        43.      .17          21.         21.         21.         .01

          ROUTED TO
+                         OUTFLO        11.      .53           7.          7.          7.         .01
+
462.93         .53

          HYDROGRAPH AT
```

```
+                          INFLO        49.      .17          23.          23.          23.          .01

         ROUTED TO
+                          OUTFLO       13.      .52           8.           8.           8.          .01
+
463.21          .52


 *** NORMAL END OF HEC-1 ***
```

DIMENSIONLESS HYDROGRAPH METHOD
Detention Pond Storage

Example

Given: A 250-acre site currently undeveloped is proposed to be a residential development.

Determine: Maximum peak discharge and required pond storage.

Rainfall Duration

For drainage areas from 10- to 500-acres a 3-hour storm is used.

Maximum Peak Discharge

Time of Lag (Tp)

Tc =	46.0	min	
Tp =	0.6 Tc =	0.46	hr

Curve Number (CN)

% Sand:		20
Sand:	CN =	56
Clay:	CN =	60
Composite:	CN =	59

Pond Inflow Hydrograph

Time of Lag (Tp)

Tc =	32.0	min	
Tp =	0.6 Tc =	0.32	hr

Curve Number (CN)

% Sand:		20
Sand:	CN =	74
Clay:	CN =	78
Composite:	CN =	77

Stage-Discharge-Storage

Stage	Pipe Discharge	Drop Inlet Discharge	Spillway Discharge	Total Discharge	Storage (ac-ft)
460.0	0	0	0	0	0.0
464.0	28	0	0	28	12.0
468.0	228	0	0	228	28.0
472.0	260	139	0	399	37.4
476.0	270	250	230	750	50.0

```
1*****************************************
***************************************
 *                                                                                                    *
*                                       *
 *                   FLOOD      HYDROGRAPH      PACKAGE           (HEC-1)                             *
*    U.S. ARMY CORPS OF ENGINEERS       *
 *                                       JUN       1998                                               *
*    HYDROLOGIC ENGINEERING CENTER      *
 *                                 VERSION   4.1                                                      *
*          609 SECOND STREET            *
 *                                                                                                    *
*       DAVIS, CALIFORNIA 95616         *
 *          RUN     DATE              19JAN02          TIME          14:20:14                         *
*           (916) 756-1104              *
 *                                                                                                    *
*                                       *
 *****************************************
***************************************


                                                  X     X  XXXXXXX   XXXXX          X
                                                  X     X  X        X     X        XX
                                                  X     X  X        X               X
                                                  XXXXXXX  XXXX     X       XXXXX   X
                                                  X     X  X        X               X
                                                  X     X  X        X     X         X
                                                  X     X  XXXXXXX   XXXXX         XXX
```

THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.
THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION
NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE , SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY,
DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL   LOSS RATE:GREEN AND AMPT INFILTRATION
KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM

```
1                                                                                        HEC-1  INPUT
PAGE  1

           LINE
ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

              1           ID        DIMENSIONLESS HYDROGRAPH METHOD
              2           ID        EXAMPLE - CITY OF GRAND PRAIRIE
              3           ID         2-, 25-, & 100-YEAR FLOODS                                FILE
= EXAMPLE2.IH1
              4           IT       2 01NOV90    1600     181
              5           IO       5       1

              6           KK     PRE
              7           KM        2-YEAR FLOOD
              8           KM        PRE-PROJECT PEAK DISCHARGE
              9           BA   .3906
             10           LS       0      59
                          *       1         0     0.39     0.76     1.49     1.81     1.99
2.41    2.80    3.21
                          *       2         0     0.49     1.04     1.85     2.22     2.45
2.91    3.45    3.95
```

```
*     5     0   0.57   1.22   2.45   3.00   3.30
3.90   4.70   5.40
*    10     0   0.63   1.36   2.86   3.55   3.85
4.65   5.50   6.40
*    25     0   0.73   1.56   3.35   4.15   4.55
5.45   6.50   7.50
*    50     0   0.80   1.71   3.82   4.65   5.15
6.20   7.35   8.52
*   100     0   0.87   1.87   4.32   5.20   5.70
6.92   8.40   9.55
*   500     0   1.00   2.20   5.40   6.60   7.40
8.80  10.50  12.00
PH       2       0    0.49    1.04    1.85    2.22    2.45
UD     .46

KK    POST
KM        2-YEAR FLOOD
KM        POND INFLOW
BA   .3906
LS       0      77
UD     .32

KK    POND
KM        2-YEAR FLOOD
KM        POND DISCHARGE
RS       1    ELEV     460
SV       0      12      28    37.4      50
SE     460     464     468     472     476
SQ       0      28     228     399     750

KK     PRE
KM        25-YEAR FLOOD
KM        PRE-PROJECT PEAK DISCHARGE
BA   .3906
LS       0      59
PH      25       0    0.73    1.56    3.35    4.15    4.55
UD     .46

KK    POST
KM        25-YEAR FLOOD
KM        POND INFLOW
BA   .3906
LS       0      77
UD     .32
```

```
1                                                                                          HEC-1 INPUT
PAGE  2

           LINE
ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10


            39            KK    POND
            40            KM        25-YEAR FLOOD
            41            KM        POND DISCHARGE
            42            RS       1    ELEV     460
            43            SV       0      12      28    37.4      50
            44            SE     460     464     468     472     476
            45            SQ       0      28     228     399     750

            46            KK     PRE
            47            KM        100-YEAR FLOOD
            48            KM        PRE-PROJECT PEAK DISCHARGE
            49            BA   .3906
            50            LS       0      59
            51            PH     100       0    0.87    1.87    4.32    5.20    5.70
            52            UD     .46

            53            KK    POST
            54            KM        100-YEAR FLOOD
            55            KM        POND INFLOW
            56            BA   .3906
            57            LS       0      77
            58            UD     .32

            59            KK    POND
            60            KM        100-YEAR FLOOD
            61            KM        POND DISCHARGE
            62            RS       1    ELEV     460
            63            SV       0      12      28    37.4      50
            64            SE     460     464     468     472     476
            65            SQ       0      28     228     399     750
            66            ZZ

1*****************************************
***************************************
 *                                                                                          *
*                                      *
 *                  FLOOD      HYDROGRAPH      PACKAGE           (HEC-1)                   *
*    U.S. ARMY CORPS OF ENGINEERS      *
 *                                       JUN        1998                                    *
*    HYDROLOGIC ENGINEERING CENTER     *
 *                                VERSION   4.1                                             *
*          609 SECOND STREET           *
 *                                                                                          *
*       DAVIS, CALIFORNIA 95616        *
 *          RUN     DATE            19JAN02         TIME         14:20:14                   *
*           (916) 756-1104             *
 *                                                                                          *
*                                      *
 ****************************************
***************************************




                                 DIMENSIONLESS HYDROGRAPH METHOD
                                 EXAMPLE - CITY OF GRAND PRAIRIE
                                 2-, 25-, & 100-YEAR FLOODS                        FILE =
EXAMPLE2.IH1

    5 IO          OUTPUT CONTROL VARIABLES
                        IPRNT           5  PRINT CONTROL
                        IPLOT           1  PLOT CONTROL
```

```
                  QSCAL          0.  HYDROGRAPH PLOT SCALE

  IT          HYDROGRAPH TIME DATA
                   NMIN          2  MINUTES IN COMPUTATION INTERVAL
                  IDATE     1NOV90  STARTING DATE
                  ITIME       1600  STARTING TIME
                     NQ        181  NUMBER OF HYDROGRAPH ORDINATES
                 NDDATE     1NOV90  ENDING DATE
                 NDTIME       2200  ENDING TIME
                 ICENT          19  CENTURY MARK

            COMPUTATION INTERVAL     .03 HOURS
                 TOTAL TIME BASE    6.00 HOURS

       ENGLISH UNITS
            DRAINAGE AREA         SQUARE MILES
            PRECIPITATION DEPTH   INCHES
            LENGTH, ELEVATION     FEET
            FLOW                  CUBIC FEET PER SECOND
            STORAGE VOLUME        ACRE-FEET
            SURFACE AREA          ACRES
            TEMPERATURE           DEGREES FAHRENHEIT
```

```
1
                                                                      RUNOFF SUMMARY
                                                                FLOW IN CUBIC FEET PER SECOND
                                                             TIME IN HOURS,  AREA IN SQUARE MILES

                                       PEAK   TIME OF     AVERAGE FLOW FOR MAXIMUM PERIOD
BASIN     MAXIMUM     TIME OF
          OPERATION                     STATION                  FLOW                   PEAK
AREA      STAGE     MAX STAGE
+                                                   6-HOUR     24-HOUR     72-HOUR

          HYDROGRAPH AT
+                             PRE         28.     2.33          6.          6.          6.
.39

          HYDROGRAPH AT
+                            POST        216.     1.93         30.         30.         30.
.39

          ROUTED TO
+                            POND         27.     3.27         15.         15.         15.
.39
+
463.88        3.27

          HYDROGRAPH AT
+                             PRE        226.     2.17         41.         41.         41.
.39

          HYDROGRAPH AT
+                            POST        653.     1.93         94.         94.         94.
.39

          ROUTED TO
+                            POND        226.     2.53         74.         74.         74.
.39
+
467.97        2.53

          HYDROGRAPH AT
+                             PRE        396.     2.17         69.         69.         69.
.39

          HYDROGRAPH AT
+                            POST        934.     1.93        135.        135.        135.
.39

          ROUTED TO
+                            POND        396.     2.43        113.        113.        113.
.39
+
471.93        2.43


 *** NORMAL END OF HEC-1 ***
```

DIMENSIONLESS HYDROGRAPH METHOD
Detention Pond Storage

Example

Given: A 250-acre site currently undeveloped is proposed to be developed. The watershed is sub-divided in to three watersheds of 50-, 80-, and 120-acres. The 80-acre watershed is upstream of the 120-acre watershed. The 50- and 120-acre watersheds drain directly in to the proposed pond.

Determine: Maximum peak discharge and required pond storage.

Rainfall Duration

For drainage areas from 10- to 500-acres a 3-hour storm is used.

Maximum Peak Discharge

Sub-Watershed Data

Sub-Watershed	Area (ac)	Existing Land Use (%) % Sand	Undeveloped
1	80.0	30.0	100
2	120.0	10.0	100
3	50.0	20.0	100

Time of Lag (Tp)

Sub-Watershed	Tc (min)	Tp (hr)
1	28.0	0.280
2	38.0	0.380
3	22.0	0.220

Curve Number (CN)

Land Use	Sandy Soil CN	Clay Soil CN
Undeveloped	56	60

Sub-Watershed	Sandy Soil CN	Clay Soil CN	Composite CN
1	56	60	59
2	56	60	60
3	56	60	59

Pond Inflow Hydrograph

Sub-Watershed Data

Sub-Watershed	Area (ac)	% Sand	Proposed Land Use (%) Residential	Commericial	Industrial
1	80.0	30.0	70	20	10
2	120.0	10.0	100	0	0
3	50.0	20.0	80	20	0

Time of Lag (Tp)

Sub-Watershed	Tc (min)	Tp (hr)
1	19.0	0.190
2	26.0	0.260
3	16.0	0.160

Curve Number (CN)

Land Use	Sandy Soil CN	Clay Soil CN
Residential	74	78
Commericial	82	86
Industrial	82	86

Sub-Watershed	Sandy Soil CN	Clay Soil CN	Composite CN
1	76	80	79
2	74	78	78
3	76	80	79

Percent Impervious (%Imp)

Sub-Watershed	%Imp
1	26
2	0
3	17

Stage-Discharge-Storage

Stage	Pipe Discharge	Drop Inlet Discharge	Spillway Discharge	Total Discharge	Storage (ac-ft)
460.0	0	0	0	0	0.0
464.0	26	0	0	26	13.1
468.0	222	0	0	222	32.0
472.0	260	101	0	361	43.0
476.0	270	210	220	700	50.0

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1***************************************
***************************************
 *                                                                                                                              *
*                                      *
 *                  FLOOD      HYDROGRAPH      PACKAGE           (HEC-1)                 *
*    U.S. ARMY CORPS OF ENGINEERS     *
 *                                     JUN       1998                                                                  *
*    HYDROLOGIC ENGINEERING CENTER    *
 *                                VERSION   4.1                                                                   *
*          609 SECOND STREET         *
 *                                                                                                                              *
*       DAVIS, CALIFORNIA 95616       *
 *         RUN     DATE            19JAN02         TIME         14:16:30               *
*          (916) 756-1104            *
 *                                                                                                                              *
*                                      *
 ***************************************
***************************************


                                                  X     X  XXXXXXX   XXXXX         X
                                                  X     X  X        X     X       XX
                                                  X     X  X        X              X
                                                  XXXXXXX  XXXX     X       XXXXX  X
                                                  X     X  X        X              X
                                                  X     X  X        X     X        X
                                                  X     X  XXXXXXX   XXXXX        XXX


            THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73),
HEC1GS, HEC1DB, AND HEC1KW.

            THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED
WITH THE 1973-STYLE INPUT STRUCTURE.
            THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP
81. THIS IS THE FORTRAN77 VERSION
            NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE , SINGLE EVENT DAMAGE CALCULATION,
DSS:WRITE STAGE FREQUENCY,
            DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL    LOSS RATE:GREEN AND
AMPT INFILTRATION
            KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM


1                                                                              HEC-1 INPUT
PAGE  1

           LINE
ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

              1           ID        DIMENSIONLESS HYDROGRAPH METHOD
              2           ID        EXAMPLE - CITY OF GRAND PRAIRIE
              3           ID         2-, 25-, & 100-YEAR FLOODS                          FILE
= EXAMPLE3.IH1
              4           IT       2 01NOV90    1600     181
              5           IO       5       1

              6           KK   AREA1
              7           KM        2-YEAR FLOOD
              8           KM        PRE-PROJECT PEAK DISCHARGE
              9           BA   .1250
             10           LS       0      59
                          *        1        0     0.39     0.76     1.49     1.81     1.99
2.41    2.80    3.21
                          *        2        0     0.49     1.04     1.85     2.22     2.45
2.91    3.45    3.95
```

```
                          *       5         0     0.57     1.22     2.45     3.00     3.30
    3.90    4.70    5.40
                          *      10         0     0.63     1.36     2.86     3.55     3.85
    4.65    5.50    6.40
                          *      25         0     0.73     1.56     3.35     4.15     4.55
    5.45    6.50    7.50
                          *      50         0     0.80     1.71     3.82     4.65     5.15
    6.20    7.35    8.52
                          *     100         0     0.87     1.87     4.32     5.20     5.70
    6.92    8.40    9.55
                          *     500         0     1.00     2.20     5.40     6.60     7.40
    8.80   10.50   12.00
                 11       PH       2       0    0.49    1.04    1.85    2.22    2.45
                 12       UD     .28

                 13       KK   ROUT2
                 14       RD
                 15       RC    .090    .070    .090    3400    .008
                 16        RX        0       50      100      101      105      106      156
    206
                 17        RY      114      104      103      100      100      103      104
    114

                 18       KK   AREA2
                 19       BA   .1875
                 20       LS       0      60
                 21       UD     .38

                 22       KK   COMB2
                 23       HC       2

                 24       KK   AREA3
                 25       BA   .0781
                 26       LS       0      59
                 27       UD     .22

                 28       KK   COMB3
                 29       HC       2

                 30       KK   AREA1
                 31       KM        POST-PROJECT
                 32       BA   .1250
                 33       LS       0      79      26
                 34       UD     .19
```

