CITY OF HOUSTON
Public Works and Engineering
Department

Bill White
Mayor

Michael S. Marcotte, P.E., DEE
Director
P.O. Box 1562
Houston, Texas 77251-1562

T. 713.837.0037
F. 713.837.0040
www.houstontx.gov

October 14, 2005

Ms. Christina Lindsay
Executive Director
Houston Council of Engineering Companies (HCEC)
2150 West 18th Street, Suite 205
Houston, Texas 77008

REF: Technical Paper No. 101
HCEC Review Comments-Overland Flow

Dear Ms. Lindsay:

We appreciate your assistance in reviewing and commenting on Technical Paper No.101 (TP101), Guidelines for Consideration of Overland Flow for the Extreme Event for Improvement Projects in the City of Houston, Harris County, Texas Region. We have reviewed your comments and recommendations, and appreciate the input provided by HCEC. A response indicating the City's action is provided for each comment.

Technical paper No. 101 has been revised to reflect the actions indicated. A copy is attached for your use. The document will be posted on the City's website at:
http://www.publicworks.cityofhouston.gov/documents/index.htm, under "What's New" and Design Manuals.

Thank you for HCEC's assistance in review and development of this document. If you have any questions, please contact me at 713- 837-7114, or Mrs. Kathlie S. Jeng-Bulloch, P.E., at 713-837-7690.

Sincerely,

John J. Sakolosky, P.E.
City Engineer

Attachments:	Response – HCEC Review Comments-Overland Flow Final version of Technical Paper No. 101

JJS:KSJB:ksjb

c: Michael S. Marcotte, P.E., DEE
Paul R. Nelson
Kathlie S. Jeng-Bulloch, P.E.
Daniel Krueger, P.E.
Roger Arya

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Technical Paper No. 101
City Responses to
HCEC Review Comments – Overland Flow 1

Category 1 Recommendations

1. Clarify usage of the terms “Extreme Event” and “100-year event”. They are used interchangeably in the document and Chapter 9, but typically the “Extreme Event” has been characterized as something larger than the 100-year event that is not quantifiable.

   Recommendation: Use the term “1% Exceedance Strom Event” or “100-year event” in place of “Extreme Event” throughout the document. The title would read “Guidelines for Consideration of Overland Flow for the 100-year Event…”

   Response: These terms will be defined in the paper to clarify their use.

2. Replace the word “flood” with “ponding” throughout the document when referring to water levels that inundate a street but do not cause structural flooding.

   Response: These terms will be defined in the paper to clarify their use.

3. No guidance is provided for the use of dynamic hydraulic model, Method 4.

   Recommendation: Some guidance should be given so that the engineering community and City staff has a basic understanding and agreement of the assumptions that are used to develop a model. At a minimum, parameters for precipitation and hydrograph shapes should be defined.

   Response: A reference to HCFCD guidelines for typical parameters will be added.

4. Section 4.0, page 8 paragraph1. The paragraph allows mixing and matching of methodologies for a given system. We believe a system should be evaluated using one or another method and should not be used interchangeably.

   Recommendation: Delete paragraph.

   Response: This paragraph will be removed

5. Section 4.1, page 9, paragraph 3. The statement that thee system is not being sized for the 100-year storm is incorrect. The system **is** being sized for a 100-year storm with a boundary condition defined as the MPE.

   Recommendation: Delete paragraph.

Response: This paragraph will be reworded

6. Section 4.2 Method 2. A more detailed step-by-step description of the analytical process for Method 2 is warranted. A sample project that employs the use of HEC-RAS should be evaluated to define the procedure and assumptions.

   Recommendation: Provide HCEC with a specific application of Method 2 utilizing HEC-RAS for further discussion on the applicability of the method. A step-by-step guideline for Method 2 utilizing the weir equation has been developed by HCEC for consideration of inclusion in TP101 (see attached).