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1                                                                          HEC-1 INPUT
PAGE  2

         LINE
ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10


          35          KK   ROUT2
          36          RD    3100    .009    .015       0    TRAP      10       2

          37          KK   AREA2
          38          BA   .1875
          39          LS       0      78       0
          40          UD     .26

          41          KK   COMB2
          42          HC       2

          43          KK   AREA3
          44          BA   .0781
          45          LS       0      79      17
          46          UD     .16

          47          KK   COMB3
          48          HC       2

          49          KK    POND
          50          RS       1    ELEV     460
          51          SV       0    18.5    36.5    48.5      54
          52          SE     460     464     468     472     476
          53          SQ       0      26     222     361     700

          54          KK   AREA1
          55          KM        25-YEAR FLOOD
          56          KM        PRE-PROJECT PEAK DISCHARGE
          57          BA   .1250
          58          LS       0      59
          59          PH      25       0    0.73    1.56    3.35    4.15    4.55
          60          UD     .28

          61          KK   ROUT2
          62          RD
          63          RC    .090    .070    .090    3400    .008
          64           RX        0       50      100      101      105      106      156
206
          65           RY      114      104      103      100      100      103      104
114

          66          KK   AREA2
          67          BA   .1875
          68          LS       0      60
          69          UD     .38

          70          KK   COMB2
          71          HC       2

          72          KK   AREA3
          73          BA   .0781
          74          LS       0      59
          75          UD     .22
```

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1                                                                        HEC-1 INPUT
PAGE  3

          LINE
ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10


            76           KK   COMB3
            77           HC       2

            78           KK   AREA1
            79           KM        POST-PROJECT
            80           BA   .1250
            81           LS       0      79      26
            82           UD     .19

            83           KK   ROUT2
            84           RD    3100    .009    .015       0    TRAP      10       2

            85           KK   AREA2
            86           BA   .1875
            87           LS       0      78       0
            88           UD     .26

            89           KK   COMB2
            90           HC       2

            91           KK   AREA3
            92           BA   .0781
            93           LS       0      79      17
            94           UD     .16

            95           KK   COMB3
            96           HC       2

            97           KK    POND
            98           RS       1    ELEV     460
            99           SV       0    18.5    36.5    48.5      54
           100           SE     460     464     468     472     476
           101           SQ       0      26     222     361     700

           102           KK   AREA1
           103           KM        100-YEAR FLOOD
           104           KM        PRE-PROJECT PEAK DISCHARGE
           105           BA   .1250
           106           LS       0      59
           107           PH     100       0    0.87    1.87    4.32    5.20    5.70
           108           UD     .28

           109           KK   ROUT2
           110           RD
           111           RC    .090    .070    .090    3400    .008
           112             RX        0       50      100      101      105      106      156
206
           113             RY      114      104      103      100      100      103      104
114

           114           KK   AREA2
           115           BA   .1875
           116           LS       0      60
           117           UD     .38
```

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1                                                                                      HEC-1 INPUT
PAGE  4

          LINE
ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10


           118            KK   COMB2
           119            HC       2

           120            KK   AREA3
           121            BA   .0781
           122            LS       0      59
           123            UD     .22

           124            KK   COMB3
           125            HC       2

           126            KK   AREA1
           127            KM        POST-PROJECT
           128            BA   .1250
           129            LS       0      79      26
           130            UD     .19

           131            KK   ROUT2
           132            RD    3100    .009    .015       0    TRAP      10       2

           133            KK   AREA2
           134            BA   .1875
           135            LS       0      78       0
           136            UD     .26

           137            KK   COMB2
           138            HC       2

           139            KK   AREA3
           140            BA   .0781
           141            LS       0      79      17
           142            UD     .16

           143            KK   COMB3
           144            HC       2

           145            KK    POND
           146            RS       1    ELEV     460
           147            SV       0    18.5    36.5    48.5      54
           148            SE     460     464     468     472     476
           149            SQ       0      26     222     361     700
           150            ZZ
```

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1*****************************************
***************************************
 *                                                                                               *
*                                      *
 *                  FLOOD      HYDROGRAPH      PACKAGE           (HEC-1)                         *
*    U.S. ARMY CORPS OF ENGINEERS      *
 *                                     JUN       1998                                            *
*    HYDROLOGIC ENGINEERING CENTER     *
 *                               VERSION   4.1                                                   *
*          609 SECOND STREET           *
 *                                                                                               *
*       DAVIS, CALIFORNIA 95616        *
 *          RUN     DATE             19JAN02          TIME          14:16:30                     *
*           (916) 756-1104             *
 *                                                                                               *
*                                      *
 *****************************************
***************************************



                                 DIMENSIONLESS HYDROGRAPH METHOD
                                 EXAMPLE - CITY OF GRAND PRAIRIE
                                 2-, 25-, & 100-YEAR FLOODS                          FILE =
EXAMPLE3.IH1

    5 IO         OUTPUT CONTROL VARIABLES
                       IPRNT          5  PRINT CONTROL
                       IPLOT          1  PLOT CONTROL
                       QSCAL         0.  HYDROGRAPH PLOT SCALE

      IT         HYDROGRAPH TIME DATA
                        NMIN          2  MINUTES IN COMPUTATION INTERVAL
                       IDATE     1NOV90  STARTING DATE
                       ITIME       1600  STARTING TIME
                          NQ        181  NUMBER OF HYDROGRAPH ORDINATES
                      NDDATE     1NOV90  ENDING DATE
                      NDTIME       2200  ENDING TIME
                      ICENT          19  CENTURY MARK

                   COMPUTATION INTERVAL     .03 HOURS
                        TOTAL TIME BASE    6.00 HOURS

           ENGLISH UNITS
                DRAINAGE AREA         SQUARE MILES
                PRECIPITATION DEPTH   INCHES
                LENGTH, ELEVATION     FEET
                FLOW                  CUBIC FEET PER SECOND
                STORAGE VOLUME        ACRE-FEET
                SURFACE AREA          ACRES
                TEMPERATURE           DEGREES FAHRENHEIT
1
                                                              RUNOFF SUMMARY
                                                      FLOW IN CUBIC FEET PER SECOND
                                                   TIME IN HOURS,  AREA IN SQUARE MILES

                                             PEAK   TIME OF     AVERAGE FLOW FOR MAXIMUM PERIOD
BASIN     MAXIMUM     TIME OF
          OPERATION                      STATION                      FLOW                 PEAK
AREA      STAGE     MAX STAGE
+                                                                 6-HOUR     24-HOUR     72-HOUR

          HYDROGRAPH AT
+                            AREA1        11.    2.07           2.          2.          2.
.13

          ROUTED TO
```

```
+                              ROUT2        10.     2.63           2.          2.          2.
.13

          HYDROGRAPH AT
+                              AREA2        17.     2.20           3.          3.          3.
.19

          2 COMBINED AT
+                              COMB2        22.     2.60           5.          5.          5.
.31

          HYDROGRAPH AT
+                              AREA3         7.     2.00           1.          1.          1.
.08

          2 COMBINED AT
+                              COMB3        26.     2.60           6.          6.          6.
.39

          HYDROGRAPH AT
+                              AREA1       151.     1.73          17.         17.         17.
.13

          ROUTED TO
+                              ROUT2       150.     1.80          17.         17.         17.
.13

          HYDROGRAPH AT
+                              AREA2       125.     1.87          15.         15.         15.
.19

          2 COMBINED AT
+                              COMB2       273.     1.83          32.         32.         32.
.31

          HYDROGRAPH AT
+                              AREA3        90.     1.70           9.          9.          9.
.08

          2 COMBINED AT
+                              COMB3       348.     1.80          41.         41.         41.
.39

          ROUTED TO
+                               POND        24.     3.27          15.         15.         15.
.39
+
463.74        3.27
```

```
          HYDROGRAPH AT
+                         AREA1       91.    1.93          13.         13.         13.
.13
          ROUTED TO
+                         ROUT2       83.    2.67          13.         13.         13.
.13
          HYDROGRAPH AT
+                         AREA2      128.    2.07          21.         21.         21.
.19
          2 COMBINED AT
+                         COMB2      176.    2.13          34.         34.         34.
.31
          HYDROGRAPH AT
+                         AREA3       62.    1.87           8.          8.          8.
.08
          2 COMBINED AT
+                         COMB3      222.    2.10          43.         43.         43.
.39
          HYDROGRAPH AT
+                         AREA1      323.    1.73          40.         40.         40.
.13
          ROUTED TO
+                         ROUT2      323.    1.80          40.         40.         40.
.13
          HYDROGRAPH AT
+                         AREA2      357.    1.83          47.         47.         47.
.19
          2 COMBINED AT
+                         COMB2      674.    1.80          87.         87.         87.
.31
          HYDROGRAPH AT
+                         AREA3      205.    1.70          23.         23.         23.
.08
          2 COMBINED AT
+                         COMB3      854.    1.80         110.        110.        110.
.39
          ROUTED TO
+                          POND      220.    2.47          76.         76.         76.
.39
+
467.96        2.47
          HYDROGRAPH AT
+                         AREA1      159.    1.93          22.         22.         22.
.13
          ROUTED TO
+                         ROUT2      145.    2.57          22.         22.         22.
.13
          HYDROGRAPH AT
+                         AREA2      220.    2.07          35.         35.         35.
.19
          2 COMBINED AT
+                         COMB2      275.    2.10          57.         57.         57.
.31
```

```
          HYDROGRAPH AT
+                  AREA3      108.    1.87          14.        14.        14.
.08

          2 COMBINED AT
+                  COMB3      361.    2.03          71.        71.        71.
.39

          HYDROGRAPH AT
+                  AREA1      427.    1.73          54.        54.        54.
.13

          ROUTED TO
+                  ROUT2      427.    1.80          54.        54.        54.
.13

          HYDROGRAPH AT
+                  AREA2      502.    1.83          67.        67.        67.
.19

          2 COMBINED AT
+                  COMB2      921.    1.80         121.       121.       121.
.31

          HYDROGRAPH AT
+                  AREA3      273.    1.70          32.        32.        32.
.08

          2 COMBINED AT
+                  COMB3     1166.    1.80         153.       153.       153.
.39

          ROUTED TO
+                   POND      357.    2.33         116.       116.       116.
.39
+
471.88        2.33
```

```
1
                                                    SUMMARY  OF  KINEMATIC  WAVE  -  MUSKINGUM-CUNGE
ROUTING
                                                        (FLOW IS DIRECT RUNOFF WITHOUT BASE FLOW)

INTERPOLATED TO

COMPUTATION INTERVAL
            ISTAQ    ELEMENT      DT       PEAK    TIME TO     VOLUME      DT        PEAK
TIME TO     VOLUME
                                                     PEAK
PEAK

                                 (MIN)     (CFS)      (MIN)      (IN)     (MIN)      (CFS)
(MIN)       (IN)

              ROUT2  MANE         1.60     10.19    156.80        .14      2.00      10.17
158.00         .14


 CONTINUITY  SUMMARY  (AC-FT)  -  INFLOW=  .9339E+00  EXCESS=  .0000E+00  OUTFLOW=  .9245E+00
BASIN STORAGE= .7396E-03 PERCENT ERROR=     .9


              ROUT2  MANE         2.00    150.37    108.00       1.23      2.00     150.37
108.00        1.23


 CONTINUITY  SUMMARY  (AC-FT)  -  INFLOW=  .8209E+01  EXCESS=  .0000E+00  OUTFLOW=  .8210E+01
BASIN STORAGE= .1864E-02 PERCENT ERROR=     .0


              ROUT2  MANE         1.80     83.32    160.20        .98      2.00      83.29
160.00         .98


 CONTINUITY  SUMMARY  (AC-FT)  -  INFLOW=  .6579E+01  EXCESS=  .0000E+00  OUTFLOW=  .6544E+01
BASIN STORAGE= .6864E-03 PERCENT ERROR=     .5


              ROUT2  MANE         2.00    323.03    108.00       2.97      2.00     323.03
108.00        2.97


 CONTINUITY  SUMMARY  (AC-FT)  -  INFLOW=  .1981E+02  EXCESS=  .0000E+00  OUTFLOW=  .1981E+02
BASIN STORAGE= .1818E-02 PERCENT ERROR=     .0


              ROUT2  MANE         2.00    145.09    154.00       1.64      2.00     145.09
154.00        1.64


 CONTINUITY  SUMMARY  (AC-FT)  -  INFLOW=  .1099E+02  EXCESS=  .0000E+00  OUTFLOW=  .1094E+02
BASIN STORAGE= .6409E-03 PERCENT ERROR=     .5


              ROUT2  MANE         2.00    426.66    108.00       4.01      2.00     426.66
108.00        4.01


 CONTINUITY  SUMMARY  (AC-FT)  -  INFLOW=  .2670E+02  EXCESS=  .0000E+00  OUTFLOW=  .2671E+02
BASIN STORAGE= .1778E-02 PERCENT ERROR=     .0



 *** NORMAL END OF HEC-1 ***
```

APPENDIX F

GUIDELINES FOR PREPARING LOT GRADING

AND DRAINAGE PLANS FOR LAKERIDGE SUBDIVISION

MEMORANDUM

TO: Lakeridge Subdivision Lot Owners
FROM: City of Grand Prairie Engineering
SUBJECT: **LOT GRADING AND DRAINAGE PLANS**

In order to allow this subdivision to develop as a “rural estate" or "estate residential" type subdivision, the developer was required to put in the major road-crossing drainage systems and to design for some side lot drainage features. The remainder of the drainage features are on or across the individual lots. There were definite drainage easements shown on the plats for some of the drainage systems, and there were some drainage easements indicated only by the statement that there was a 30 foot drainage easement centered on each "natural drainage course". To try to insure that one property owner did not do things that would impede the drainage of others, it is a plat requirement that each lot buyer/owner have prepared and get approved, at the time of building permit application, an individual Lot Grading and Drainage Plan.

The Lot Grading and Drainage Plan must include, at a minimum:

1. A plot plan of the lot showing:
   a. Lot lines,
   b. Street name,
   c. Lot number,
   d. Address,
   e. North arrow,
   f. Scale (at least an approximate scale),
   g. Adjacent lot lines (extending away from said lot 50 feet (+/-)),
   h. Adjacent lot numbers,
   i. Contour lines, and
   j. The general house layout (location).

2. The plan must show any driveway culvert pipes, if any are required. The person preparing the plan must review the subdivision plans to determine if a driveway culvert pipe is required and if so, what size (diameter) and length of pipe will be required (see attached lists of lots that require driveway pipes and their sizes). Note that all driveway culvert pipes shall be reinforced concrete pipe, Class III (or as designated by a Licensed Civil Engineer and approved by the City). Corrugated metal pipe and PVC pipe will not be approved for driveway pipe culverts

3. The lot grading plan shall show high points, low points, and drainage swales, as necessary, to show that positive drainage flow away from any building foundation is being provided. High points and low points shall be identified by HP and LP, respectively, and with a line or an “X”. Ditch checks may need to be provided if velocities are over 6 feet per second.

Generally, there are three different types of lots or situations that will need to be considered when determining who can prepare the Lot Grading and Drainage Plan, and how much information it must have on it. These are:

I. High Point Lot (see Exhibit "A")

If the lot is located generally at the crest of a hill, the builder or the homeowner can prepare the grading plan. The grading plan should show the location of drainage swales on the high side of the lot and a series of arrows showing the direction of the flow of the stormwater on the remainder of the lot. If there is any offsite runoff, the drainage plan should show where the water is coming from (i.e., show what lots are draining onto the property) and indicate where the drainage from this lot will go (show what lots the water will flow onto or what street right of way or drainage system the water will flow into).

II. Major Drainage Lot (see Exhibit "B")

If the lot has significant off-site drainage crossing the lot and/or is one of the lots that has a drainage ditch and easement on it, then the "designed" swale or ditch and any constructed drainage improvements, such as pipe culverts must be shown on the drainage plan along with the Q_{100} slope of pipe, and velocity. These items can generally be obtained from the subdivision plans. This is important in order that the new property owners will know when stormwater flows will be crossing their property, so that they can be sure to keep these drainage ways maintained and open.

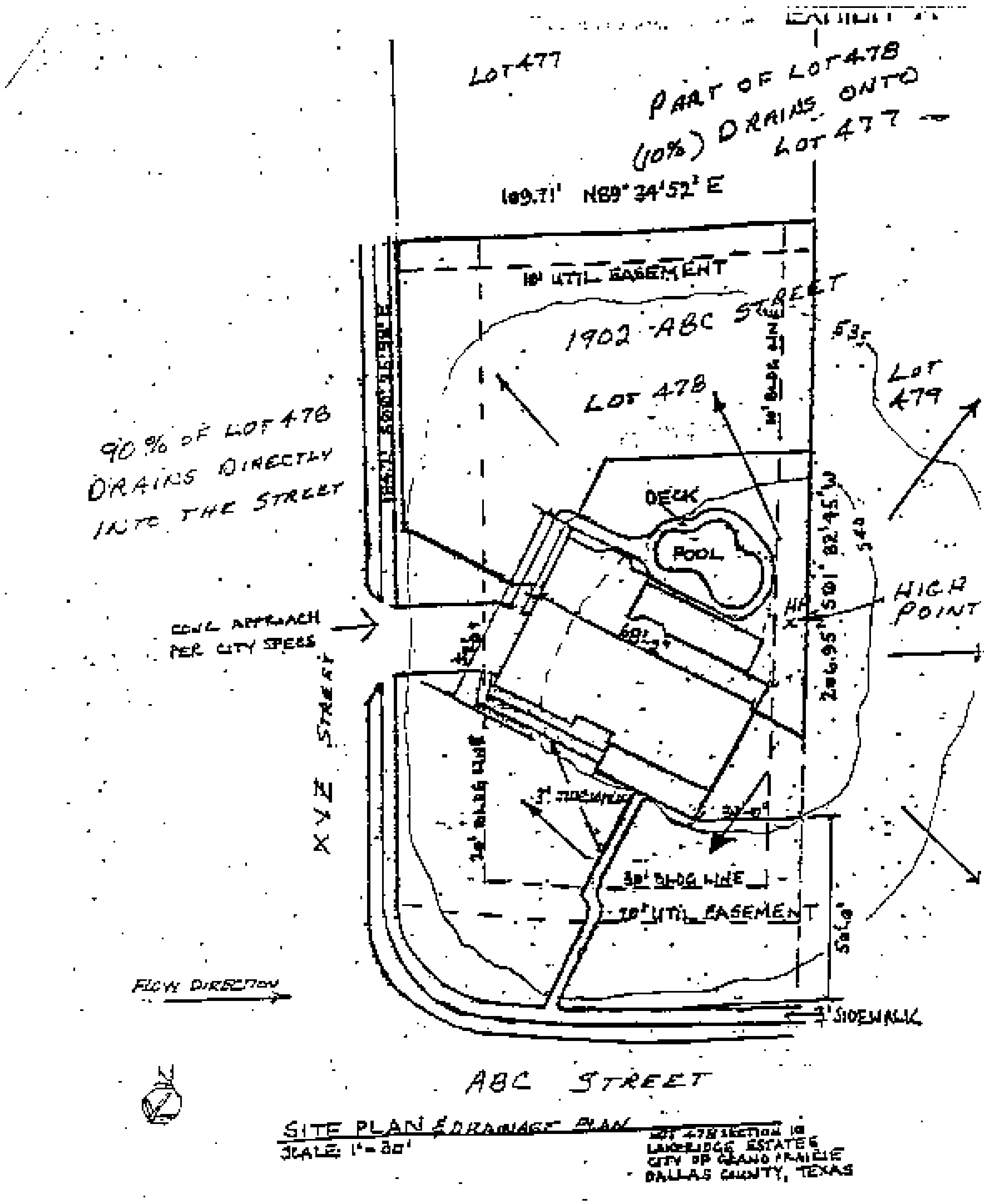
These drainage plans should be prepared by a Licensed Civil Engineer, but may be prepared by the owner or the builder if all required items are shown.

III. Lot with Additional Culvert (see Exhibit “C”)

If an additional on-site pipe culvert, other than a driveway entrance culvert is needed or desired, a Licensed Civil Engineer shall prepare the drainage plan. That culvert size and slope must be designed by a Licensed Civil Engineer, and the 100-year frequency runoff (Q_{100}), the slope of pipe, and the velocity must be shown. Exit velocity must be less than 6.0 feet per second. Headwalls or end treatments must be provided and shown on the plan.

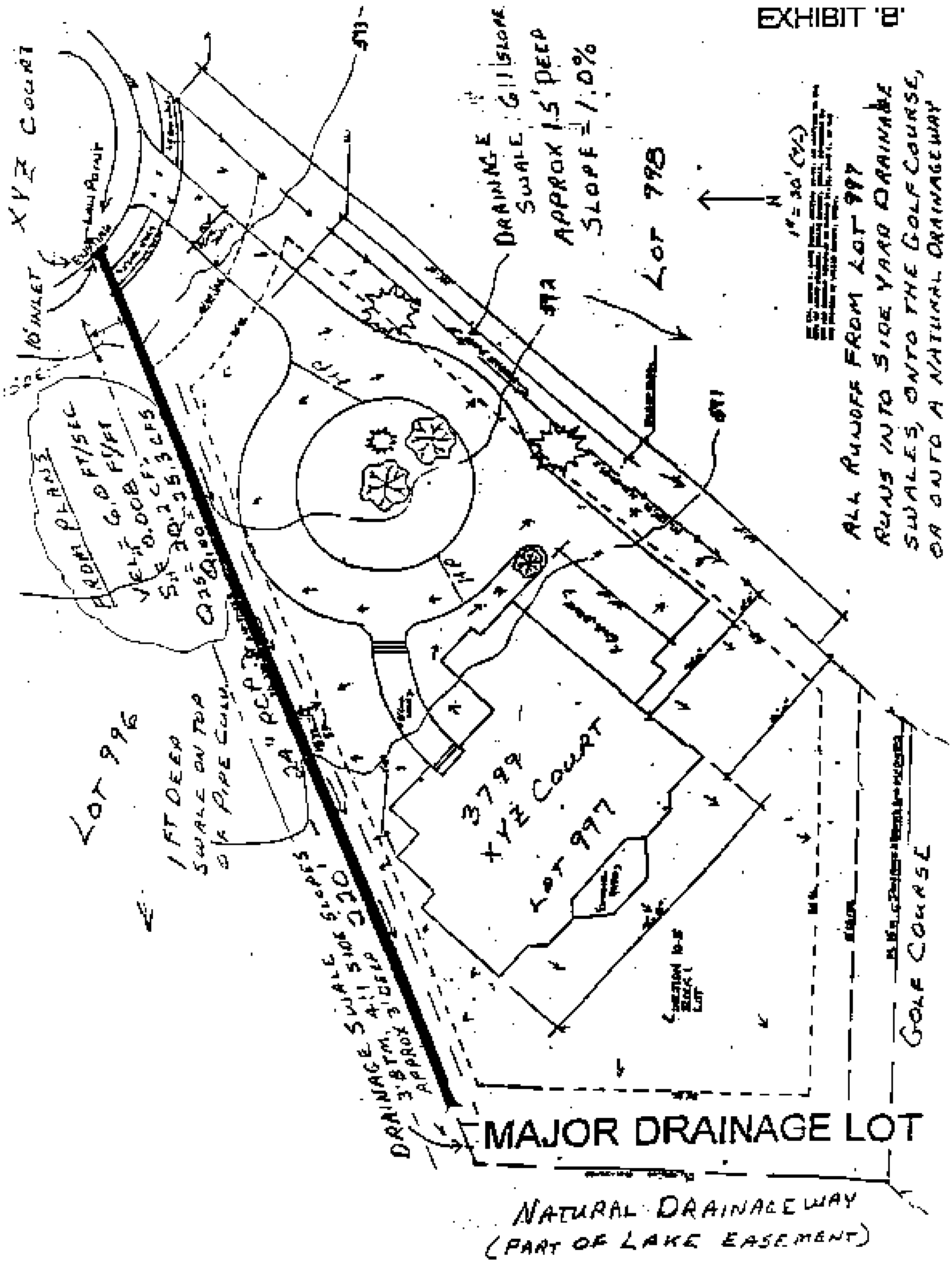
The engineer preparing the drainage plan must review the plat to determine if there is a "natural drainage course" or a "designed" drainage swale on or across the lot. The drainage area map and the contour map must also be reviewed to determine whether there is water from a street and/or from more than one other lot that crosses the lot (public water) for which the drainage plan is being prepared. If there is such off-site drainage, the Q_{100} flow, the "designed" drainage swale cross-section, and any existing pipe or box culvert system(s) that touch the lot shall be shown on the drainage plan.