   Response: An example utilizing HEC-RAS can be added at a later date as an appendix when a real world example of its use has been produced. The use of a weir equation to determine overland flow results in a conservative estimate of that flow (i.e. results in a relatively smaller flow for a given depth, therefore more of the total storm water flow must be accounted for by conduit flow). For a road on a slope (the slope being considered from High Point to High Point), the road acts more as a channel than a weir. For roads with a slope greater than 0.1%, for example, a given flow will be at critical depth at the control point. In this case, the critical flow for a given depth is greater than that calculated for weir flow. The flatter the slope of the road the closer the channel flow is to weir flow in terms of depth. This assumes using only the width of the road (no flow beyond the back of curb), and the depth is relative to the gutter. A rating curve for weir flow in a street will be included as a simplified and conservative way to estimate allowable overland flow at the control point. However, a constraint will be that it only considers flow within the actual width of the road (curb face to face). In addition, the generalized weir flow equation will be used to take into account the geometry of the road cross section (i.e. cross slope of the road section), which is more accurate than the horizontal weir equation proposed by HCEC, and depth will be taken from the gutter line.

7. Section 4.3 Method 3. A more detailed step-by-step description of the analytical process for Method 3 is warranted.

   Recommendation: A step-by-step guideline for Method 3 had been developed by HCEC for consideration of inclusion in TP101 (see attached).

   Response: The method proposed by HCEC is virtually the same as that presented in the paper, and is no more detailed. Both methods require the determination of five variables, however, the method presented in the paper uses one calculation to determine the results whereas that presented by HCEC requires the calculation of five equations resulting in more time and a greater chance for error. No change will be made to this method other than some small rewording.

8. Section 5.0: 10-year Tailwater. There appears to be little technical basis for the approach used to estimate the 10-year tailwater.

   Recommendation: Rewrite Section 5.0 to describe a process whereby the 10-year tailwater is estimated by taking the ratio of 10-year runoff depth to 100-year runoff depth multiplied by the 100-year storage volume. This provides an estimate of the 10-year storage volume from which the 10-year tailwater can be calculated based upon the stage-storage relationship. Replace the 2-year runoff depths in Section 4.3, page 14, Table 1 with 10-year runoff depths since the 2-year runoff is not needed for this analysis.

   Response: From HCEC's comment, we feel that there is some confusion in this section. The discrepancy is between reference to the tailwater elevation in a detention basin and the tailwater elevation in an outfall channel. The method described in the HCEC recommendation was already mentioned in the paper albeit not as explicitly. This section will be reworded to clarify this issue.

Category 2 Recommendations

1. Section 1.0, page 3, paragraph 2. The MPE is the maximum ponding elevation, but specific project design will define the 100-year water surface elevation at or below the MPE. The 100-year water surface elevation should be the determining factor for establishing finished slab elevations.

   Recommendation: Add the following statement in front of the second to last sentence: "The 100-year water surface elevation, which is to be designed at or below the MPE, will be used to determine the acceptability of finished slab elevations.

   Response: These terms will be defined in the paper to clarify their use.

2. Figure 1 – Overland and Conduit Flow Relationship, page 4. Clarify "Rainfall" in the equation.

   Recommendation: Replace "Rainfall" with "Subcatchment Runoff".

   Response: "Rainfall" will be replaced with "Runoff", and will be clarified.

3. Section 4.1, page 8, last paragraph, through page 9, second paragraph. Consider simplifying the discussion and remove subjective reference to upsizing pipes.

   Recommendation: Replace paragraphs with the following: "Use the 10-year WSEL as a starting tailwater elevation, the storm sewer system HGL is calculated using 100-year flows. An appropriate design requires that the 100-year HGL is below the MPE."

Response: This will be reworded

4. Section 4.1, page9, water balance equation. The equation parameters need to be described in the previous Section 2.0, not in Section 4.1 (Method 1) since the equation is used for all methods.

   Recommendation: Move the water balance equation to Section 2.0.