NOTE: If anyone other than the home owner buyer prepares the grading and drainage plan, the preparer shall provide a copy to the home owner.



HIGH POINT LOT

EXHIBIT 'B'


XYZ COURT
LOT 998
LOT 996
3799 XYZ COURT LOT 997
MAJOR DRAINAGE LOT
GOLF COURSE
ALL RUNOFF FROM LOT 997 RUNS INTO SIDE YARD DRAINAGE SWALES, ONTO THE GOLF COURSE, OR ONTO A NATURAL DRAINAGEWAY
DRAINAGE SWALE 6:1 SLOPE APPROX 1.5' DEEP SLOPE = 1.0%
1 FT DEEP SWALE ON TOP OF PIPE CULV.
NATURAL DRAINAGEWAY (PART OF LAKE EASEMENT)

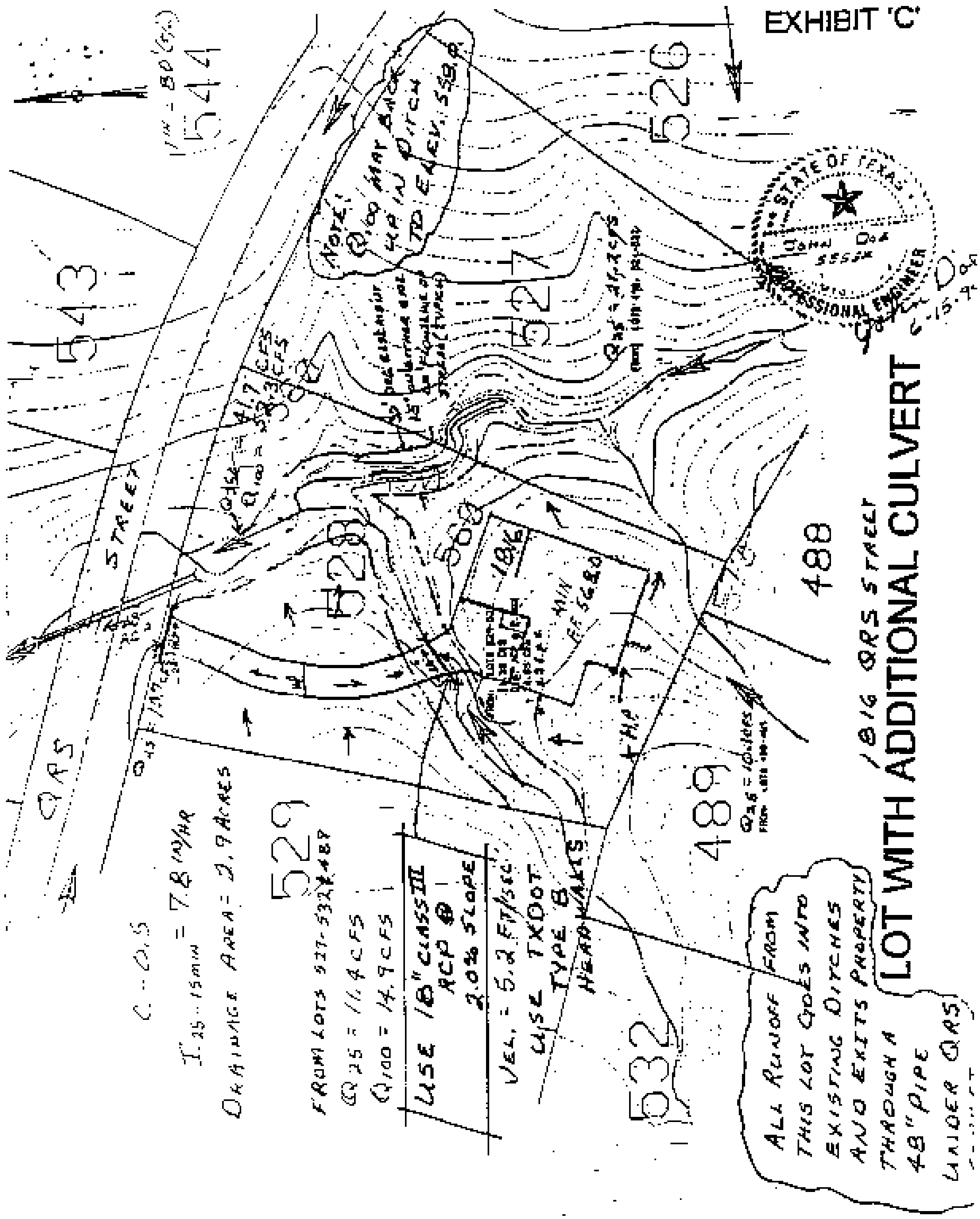


EXHIBIT 'C'
LOT WITH ADDITIONAL CULVERT
1816 QRS STREET
QRS STREET
C = 0.5
I 25 - 15min = 7.8 IN/HR
DRAINAGE AREA = 2.9 ACRES
FROM LOTS 527-532 & 489
Q25 = 11.4 CFS
Q100 = 14.9 CFS
USE 18" CLASS III RCP @ 2.0% SLOPE
VEL. = 5.2 FT/SEC.
USE TXDOT TYPE B HEADWALLS
ALL RUNOFF FROM THIS LOT GOES INTO EXISTING DITCHES AND EXITS PROPERTY THROUGH A 48" PIPE UNDER QRS
NOTE: Q100 MAY BACK UP IN DITCH TO ELEV. 558
Q25 = 41.7 CFS
Q100 = 53.3 CFS
STATE OF TEXAS
JOHN DOE
55555
PROFESSIONAL ENGINEER
1" = 80'

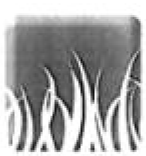
APPENDIX G

SITE DESIGN PRACTICES

APPENDIX G.1

CITY OF GRAND PRAIRIE 2010 COMPREHENSIVE PLAN
(SECTION 9 AND APPENDIX E – FLOODPLAIN MANAGEMENT)

Section 9
Watershed Planning and Environmental Quality



Watershed Planning/Environmental Quality

The City of Grand Prairie takes a proactive approach to environmental quality. Several programs were mentioned in Section 2, "Baseline Analysis," including Green Grand Prairie, the Water Conservation Plan and Solid Waste and Recycling programs.

This section will take a closer look at some of the programs that are related to the physical development of the city, such as Watershed Planning, Floodplain Management, the City-Wide Drainage Master Plan Road Map and the Storm Water Management Program.

Watershed Planning

The city has twelve major water basins that overlap with adjacent cities and counties (see map at the back of this plan). Watershed and storm water master plans can be used to identify drainage and stream segments in need of improvement or restoration, and potential locations for regional storm water control facilities. For additional information, see the *City of Grand Prairie Watershed Technical Report, February 2005.*

Drainage Studies

Ultimately, floodplain elevations will be established for build-out conditions in the watersheds. Watershed planning also takes into account the accumulative impacts of new development on the rivers, lakes, streams and other water bodies in each watershed.

When considering new development proposals, the Engineering Division reviews drainage studies to determine potential impacts on adjacent properties and to determine whether the proposed infrastructure can handle the additional storm water runoff. For additional information, see Article 14, "Drainage" of the Unified Development Code (UDC).

In addition, Storm Water Pollution Prevention Plans (SWPPPs) are submitted with new development proposals to prevent and control erosion. Preventing pollution is important to protect the drinking water supply, maintain a healthy environment for riparian wildlife and preserve water recreation opportunities.

Open Space and Public Waterways

This objective recognizes the importance of watershed planning and floodplain preservation in preventing flooding and for protecting environmental quality. This objective also meets *Goal 7: Maintain and Improve Drainage in the City Through Watershed Planning and Floodplain Management.*

Objective 28
Preserve floodplains to reduce the risk of flooding under the "fully developed drainage basin for the 100 year flood" condition.

Objective 28: Policy 1
Utilize the floodplain and adjacent land to provide flood water conveyance and regional storm water detention.

Article 15, "Floodplain Management" of the UDC establishes floodplain regulations to promote the public health, safety, and welfare and to minimize public and private losses due to flood conditions in specific areas. In addition, the city has a Floodplain Administrator whose responsibilities include, but are not limited to:

- Reviewing building permit applications for sites located in, or adjacent to the regulatory floodplain;
- Monitoring permits required by local, state or federal agencies;
- Communicating with the Federal Emergency Management Agency; and
- Overseeing the city's Flood Insurance Rate Maps (FIRM).

The Parks and Recreation Department also contributes to floodplain preservation by acquiring floodplain to use for linear parks and open space. In addition, the following strategies must be considered for any new development along floodplains.

- Discourage development in the floodplain.
- Encourage dedication of property within the floodplain to the city through the platting process.
- Acquire floodplain when economically feasible and encourage retention of open space in new development.
- Use the "build out" floodplain map developed as part of the Comprehensive Plan to site all future development.
- Establish regional retention ponds, where practical, to manage increased runoff from upstream sources.

The next policy recognizes that there are existing residential buildings in floodplains, whose time of original construction predates current flood damage reduction policies. Some of these structures may be prone to repetitive losses due to flood damage.

<u>*Objective 28: Policy 2*</u>

Reduce the number of repetitive loss structures and flood damage to other existing residential buildings through Capital Improvement Projects (CIPs) that employ a combination of buyouts and structural improvements.

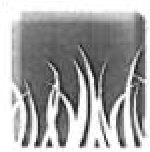
In addition, the following strategies are currently employed or considered to minimize losses due to flood damage.

- Adopt FEMA standards in the Floodplain Management Ordinance that prohibits designated construction in the floodplain.
- Ensure residents are given adequate warning of floods.
- Ensure real estate disclosure of flooding to all potential property purchasers.
- Make public information available on flooding problems and hazards throughout the city.
- Notify citizens that flood insurance is available.

A number of storm water controls, or Best Management Practices, may also be implemented in site design. Some of the most common methods are described here.

Site Planning

A basic five step process can be implemented including the review



of (1) Concept Plans, (2) Preliminary Plats, and (3) Final Plats. When the site plan in approved, it is then used as a guideline for (4) Construction Inspections and the ongoing (5) Operation and Maintenance. The basic Principles of Storm Water Management Planning are:

- *The site design should utilize an integrated approach to deal with storm water quality protection, streambank protection and flood control requirements.* (The basic site design should be done in unison with the layout of the storm water infrastructure to attain the storm water management goals.)
- *Storm water management practices should strive to utilize the natural drainage system and require as little maintenance as possible.* (Almost all sites contain natural features which can be used to help manage and mitigate runoff from development, such as depressions, permeable soils, wetlands, floodplains and undisturbed vegetated areas.)
- *Structural storm water controls should be implemented only after all site design and nonstructural options have been exhausted.* (The use of natural techniques offers significant benefits over structural storm water controls.)

A soccer field may serve as a temporary storage facility for storm water.

- *Structural storm water solutions should attempt to be multi-purpose and be aesthetically integrated into a site's design.* (A parking lot, soccer field, or city plaza can serve as a temporary storage facility for storm water. In addition, water features such as ponds and lakes, when correctly designed and integrated into a site, can increase the aesthetic value of a development.)
- *"One size does not fit all" in terms of storm water management solutions.* (Each site, project and watershed presents different challenges and opportunities.)

1. Site Design Practices and Technique benefit developer and the public by:

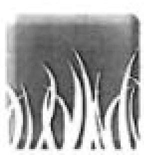
- Preserving the natural hydrology and drainage ways of a site
- Reducing the amount of impervious cover and associated runoff and pollutants generated
- Preserving a site's natural character and aesthetic features
- Preserving riparian ecosystems and habitats
- Reducing the need for grading and land disturbance
- Making efficient use of natural site features for preventing and mitigating storm water impact

2. Conservation of Natural Features and Resources:

a. *Preserve Undisturbed Natural Areas* – Preserving natural conservation areas such as undisturbed forested and vegetated areas, natural drainage ways, stream corridors and wetlands on a development site helps to preserve the original hydrology of the site and aids in reducing the generation of storm water runoff and pollutants. Undisturbed vegetated areas also stabilize soils, provide for filtering and infiltration, decreases evaporation, and increases transpiration.

b. *Preserve Riparian Buffers* – A riparian buffer is a special type of natural conservation area along a stream, wetland or shoreline where development is restricted or prohibited. The primary function of a buffer is to protect and physically separate a stream, lake or wetland from future disturbance or encroachment. If properly designed, a buffer can provide storm water management functions, can act as a right-of-way during floods, and can sustain the integrity of stream ecosystems and habitats, can be used as nonstructural storm water filtering and infiltration zones, and keeping structures out of the floodplain.

c. *Avoid Floodplains* – Floodplains are the low-lying lands that border streams and rivers. When a stream reaches its capacity and overflows its channel after storm events, the floodplain provides for storage and conveyance of these excess flows. In their natural state they reduce flood velocities and peak flow rates by the passage rate of flows through dense vegetation. Floodplains also play an important role in reducing sedimentation by filtering runoff, and provide habitat for both aquatic and terrestrial life. Development in floodplain areas can reduce the ability of the floodplain to convey storm water, potentially causing safety problems or significant damage to the site in question, as well as to both upstream and downstream properties. The regulation of the use of floodplain areas minimizes the risk to human life as well as helps to avoid flood damage to structures and property. As such, floodplain areas should be avoided on a development site. Ideally, the entire 100-year full -build out floodplain should be avoided for clearing or building activities, and should be preserved in a natural undisturbed state where possible. Floodplain protection is complementary to riparian

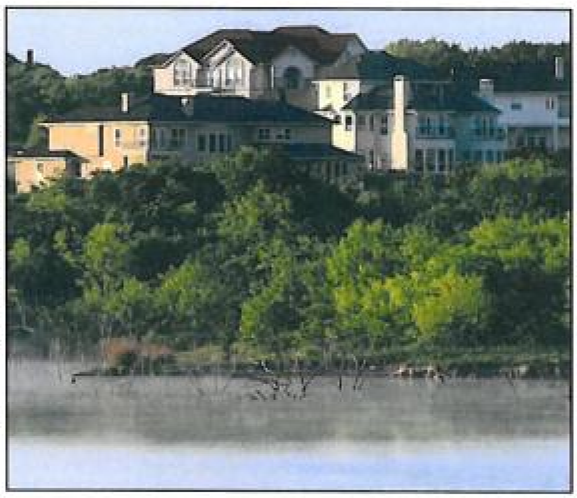


parian buffer preservation, and can be combined with riparian buffer protection to create linear greenways. Depending on the site topography, 100-year floodplain boundaries may lie inside or outside of a preserved riparian buffer corridor.

d. *Avoid Steep Slopes* – Developing on steep slope areas has the potential to cause excessive soil erosion and increased storm water runoff during and after construction. Past studies by the SCS (now NRCS) and others have shown that soil erosion is significantly increased on slopes of 15% or greater. In addition, the nature of steep slopes means that greater areas of soil and land area are disturbed to locate facilities on them compared to flatter slopes. On slopes greater than 25%, no development, regarding, or stripping of vegetation should be considered unless the disturbance is for roadway crossings or utility construction and it can be demonstrated that the roadway or utility improvements are absolutely necessary in the sloped area. Building on flatter areas will reduce the need for cut-and-fill and grading.

3. Lower Impact Site Design Techniques

a. *Fit Design to the Terrain* – All site layouts should be designed to conform with or "fit" the natural landforms and topography of a site. This helps to preserve the natural hydrology and drainage ways on the site, as well as reduces the need for grading and disturbance of vegetation and soils. Roadway patterns on a site should be chosen to provide access schemes which match the terrain. In rolling or hilly terrain, streets should be designed to follow natural contours to reduce clearing and grading. Street hierarchies with local streets branching from collectors in short loops and cul-de-sacs along ridgelines help to prevent the crossing of streams and drainage ways. In flatter areas, a traditional grid pattern of streets or "fluid" grids which bend and may be interrupted by natural drainage ways may be more

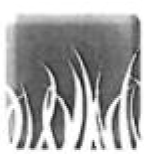


Homes along Joe Pool Lake are clustered together and set apart from steep slopes.

appropriate. A grid pattern may also allow for narrower streets and less imperviousness as having more than one route for emergency vehicles makes it easier to relax minimum street width requirements. In either case, buildings and impervious surfaces should be kept off steep slopes, away from natural drainage ways, and out of floodplains and other lower lying areas. In addition, the major axis of building should be oriented parallel to existing contours.

b. *Locate Development in Less Sensitive Areas* – A site layout should also be designed so the areas of development are placed in the locations of the site that minimize the hydrologic impact for the project. This is accomplished by steering development to areas of the site that are less sensitive to land disturbance or have a lower value in terms of hydrologic function using the following methods: Locate buildings and impervious surfaces away from stream corridors, wetlands and natural drainage ways. Use buffers to preserve and protect riparian areas and corridors. Avoid land disturbing activities or construction on areas with steep slopes or unstable soils. Minimize the clearing of areas with dense tree canopy or thick vegetation, and ideally preserve them as natural conservation areas. Ensure natural drainage ways and flow paths are preserved, where possible.

c. *Reduce Limits of Clearing and Grading* – Minimal disturbance methods should be used to limit the amount of clearing and grading that takes place on a development site, preserving more of the undisturbed vegetation and natural hydrology of a site. These methods include:



1. Establishing a limit of disturbance (LOD) based on maximum disturbance zone radii/lengths. These maximum distances should reflect reasonable construction techniques and equipment needs together with the physical situation of the development site such as slopes or soils. LOD distances may vary by type of development, size of lot or site, and by the specific development feature involved.
2. Using site "foot printing" which maps all of the limits of disturbance to identify the smallest possible land area on a site which requires clearing or land disturbance.
3. Fitting the site design to the terrain
4. Using special procedures and equipment which reduce land disturbance

d. *Utilize Open Space Development* – Open space development, also known as conservation development or clustering, is a site design technique that concentrates structures and impervious surfaces in a compact area in one portion of the development site in exchange for providing open space and natural areas elsewhere on the site. Typically, smaller lots and/or nontraditional lot designs are used to cluster development and create more conservation areas on the site.

1. Can be used to help protect natural conservation areas and other site features
2. Reduces infrastructure needs and overall development costs
3. Consider Creative Designs – A Planned Development (PD) can be used to implement many of the other site design practices to create site designs that maximize natural nonstructural approaches to storm water management. This approach may be useful for implementing an open space development.

Prairie Paws pet adoption center uses landscaped storm water management "islands."

4. Reduction of Impervious Cover

a. *Reduce Building Footprints* – In order to reduce the imperviousness associated with the footprint and rooftops of buildings and other structures, alternative and/or vertical (taller) building designs should be considered. Consolidate functions and buildings, as required, or segment facilities to reduce the footprint of individual structures.

b. *Reduce the Parking Footprint* – Reduce the overall imperviousness associated with parking lots by providing compact car spaces, minimizing stall dimensions, incorporating efficient parking lanes, parking decks, and using porous paver surfaces or porous concrete in overflow parking areas where feasible and where soils allow for infiltration.

c. Create Parking Lot Storm Water "Islands" – Parking lots should be designed with landscaped storm water management "islands" which reduce the overall impervious cover of the lot as well as provide for runoff treatment and control in storm water facilities.

When possible, expanses of parking should be broken up with landscaped islands which include shade trees and shrubs. Fewer large islands will sustain healthy trees better than more numerous very small islands. The most effective solutions in designing for tree roots in parking lots is to use a long planting strip, constructed with sub-surface and compaction resistant soil. Structural control facilities such as filter strips, dry swales and bioretention areas can be incorporated into parking lot islands. These facilities can be attractively integrated into landscaped areas and can be maintained by commercial landscaping firms.

5. Utilization of Natural Features for Storm Water Management

a. *Use Buffers and Undisturbed Areas* – Runoff can be directed towards riparian buffers and other undisturbed natural areas delineated in the initial stages of site planning to infiltrate runoff, reduce runoff velocity and remove pollutants. Natural depressions can be used to temporarily store (detain) and infiltrate water, particularly in areas with permeable (hydrologic soil group A and B) soils.

b. Use Natural Drainage ways Instead of Storm Sewers – The use of natural open channels allows for more storage of storm water flows on-site, lower storm water peak flows, a reduction in erosive runoff velocities, infiltration of a portion of the runoff volume, and the capture and treatment of storm water pollutants. It is critical that natural drainage ways be protected from higher post-development flows by applying downstream stream bank protection methods to prevent erosion and degradation.

1. Use of natural drainage ways reduces the cost of constructing storm sewers or other conveyances, and may reduce the need for land disturbance and grading

2. Natural drainage paths are less hydraulically efficient than man-made conveyances, resulting in longer travel times and lower peak discharges

c. *Use Vegetated Swale Instead of Curb and Gutter* – Open vegetated channels along a roadway remove pollutants by allowing infiltration and filtering to occur, unlike curb and gutter systems which move water with virtually no storm water treatment. Older roadside ditches which have not been maintained suffer from erosion, standing water and break up of the road edge. Grass channels and enhanced dry swales are two alternatives when properly installed and maintained under the right site conditions, are excellent methods for treating storm water on-site. In addition, open vegetated channels can be less expensive to install than curb and gutter systems.

d. *Drain Rooftop Runoff to Pervious Areas* – Storm water quantity and quality benefits can be achieved by routing the runoff from impervious areas to pervious areas such as lawns, landscaping, filter strips and vegetated channels. Revegetated areas such as lawns and engineered filter strips and vegetated channels can act as biofilters for storm water runoff and provide for infiltration in pervious soils. In this way, the runoff is "disconnected" from a hydraulically efficient structural conveyance such as a curb and gutter or storm drain system.

The city requires Concept Plans to address these site design strategies. The next objective states that they city will continue the objective to require Concept Plans, as well as the accompanying policies and strategies. This objective meets *Goal 7: Maintain and Improve Drainage in the City Through Watershed Planning and Floodplain Management.*

Objective 29

The City will continue to require Concept Plans (prior to zoning and/or any development activity) for the purpose of determining the pattern of land development and urban design.

Objective 29: Policy 3

Develop incentives to reduce the need for grading and land disturbance, providing cost savings for developers.

Objective 29: Policy 4

Site layouts should be designed to conform with, or "fit" natural site topography/landforms.

Objective 29: Policy 5

Natural drainage ways, streams and wetlands will be used, where possible, to manage storm water runoff.

The following strategies reinforce the polices for Objective 29.

- Design site layouts to conform with or "fit"natural site topography/landforms.
- Utilize natural drainage ways, streams and wetlands to manage storm water runoff where possible.
- Utilize average density (i.e. units per acre) in defining residential zoning district standards in order to encourage preservation of open space, the creation of parks and to encourage a variety of housing types.
- A minimum of 15% common or public open space should be provided in all residential developments involving more than 20 units or 10 acres.

Water Quality

The City of Grand Prairie administers local, state and federal regulations affecting area businesses and the quality of the wastewater discharged into the city's sanitary sewer. The city sends its domestic and commercial wastewaters to the Trinity River Authority for treatment before being released into the

Curbside Recycling is offered to Grand Prairie residents.

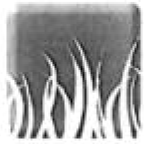
Trinity River. In 1977, the Clean Water Act established guidelines for improving the water quality of the nation's waterways through regulation of industrial wastewater. These regulations were implemented to protect the wastewater treatment process and workers, allow beneficial re-use of bio-solids, allow reclamation of treated effluent for other purposes, and maintain exceptional water quality in the receiving streams. A permit to discharge industrial wastewater to the sanitary sewer is required for certain types of industrial activities. These permits are locally regulated by the Environmental Quality Division of Environmental Services.

There are some sites that have been moderately to heavily impacted by manmade processes. Some of these sites may have been exposed to chemicals, like the chemicals used for dry cleaning; and others have been adapted with structures that make it difficult to convert the site to another use, like the underground storage tanks that are used for gasoline stations.

The Environmental Quality Division also monitors sites that are established as municipal setting designations, and implements grants for brownfield sites. The following objective relates to monitoring and restoring environmentally sensitive sites. It also meets *Goal 2: Encourage Resource Conservation and Renewable Energy.*

Objective 30

Environmentally sensitive sites – areas that have been impacted by manmade processes – need to be restored and monitored.



Objective 30: Policy 6

Pursue opportunities to remediate and redevelop brownfield sites and other developed lands that suffer from environmental constraints.

Objective 30: Policy 7

Monitor sites that are established as municipal setting designations to prevent the creation of new water wells, and to allow appropriate redevelopment.

Solid Waste and Recycling is also a division of the Environmental Services Department. This division implements programs associated with landfill operations, recycling and many of the trash cleanups throughout the year. The following objective addresses the need to reduce, reuse and recycle waste; and properly dispose of hazardous materials. It also meets *Goal 2: Encourage Resource Conservation and Renewable Energy.*

Objective 31

Develop responsible alternatives to landfilling of solid waste.