   Response: The parameters will be defined in Section 2.0; however, the equation will remain in Section 4.1 (without redefining the parameters) for continuity, clarity, and emphasis.

5. Section 4.4, page 15, paragraph 3. As stated, the complex data requirements for a dynamic model require consultation with City staff. Need to clarify who and what would need to be consulted.

   Recommendation: Delete paragraph.

   Response: Who should be contacted at the City will be clarified.

6. Section 6.0, page 18, minimum requirement #3. More guidance is needed for the minimum dimensions of the overland flow swale.

   Recommendation: If Q_0 is zero, swale has a minimum depth of 1-foot.

   Response: This will be incorporated into the document.

7. Section 6.0, pages 19-21. The three design examples provided in Section 6.0 were used to justify the need for TP101 and illustrate Method 1. Now that TP101 has been expanded to include methods 2 and 3 these examples do not appear to add the discussion.

   Recommendation: Delete the three design examples.

   Response: The examples provide a good discussion of typical situations and provide an acceptable graphic showing how the storm sewer system will function. Therefore, the examples will remain for additional clarity.

Category 3 Recommendations

1. The tone of the document should be reworked to “tell the story” in a technical manner, not to advocate the need for overland flow analysis.

   Recommendation: The various subjective comments included in the document should be removed. The discussion in Section 4.1 regarding Method 1 should be streamlined to convey only the technical basis of the analysis consistent with the Method 2 and 3 write ups provided herein

Response: Agree, the paper will be edited relative to this intent.

2. The stated complexity of SWMM modeling and the “arduous”, “complex”, “sophisticated”, “much too complex”, etc. adjectives used throughout the document to describe Method 4 seem out of place in technical paper.

   Recommendation: Remove the descriptive adjectives.

   Response: Agree, and will be modified as such.

3. Section 1.0, page 3, paragraph 3. Remove statements regarding complexity of dynamic hydraulic models and clarify wording.

   Recommendation: Delete “resulting in a more refined storm water infrastructure design”; replace “results of controlling” with “for evaluating”; delete “within our ROWs without the onerous computational procedures that are so often associated with other more dynamic methods.”

   Response: Agree. This will be reworded.

4. Section 3.0, page 6, paragraph 3. The paragraph addresses the complexity of dynamic models, which was preciously covered in Section 1.0, paragraph 3.

   Recommendation: Delete paragraph.

   Response: This paragraph will be retained, but reworded.

5. Section 4.0, page 7, paragraph 2. The paragraph addressed the definition of the MPE, which was previously covered in section 1.0, paragraph 2.

   Recommendation: Delete paragraph.

   Response: This paragraph will be retained, but reworded.

6. Section 4.1, page 8, paragraph 2. Remove statements regarding complexity of dynamic hydraulic models.

   Recommendation: Delete “however, the fast and easy computational process and the resulting minor increase in size of some storm sewer reaches to achieve the desire results can readily offset the burdensome storage and routing computations that would alternately be required.

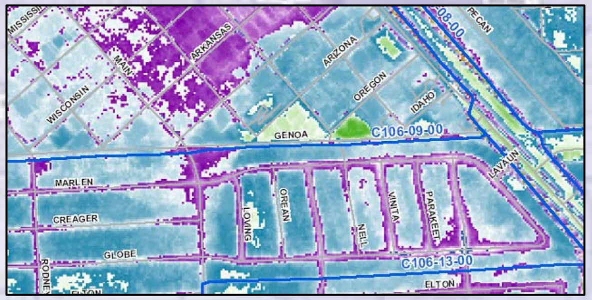
   Response: Agree. This paragraph will be reworded.

7. Section 4.3, page 14, paragraph2. Define “analysis control points.”

   Response: These terms will be defined in the paper to clarify their use.