Objective 31: Policy 8

Continue educational efforts to reduce levels of consumption and waste generation at the household and community levels.

Objective 31: Policy 9

Continue educational efforts to educate the public about both short- and long-term risks associated with the use and improper disposal of hazardous materials.

For more information on the Household Hazardous Waste collection event, see Section 2, Baseline Analysis of this plan.

Another way to develop responsible alternatives to landfilling of solid waste is to reuse building materials and recycle building construction waste materials. A good example of recycling is to use demolished street pavement materials as a base layer for new parking. It is possible to get LEED credit for these types of practices, and the city does encourage and recognize both private and public buildings that receive LEED certification.

<u>*Objective 31: Policy 10*</u>
Encourage the use of recycled building materials and recycled building construction waste materials.

Household Hazardous Waste collection event.

The next objective relates to encouraging resource conservation by reducing water consumption. It also meets *Goal 2: Encourage Resource Conservation and Renewable Energy.*

<u>*Objective 32*</u>
Reduce water consumption and improve water quality.

<u>*Objective 32: Policy 11*</u>
Continue to encourage and promote "Texas Smartscape" strategies for landscaping standards.

The city currently provides a list of recommended drought-tolerant, native species for Landscape Plans for commercial development. When these species are utilized, staff notifies the Planning and Zoning Commission that a proposed project is using species from the recommended list.

<u>*Objective 32: Policy 12*</u>
Develop landscaping options that use less water, such as the use of native plants and drip irrigation for public facilities, and advertising excellent examples of these principles for the private sector to mirror.

Article 8, "Landscape and Screening" of the Unified Development Code was recently revised to provide more stringent requirements for irrigation in compliance with state requirements.

Storm Water Management Program

The next few pages describe in some detail, the city's participation in the Clean Water Act relative to Storm Water Management.

Clean Water Act

Phase I of the US Environmental Protection Agency's (EPA's) storm water program was promulgated in 1990 under the Clean Water Act. Phase I relies on the National Pollutant Discharge Elimination System (NPDES) permit coverage to address storm water runoff from municipalities serving populations of 100,000 and greater, construction activity disturbing 5 acres of land or greater, and ten categories of industrial activity.

On September 14, 1998, control over storm water permitting shifted from the federal EPA NPDES to the Texas Commission on Environmental Quality (TCEQ), called the Texas Pollutant Discharge Elimination System (TPDES). The TPDES permit includes construction activity disturbing one acre or more and Phase II of the EPA program requires Grand Prairie to secure a permit under the TPDES as an MS4 city. Grand Prairie received it MS4 permit on August 13, 2007 under the Storm Water Management Program (SWMP).

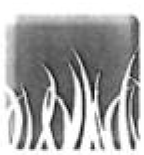
Storm Water Phase II Rule

The Storm Water Phase II rule, promulgated December 8, 1999, was the next step in the EPA's efforts to preserve, protect, and improve the nation's water resources from polluted storm water runoff. The Phase II program requires small MS4s (serving populations <100,000 based on the 1990 census) in urbanized areas to implement programs and practices to control polluted storm water runoff through the TPDES permit program. This program includes the City of Grand Prairie and the Dallas County Flood Control District #1 (DCFCD). As a result, the City is required to:

- reduce the discharge of pollutants to the maximum extent practicable (MEP);
- protect water quality;
- satisfy the appropriate water quality requirements of the Clean Water Act; and
- manage storm water quality activities through the Storm Water Management Program (SWMP).

Storm Water Management Program (SWMP)

On August 13, 2007 the TCEQ issued the MS4 TPDES General Permit TXR040000 authorizing storm water and certain non-storm water discharges to the City's MS4. Small MS4s that meet the regulated criteria for Phase II of the TPDES Storm Water Program were required to submit a Notice of Intent (NOI) and Storm Water Management Program (SWMP) within 180 days of the permit issuance. By submitting a SWMP and NOI to comply with the TPDES Phase II regulations, the City of Grand Prairie and DCFCD acknowledge the regulatory authority of the TCEQ and agrees to comply with TPDES TXR040000 permitting requirements to discharge directly into surface waters. This permit and authorization shall expire five years after the date of issuance. An annual report documenting compliance with the SWMP will be submitted within 90 days of the end of each permit year (August 13, 2007 anniversary date) or by November 13.



The City of Grand Prairie and DCFCD developed the SWMP in accordance with the requirements of the TPDES General Permit TXR040000. The SWMP will facilitate the City's and DCFCD's efforts in reducing storm water pollutants from the City's MS4, thereby protecting the City's storm water quality to the maximum extent practicable (MEP). Included in the SWMP are specific best management practices (BMPs) that will be implemented to reduce pollutants, measurable goals for each BMP, and an implementation schedule developed for the five-year permit term. Various BMPs were developed for each of the six minimum control measures (MCMs) that are required by the Phase II Rule. These six MCMs are:

- Public Education and Outreach on Storm Water Impacts;
- Public Participation and Involvement;
- Illicit Discharge Detection and Elimination;
- Construction Site Runoff Control;
- Post-Construction Runoff Control; and
- Pollution Prevention and Good Housekeeping.

The SWMP's policies and strategies meet *Goal 2: Encourage Resource Conservation and Renewable Energy* and Objective 32.

Objective 32

Reduce water consumption and improve water quality.

Objective 32: Policy 13

Utilize Storm Water Management to improve the quality of storm water runoff.

Program Overview

There are approximately 170 stream miles in Grand Prairie draining to three major water bodies: the West Fork of the Trinity River, Joe Pool Lake, and Mountain Creek Lake. The West Fork of the Trinity River runs across the city from west to east on the northern part of town, dominating drainage patterns to the Trinity River. The majority of creeks run northeast on the south side of the Trinity River and southeast on the north side of the Trinity River. Major creeks that drain directly to the Trinity River within city limits are Dalworth Creek, Johnson Creek, and Bear Creek. Major creeks draining to Joe Pool Lake and Mountain Creek Lake are Mountain Creek, Fish Creek, and Cottonwood Creek.

Joe Pool Lake is the focus of recreation in southern Grand Prairie. Much of the development and community activities focus on the recreational aspects of Joe Pool Lake. This lake was impounded in 1986 and has two forks created by Mountain Creek and Walnut Creek. The shorelines of the western main body, the entire Walnut Creek branch, as well as the western shoreline of the Mountain Creek branch are within city limits.

Mountain Creek Lake, impounded in 1937, is on the east side of the city. The drainage is dominated by Mountain Creek, after the Joe Pool Lake dam. The lake is within Dallas city limits; however, some tributaries originate in Grand Prairie, including Fish Creek and Cottonwood Creek. A fishing ban was issued for this lake in 1996 by the Texas Department of State Health Services for poly-chlorinated biphenyls, a group of dangerously harmful organic compounds once widely used in industrial activities.

Historical City Storm Water Management

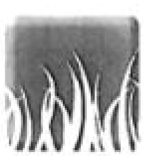
The Engineering Division of the Planning and Development Department oversees and inspects the infrastructure construction of new development and redevelopment. The Engineering Division ensures the effectiveness of erosion control measures during development and redevelopment through permitting. The Engineering Division also encourages the preservation of natural channels and requires drainage easements and control measures in the 100-year floodplain.

The Environmental Services Department was created and developed to support and protect public health and promote environmental quality. The Environmental Quality Division was created in 1984 to support the pretreatment program and address other water quality issues primarily through an inspection program, monitoring, and citizen involvement. Problematic areas pertaining to storm water have been identified and addressed in the past through the storm water program. Some of these issues have included salvage yards, sanitary sewer overflows, household hazardous waste disposal, and hazardous material spills. These issues have been addressed through enforcement when necessary.

A stream monitoring program began in 1986 as the interest in the condition of the waters within city limits increased. The City currently samples at 22 sites in and near city limits once a month. The monitoring includes water quality indicators such as temperature, clarity, and chemistry. Quarterly and annually, the water is tested for potentially harmful chemicals such as nutrients and pesticides. This information has been used to identify sources of pollution and reduce illicit discharges. To identify problematic water quality issues and potential illicit discharges, the City has also taken advantage of sampling done by the Trinity River Authority in Joe Pool Lake.

Management Program Development Process

The unique hydrology and water quality concerns of the City of Grand Prairie have been considered in developing this Storm Water Management Program. In preparing the Program, the City of Grand Prairie's Environmental Quality Division has conducted meetings with a multitude of city personnel to discuss the different activities that may have storm water impacts. Some of the functions that have been identified as having a potential impact have included streets services, equipment maintenance services, landfill, airport, code enforcement, police, fire, parks and recreation, engineering, and building inspections. In addition, the Planning and Development Department utilized the consulting firm Alan Plummer and Associates, Inc. to help in the preparation of the Program regarding construction and post-construction.



The Program describes a number of Best Management Practices (BMPs) that address storm water issues identified as most prevalent or problematic in the watersheds served by the MS4. The BMPs meet a number of objectives created by the aforementioned departments. These objectives, organized by minimum control measure, are to:

Prairie Lakes Golf Course.

Public Education:

- Inform residents, visitors, public service employees, businesses, commercial and industrial facilities, and construction site personnel of steps they can take to improve storm water quality and explain the impacts of non-point source pollution to storm water.
- Educate commercial, industrial, and institutional groups about the impacts of their work on the storm water quality and the steps needed to reduce these effects.
- Address the viewpoints of various economic and cultural groups in the design of the education program.

Public Involvement:

- Comply with any State and local public notice requirements when implementing a public involvement/participation program.
- Include the public in the development, implementation, and review of the storm water management program.
- Include input from different economic and cultural groups.

Illicit Discharge Detection and Elimination:

- Develop a comprehensive map of the storm sewer system.
- Develop a program for the detection and tracking of illicit discharges.
- Develop an ordinance that will effectively eliminate illicit discharges.

Construction:

- Have an ordinance or other regulatory mechanism requiring the implementation of proper erosion and sediment controls, and controls for other wastes, on applicable construction sites.

- Have procedures for site plan review of construction plans that consider potential water quality impacts.
- Have procedures for site inspection and enforcement of control measures.
- Have sanctions to ensure compliance (established in the ordinance or other regulatory mechanisms).
- Establish procedures for the receipt and consideration of information submitted by the public.

Post-Construction:

- Develop and implement strategies which include a combination of structural and/or non-structural BMPs.
- Have an ordinance or other regulatory mechanism requiring the implementation of post-construction runoff controls to the extent allowable under State, Tribal, or local laws.
- Ensure adequate long-term operation and maintenance of controls.

Pollution Prevention/Good Housekeeping for Municipal Operations:

- Review maintenance activities.
- Review maintenance schedules.
- Long-term inspection procedures for structural and non-structural storm water controls to reduce floatables and other pollutants discharged from the separate storm sewer.
- Controls for reducing or eliminating the discharge of pollutants from streets, roads, highways, municipal parking lots, maintenance and storage yards, fleet or maintenance shops with outdoor storage areas, salt/sand storage locations, disposal areas, and waste transfer stations.
- Procedures for properly disposing waste removed from the separate storm sewers and areas listed above (such as accumulated sediments, floatables, and other debris).

Public Review of the SWMP

In accordance with the General Permit TXR040000, Part II, Section D, Number 12, the SWMP will be available for review at the Grand Prairie Memorial Library Repository, located at 901 Conover Drive, Grand Prairie, Texas 75051, and is also available on the City website at www.gptx.org.

Permitting Options

The City of Grand Prairie and Dallas County Flood Control District #1 (DCFCD) are jointly submitting this Storm Water Management Program as described in an interlocal agreement approved by the aforementioned entities on February 5, 2008. According to Part III of the General Permit, a permittee may enter into interlocal agreements with municipalities where the small MS4 is located in order to meet the goals of the permit if the permittee does not have enforcement authority and is unable to meet the goals of the general permit through its own powers. Approximately 20% of the DCFCD is located within the City of Grand Prairie boundaries;

however, the DCFCD does not have enforcement capabilities. As a result, the City of Grand Prairie and DCFCD have agreed to the joint submission of this SWMP where the DCFCD is solely responsible for only two (2) BMPs (BMP 6.10 and 6.11). The City of Grand Prairie is entirely responsible for all other BMPs described in this SWMP. On October 1st following the end of each permit year, the DCFCD will provide detailed information to the City of Grand Prairie on activities that occur within the DCFCD and City of Grand Prairie boundaries so that the City of Grand Prairie may complete its annual report for the TCEQ. *[Excerpt from Storm Water Management Program]*

Water Conservation Plan

The City of Grand Prairie adopted the Water Conservation Plan by resolution on April 7, 2009, as well as an ordinance providing for enforcement of certain mandatory provisions of the Water Conservation Plan. The Texas Commission on Environmental Quality (TCEQ), Texas Water Development Board approved the plan on May 7, 2009. The Water Conservation Plan's policies and strategies meet *Goal 2: Encourage Resource Conservation and Renewable Energy* and Objective 32.

<u>*Objective 32*</u>
Reduce water consumption and improve water quality.

<u>*Objective 32: Policy 14*</u>
The city will continue implementing regional water conservation initiatives, such as the prohibition on irrigating between 10:00 a.m. and 6:00 p.m.

Energy Conservation

The next objective recognizes that renewable energy is a proactive approach to conserving our resources. It also meets *Goal 2: Encourage Resource Conservation and Renewable Energy.*

<u>*Objective 33*</u>
Incorporate regional energy efficiencies into residential and nonresidential construction.

<u>*Objective 33: Policy 15*</u>
Develop incentives for new residential con-

Wind Turbines on a Warehouse (AeroVironment rooftop array).

struction to reduce energy consumption. Encourage the use of regenerative heating and cooling source alternatives to fossil fuels, such as wind or solar powered systems.

Small Wind Energy Systems

In 2007, the Planning Division of the Planning and Development Department created an ordinance addressing Small Wind Energy Systems under Article 9, Section 4, of the Unified Development Code. Under the current ordinance, small wind energy systems require a Specific Use Permit (SUP) in all zoning districts and floodplain areas and should contain a minimum lot size of two (2) acres. The requirement for a minimum lot size is due to the need to maintain a wind corridor for the wind turbine. The ordinance includes provisions for freestanding tower systems and rooftop mounted wind turbines.

When staff made contact with suppliers of these systems to determine the feasibility of this renewable energy alternative, suppliers stated that the D/FW area is not currently considered a significant market for private wind turbine power - primarily because of the amount of local urbanization, and the lack of significant annual winds to justify year-around operation and expense. However, the ordinance does allow residents and businesses the opportunity to investigate this option.

In an urban environment, the close proximity to existing structures, the resulting air turbulence and noise impact make height a critical consideration for residential-scale wind energy systems. Even commercial sites face challenges when trying to justify the initial cost outlay with the return on investment, when locating wind turbines in a weak wind corridor. However, as wind turbine technology improves in the future, such systems may become more prevalent in the area.

Solar Energy

The Unified Development Code does not prohibit or restrict solar energy systems. When considering this renewable energy option, many homeowners have found that private deed restrictions exclude this alternative for many residential neighborhoods.

There are residences and businesses in Grand Prairie that utilize solar panels and other solar energy technologies. When considering this renewable energy alternative, staff recommends that the owner of the property consider the building envelope for the zoning districts of adjacent properties, and the potential full-growth height for adjacent trees to protect long-term solar access.

National Green Building Standard

Since the International Code Council (ICC) released its Green Building Policy Position Statement in late 2006, it has taken many steps on the green front. In 2008, the National Green Building Standards was created for residential development, and many cities have taken steps to include green building standards in codes and ordinances. There

Fishing dock at Joe Pool Lake.

other rating systems, such as the U.S. Green Building Council's Leadership in Energy and Environmental Design (LEED) certification; however, the ICC standards is the only ANSI (American National Standards Institute) recognized standard.

It is often thought that green building is costly. This is not necessarily true. As green building becomes more mainstream, the initial cost differential between conventional and green building construction continues to blur. Some technologies, such as certain active solar systems, are cost prohibitive, but costs continue to come down. And the implementation of many of the conservation principles mean cost savings overall.

Many large-scale residential developers are implementing green building standards, and a few private nonresidential builders; however, the city will continue to encourage these standards for both sectors.

Objective 33: Policy 16
Continue to encourage and promote nonresidential green building standards such as energy efficient cool roofs, distribution transformers, variable speed control VAV and exterior lighting.

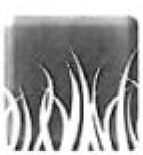
Objective 33: Policy 17
Allow building materials with low "embodied energy," which requires less energy-intensive production methods and long-distance transport.

Following are some cost-effective strategies for residential energy efficiency.

- Maintain heating and cooling systems by replacing heat pump filters and having annual checks,
- Seal air leaks by caulking and weather stripping doors and windows,
- Install insulated windows or use storm windows in the winter,
- Seal attic vents and ducts, and check for adequate insulation,
- Caulk and weather proof exterior openings for plumbing and electric service,
- Install a programmable thermostat, and
- Replace traditional light bulbs and fixtures with compact fluorescents.

Air quality will be addressed in more detail in Section 10, Intergovernmental Cooperation.

Appendix E
Floodplain Management



Floodplain Management

The Comprehensive Plan incorporates policies of the City's Drainage and Flood Plain Management Plans with the long-term goals of the City. The Appendix provides an overview of the Drainage and Floodplain Management Plans. Based on the Federal Emergency Management Agency (FEMA) Historical Claims, the City of Grand Prairie had 154 structures with 324 losses for a total of $6,649,566 in flood insurance claims paid from 1978 to 2010. The City of Grand Prairie had 304 Flood Insurance Policies in force in August 2010, with $72,255,100 in coverage.

A. History of Flooding

Historical documentation of flooding in Grand Prairie is incomplete but indicates substantial flooding has taken place as shown below. The City has over 19,000 acres, or 36.7% of its land area, as floodplain within the City limits. This includes land owned and managed by the Corps of Engineers and the majority of Joe Pool Lake.

Known Flooding in Grand Prairie

May, 1908	-Five feet above flood of May 1957 on West Fork of Trinity River
1922	-Major flood on Mountain Creek
1928	-Major flood on Mountain Creek
April, 1942	-no comments
March, 1945	-no comments
May, 1949	-Discharge of 62,000 cfs (affected by major levee breaks)
May, 1957	-Discharge of 59,200 cfs at the City of Grand Prairie gauge
1965	-Large floods on Johnson Creek and Cottonwood Creek
1969	-Major flood on Mountain Creek
1976	-Major flood on Mountain Creek
March, 1977	-Flood on Johnson Creek above 70 homes
1979	-Large floods on Johnson Creek and Cottonwood Creek
May, 1989	-Three people drowned where creeks flooded roadways
June, 1989	-Severe flooding over Carrier Parkway, Beltline Road and Matthew Road
May, 1990	-Thirty-six homes flooded
Dec., 1991	-Eighteen homes flooded
May, 1995	-Dalworth Creek flooded five homes

From 1990 to 2010, the City funded $41.2 million in capital drainage improvements and $2.5 million in maintenance of storm drains. These projects reduced the risk of flooding for more than 300 of the 650 flood-prone structures located in Grand Prairie.

Many flood prone structures have been removed from the floodplain by projects such as the Dorchester Levees, Johnson Creek Channelization and the Dry Branch Channel improvements.

The City's storm water utility fee, established in October 1993, generates approximately $1.9 million annually to be spent on capital improvement projects for the reduction of residential and non-residential flooding, for erosion mitigation, and for miscellaneous drainage projects. Investigation has shown that some flooding occurs due to poor site drainage, improper lot grading by home builders in new subdivisions, and erosion.

B. Goals

The impact of flooding within Grand Prairie has been reduced considerably due to extensive improvements, drainage projects, removal of structures from flood risk and the purchase of repetitive loss structures. To meet federal and state requirements the City maintains an active storm water management plan as a component of its Floodplain Management Plan. Federal floodplain management goals are given below.

Federal (FEMA) Goals

I. Protect human life and health.

II. Minimize expenditure of public money for costly flood control projects.

III. Minimize the need for rescue and relief efforts associated with flooding that are generally undertaken by the City at the expense of the general public.

IV. Minimize prolonged business interruptions.

V. Minimize damage to public facilities and utilities such as water and gas mains, electric, telephone and sewer lines, streets and bridges located in floodplains.

VI. Help maintain a stable tax base by providing for the sound use and development of flood-prone areas in such a manner as to minimize future flood blight areas.

VII. Help potential buyers become aware of property that is subject to flooding.

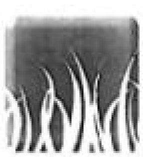
The City of Grand Prairie has adopted more restrictive measures beyond the NFIP minimum regulations. The City requires that the lowest floor of the structure be elevated to the higher of not less than one (1) foot above the base flood elevation (taking into account the effects of future full development) or two feet above the FEMA base flood elevation. Additional City objectives are shown below.

- Reduce the number of repetitive loss structures and flood damage to other existing residential buildings through Capital Improvement Projects (CIPs) that employ a combination of buyouts and structural improvements.
- Reduce the impact of increased flood flows from development in existing downstream buildings and streambed erosion.
- Acquire floodplain when economically feasible and encourage open space in developments.

- Creek studies/master plans.
- Create GIS Planning Tool for City and ETJ that includes:

  - Drainage Problems
  - Erosion Problems
  - Open Space
  - Special Flood Hazard Area
  - Undeveloped Land
  - Water Quality Monitoring Policy

- Keep City owned floodplain in natural state to ensure water quality and conserve existing flora and fauna.
- Encourage the reduction of runoff through better design.
- Add certain trees to floodplain where practical.
- Notify citizens that flood insurance is available.
- Increase the flood policy base (outreach program to insurance companies).
- Review the Floodplain Management Ordinance – Improve, revise, etc.
- Ensure that residents are given adequate warning of floods.
- Ensure real estate disclosure of flooding to all potential property owners.
- Make public information available on flooding problems and hazards throughout the city.
- Extensive study of the city's existing and "built-out" floodplain comprised a significant portion of the Comprehensive Plan Update. Updated mapping, depicting the built-out flood plain, will now be used for regulating future development.



- The City-Wide Drainage Master Plan Road Map establishes the processes for future flood control planning for the City of Grand Prairie. The city's primary goal and objective of the City-Wide Drainage Master Plan is to cost-effectively manage flood or storm waters within budgeting constraints so that conditions don't get worse as new and infill areas are developed—while evaluating and making conditions better in the areas of the city that are already developed.

C. Impacts of Development

Balancing issues of public health and safety, environmental sustainability, economic impact, legal liability, regulatory responsibility and improved quality of life, with new development requires careful analysis and mitigation of development impacts. This includes measures to reduce the concentration and types of pollutants carried by surface water runoff, establishment of detention ponds to capture storm water runoff, use of previous materials where possible, and adoption of best operating practices in engineering design and construction management.

D. Storm Water Management

In 1990, the Environmental Protection Agency (EPA) published the first set of requirements relating to storm water and its discharge ("Phase I") which pertained to cities larger than 100,000 in population and for certain industrial activities such as airports, landfills and construction operations.

In 1992, EPA designated that under 'Phase II" of the storm water regulations, storm water discharges from construction sites disturbing more than one acres would also be regulated to protect water quality.

Storm water management involves both the prevention and mitigation of storm water runoff quality and quality using a variety of methods and mechanisms. The following components are included:

- Information/System Inventory
- Development Requirements
- Storm Water System Improvements
- Operations and Maintenance
- Monitoring
- Pollution Prevention
- Public Education/ Involvement
- Funding
- Watershed Planning
- Floodplain Management

The City of Grand Prairie's Drainage and Floodplain Management programs exceed federal and state requirements and are continually being reviewed and updated to provide safety and quality of life for city residents. A number of storm water controls, or Best Management Practices, can be used in site design. Some of these are listed below.

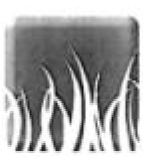
- Bioretention Areas
- Chemical Treatment
- Filtration
- Porous Surfaces
- Re-Use
- Conveyance Components (pipe systems, culverts, inlets)
- Grass Channels
- Dry Detention
- Wet Ponds
- Proprietary Structural Controls
- Wetlands

1. On-Site vs. Regional Storm Water Controls

Using individual, on-site structural storm water controls for each development is the typical approach in most communities for controlling storm water quantity and quality. The developer finances the design and construction of these controls and, initially, is responsible for all operation and maintenance. However, the local government is likely to become responsible for maintenance activities is the owner fails to comply.

A potential alternative approach is to install a few strategically located regional storm water controls in the sub watershed rather than require on-site controls. Regional storm water controls are facilities designed to manage storm water runoff from multiple projects and/or properties through a local jurisdiction-sponsored program, where the individual properties may assist in the financing of the facility, and the requirement for on-site controls is either eliminated or reduced.

On the following page are summarized some of the "pros" and "cons" of regional storm water controls.



Advantages of Regional Storm Water Controls

- Reduced Construction Costs
- Reduced Operation and Maintenance Costs
- Higher Assurance of Maintenance
- Maximum Utilization of Developable Land
- Retrofit

Disadvantages of Regional Storm Water Controls

- Location and Siting
- Capital Costs
- Maintenance
- Need for Parking

For in-stream regional facilities:

- Water Quality and Channel Protection – Without on-site water quality and channel protection, regional controls do not protect smaller streams upstream from the facility from degradation and stream bank erosion.
- Ponding impacts – Upstream inundation from a regional facility impoundment can eliminate floodplains, wetlands and other habitat.

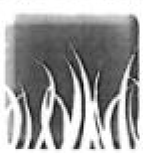
When a regional storm water control is implemented, it must be designed to handle peak flows and volumes without causing adverse impact or property damage. Full build out conditions in the regional facility drainage area should be used in the analysis.

Federal water quality provisions do not allow the degradation of water bodies from untreated storm water discharges. The EPA, TCEQ and the US Army Corps of Engineers have expressed opposition to in-stream regional hard armor (concrete) in channels. Concrete lined channels should be avoided if possible and will likely be permitted on a case-by-case basis only. It is important to note that siting and designing regional facilities should ideally be done within a context of a storm water master planning or watershed planning to be effective.

Regional Storm Water Controls

As per the City-Wide Drainage Master Plan Road Map, regional detention will be explored on a watershed by watershed basis. Potential regional detention sites will be identified during each of the watershed drainage master plans. The regional detention projects will be assigned a ranking calculation for implementation prioritization as set forth by the Road Map.

Developing Short-Term Priorities and a Long-Term Implementation Plan is critical to ensure that new floodplain improvements and storm water facilities are constructed over time to provide the most benefit to the city and community. For the City-Wide Drainage Master Plan, multiple improvement projects will be recommended for each individual watershed master plan. An overall, City-Wide implementation plan has to be developed to prioritize these projects into short-term and long-term priorities.



Two regional detention projects that were identified previously have been constructed. Site 1 (RC1), Kirby Creek at the Grand Prairie Airport was completed in 2010. Site 2 (RC2), Kirby Creek West of Robinson Road was completed in 2008.

Site 1 (RC1)

- Located on Kirby Creek at the Grand Prairie Municipal Airport
- Dry detention pond with approximately 30 acre-feet of storage capacity.
- Provides water quality protection for Grand Prairie Municipal Airport and a large paved area to the west of the airport

Site 2 (RC2)

- Located on Kirby Creek approximately 1,000 feet west of Robinson Road
- Wet pond with approximately 50 acre-feet of storage capacity

E. Flood Plain Management Ordinance

A primary component of the 2005 Comprehensive Plan Update is the City's new Flood Plain Management Ordinance which includes three major initiatives:

1. Addition of run-off coefficients tied to zoning to facilitate the calculation of drainage impact fee assessments;
2. Requirement of an earthwork permit for general construction;
3. Prohibition on locating mobile homes in the flood plain.

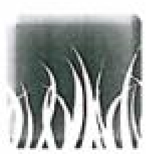
For a detailed review of the City's drainage and floodplain management programs see the following documents:

- *City of Grand Prairie Watershed Technical Report, February 2005*
- *Unified Development Code, Article 14 "Drainage"*
- *Unified Development Code, Article 15 "Floodplain Management"*

F. Watershed Planning

Watershed and storm water master plans can be used to identify drainage and stream

Arbor Creek check dam reduces channel velocities.



segments in need of improvement or restoration as well as structure and potential locations for regional storm water control facilities. Watershed planning can provide the necessary information for conserving natural areas, riparian buffers and greenways, water supply, wetland protection, stream bank and stream corridor restoration, habitat protection, protection of historical and cultural resources, enhancement of recreational opportunities, and aesthetic and quality of life issues.

G. Site Planning

A basic five step process can be implemented including the review of (1) Concept Plans, (2) Preliminary Plats, and (3) Final Plats. When the site plan in approved, it is then used as a guideline for (4) Construction Inspections and the ongoing (5) Operation and Maintenance.

For additional information on Site Planning considerations for drainage ways, see Section 9, "Environmental Quality" of this plan.

H. Resources and Documentation

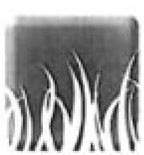
The storm water management concepts and recommendations in this report were compiled from various documents. The reader is referred to the documents below for a more detailed description.

1. Code of Federal Registry CFR 44, National Flood Insurance Program (NFIP)
2. Trinity River Corridor Development Certificate Manual 3rd Edition
3. City of Grand Prairie Floodplain Mitigation Plan
4. City of Grand Prairie Capital Improvement Program (CIP) 2002-2010
5. City of Grand Prairie Annual Repetitive Loss Plan
6. NCTCOG iSWM Manuals
7. Texas Administrative Code 11.086
8. City of Grand Prairie Drainage Design Manual
9. City of Grand Prairie Unified Development Code Articles 14, Drainage, and Article 15, Floodplain Management
10. HEC 22 Drainage Design Manual
11. The information in this appendix also comes from Watershed Studies for:

a. Cottonwood Creek
b. Johnson Creek
c. Bear Creek
d. Dalworth Creek
e. Fish Creek
f. Kirby Creek
g. Alsphaugh Branch
h. Joe Pool Lake Masterplan
i. Master Hydrology Study (F&N)
j. Hight Hollings Branch
k. Henry Branch
l. Garden Branch
m. Beacon Branch
n. City-Wide Drainage Master Plan Road Map (2010 Adoption)

2010 Comprehensive Plan Update

The Kirby Creek Regional Storm Water Controls (RC1), located at Grand Prairie Airport, was constructed since completion of the 2005 Comprehensive Plan. The other re-



gional storm water control sites, that were proposed as part of the 2005 plan, are still under review.

Since the 2005 Watershed Technical Report, a new City-Wide Drainage Master Plan has been approved. The new plan will incorporate certain elements of the 2005 report into the Drainage Master
Plans for each of the watersheds. However, the Road Map for the 2010 plan lays out a more detailed guide for how consultants are to develop the Drainage Master Plans for each of the 12 major watersheds (see Watersheds, Map #6).

This Road Map provides consistency between the studies and allows the city to rank projects across the city based on similar ranking criteria.

City-Wide Drainage Master Plan

The City-Wide Drainage Master Plan (CWDMP) was approved by City Council on August 3, 2010 by Resolution 10-4456. The CWDMP is described in more detail on the following pages.

The CWDMP establishes the processes for future flood control planning for the City of Grand Prairie. The city's primary goal and objective of the CWDMP is to cost-effectively manage flood or storm waters within budget constraints so that conditions don't get worse as new and infill areas are developed—while evaluating and making conditions better (prioritized improvements) in the areas of the city that are already developed.

City-Wide Drainage Master Plan (CWDMP) Road Map—Goals

The City-Wide Drainage Master Plan, as outlined in this Road Map, will accomplish the following goals:

1. Provide the building blocks to reduce the existing potential for floodplain and storm water damage to public health, safety, life, property, and the environment.
2. Protect and enhance the quality, quantity, and availability of surface water resources.

Channel check dam reduces velocities and protects the streambank.

3. Promote equitable, acceptable, and legal measures for floodplain and storm water management.
4. Address the remaining flooding issues in Grand Prairie, including both inadequate storm drainage systems and floodplains.
5. Provide a comprehensive, City-Wide drainage inventory and assessment with recommendations for flooding and drainage issues.
6. Provide a systematic and financially sound strategy for reducing or eliminating flooding in Grand Prairie.
7. Provide short term goals for constructing smaller projects and a long range plan for larger, more complex projects.
8. Identify and prioritize the needed improvements for small, medium, and large projects for both City-Wide and individual watersheds.

Baffle block dissipates energy on the outflow of a box culvert.

To accomplish these goals, the individual Drainage Master Plan for each watershed will need to provide the following:Careful examination of drainage and flooding issues in each watershed, including major streams, tributaries, and storm drainage systems.

1. Careful examination of drainage and flooding issues in each watershed, including major streams, tributaries, and storm drainage systems.
2. Review of citizen drainage complaints to more accurately define trouble areas.
3. Review of all existing available data for each watershed, including technical studies, reports, and design projects.
4. Understanding of unique attributes of each watershed.
5. Preparation of sound hydrologic and hydraulic and storm drain models and making these models consistent for each watershed.

   A goal of these studies is also to provide new, updated models that can be calibrated against Grand Prairie's new flood warning system stream gages.
6. Provide new and updated floodplain mapping based on the best data available, including modeling, field surveys, and topography.

7. Prepare detailed, innovative alternatives for streams, open channels, and storm drainage infrastructure. Considerations will be made for "less-than 100 -year design" in difficult cases.
8. Document all dams, levees, detention located in each watershed and determine how these are affecting flooding issues.
9. Provide updated GIS information based on watershed study results to ensure that City staff has the most current, updated information available for their use.
10. Provide a schedule for maintenance on specific streams and drainage features for each watershed.
11. Evaluate and Prioritize stream, open channel, and storm drainage infrastructure alternatives so projects can be built to address both major and minor flooding issues over time and in the best possible order. Weigh flood control benefits against project costs.
12. Provide detailed, easy to understand documentation for City staff to make the best decisions on which projects need to be considered at the appropriate timeframe in the future.

For related information on drainage studies, floodplain management and storm water management, see Section 9, Environmental Quality of this plan.

City of Grand Prairie-Individual Watershed

The City of Grand Prairie extends in a north to south direction from north of IH-30 to south of IH-20. Grand Prairie, including its two ETJ's, is located in four counties: Tarrant, Dallas, Ellis and Johnson. For the purposes of the City-Wide Drainage Master Plan effort, the City of Grand Prairie has been divided into the following major watersheds:

Table 1—Grand Prairie Individual Watersheds and Planning Study Priority

Grand Prairie Individual Watersheds	Watershed Priority
Joe Pool Lake	1
Fish Creek	2
Cottonwood Creek	3
Cedar Creek	4
Johnson/Arbor/Barrett	5
West Fork Trinity River	6
Mountain Creek	7
Dalworth Creek	8
Gopher/Turner	9
Bear Creek	10
Dry Branch	11
Alspaugh Branch	12

As shown in Table 1, the city has determined the priority of planning studies for each of the individual watershed areas. Current and future planning studies follow this general order. Figure 1 (on the following page) is a map of the watersheds across Grand Prairie.

As each watershed drainage master plan is completed and adopted for use, its recommendations shall be used for current and future development.

City of Grand Prairie 2010 Comprehensive Plan

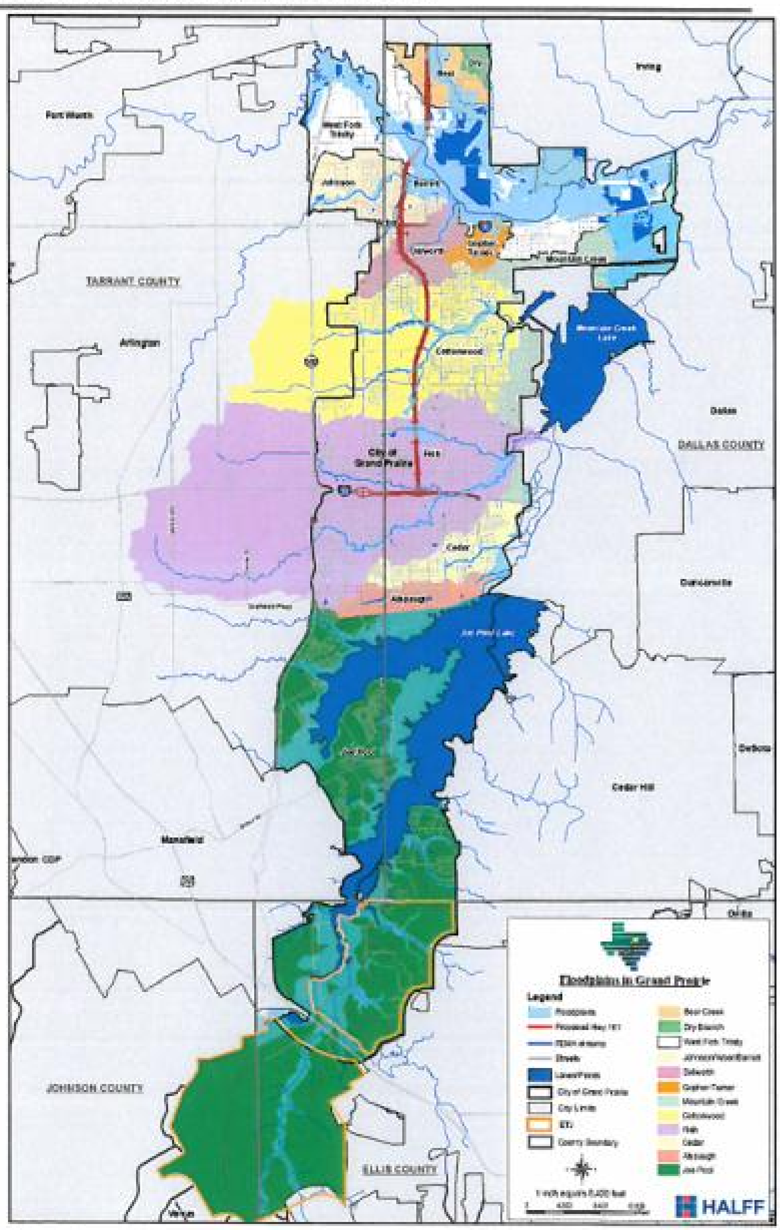


Figure 1. Watersheds across Grand Prairie.

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APPENDIX G

SITE DESIGN PRACTICES (CONTINUED)

G.2 iSWM Technical Manual – Planning Category

The link to the NCTCOG web site is given below:

http://iswm.nctcog.org/program_guidance.asp

The link to the current iSWM integrated planning and site design practices provided by NCTCOG is given below. Select ***iSWM*** **Technical Manual** and the **Planning Category** for the recommended practices:

http://iswm.nctcog.org/technical_manual.asp