City of Houston, Texas
Department of Public Works and Engineering
Technical Paper No. 101
(TP-101)

Guidelines for Consideration of Overland Flow for the Extreme Event for Improvement Projects in the City of Houston, Harris County, Texas Region



SWMP
Houston Storm Water Management Program

April 15, 2005

City of Houston, Texas
Department of Public Works and Engineering
Technical Paper No. 101
(TP-101)

Guidelines for Consideration of Overland Flow for the Extreme Event for Improvement Projects in the City of Houston, Harris County, Texas Region

April 15, 2005

Abstract

This paper provides a description of the four methods outlined in Chapter 9 of the City of Houston Infrastructure Design Manual[1]for the analysis of overland flow in public and private improvement projects. The analysis of overland flow requires the design engineer to demonstrate that the project maintains water surface elevations (WSELs) below the Maximum Ponding Elevation for the 100-year event. The methods described in this technical paper are intended to serve as guidelines to aid the design engineer in adhering to the City's overland flow criteria, without the utilization of complex computer simulations or other complex computational procedures. Analysis of open channels and roadside ditches are not specifically included in this paper; however, the theoretical application is similar.

Authors: James F. Thompson, P.E. (J.F. Thompson, Inc.)
Richard A. Kraus, P.E. (CivilTech Engineering, Inc.)

Reviewers: John J. Sakolosky, P.E. (City of Houston, Department of Public Works)
Michael S. Kane, P.E. (J.F. Thompson, Inc.)
Eric D. Nevil, P.E. (J.F. Thompson, Inc.)
Marcia M. O'Connell, P.E. (J.F. Thompson, Inc.)

Acknowledgements:

Mark Sappington, P.E. (Cobb Fendley & Associates) & Lee Lennard, P.E. (Brown & Gay, Inc.) for their contribution to the overland storage computation procedure.

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[1] City of Houston Department of Public Works and Engineering Infrastructure Design Manual, Revision 4, February 1, 2005.

TABLE OF CONTENTS

1.0 Introduction

This technical paper presents guidelines for the evaluation of overland flow in public improvement and private development projects. The guidelines are intended to aid the design engineer in demonstrating that 100-year water surface elevations (WSELs) do not exceed the Maximum Ponding Elevation

1.1. Definitions

Flood –The term flood is used where storm water, generated from the runoff during a storm event, goes beyond the bounds of the water conveyance system that is under consideration. This can be a bayou, channel, underground culverts and piping, street, or other designated conveyance system. For example, if the underground storm sewer system is being considered, then storm water in the street would be considered flooding. In this paper, which is dealing with urban storm water, flooding is considered the above ground storm water that inundates the area around and exceeds the bounds of the storm water conveyance system during storm events. Flooding is this sense is not defined as either acceptable or unacceptable, but rather just where storm water exceeds the boundaries of a defined conveyance system. In this paper, a flood is where storm water depth in the street conveyance system exceeds the Maximum Ponding Elevation as described in Chapter 9.05.D.4 and threatens the flooding of structures.

Ponding – Ponding is usually considered as standing water. In the design and analysis of storm water systems in urban areas, ponding is frequently considered in relation to some allowable amount of standing water in streets during storm events. Therefore, the amount of ponding that is acceptable is independent of flooding, although can be a cause of it, but rather is dependent on the geometry of an area and the acceptable level of water that can be contained within it that does not have an adverse affect on the area based on some set of given criteria. As such, **acceptable ponding**, as it relates to this paper, is defined as street ponding, resulting from runoff during a storm event, contained within the area around the street such that the elevation of the ponding does not exceed the Maximum Ponding Elevation or elevation of the Maximum Ponded Depth (which are discussed below), whichever is less. To differentiate flooding from ponding; flooding is where water **exceeds the boundaries** of some containment or conveyance system, whereas ponding is collected water that is contained **within the boundaries** of some defined area.

Maximum Ponding Elevation (MPE) – As defined in Chapter 9.05.D.4.g in the City of Houston Design Manual, the Maximum Ponding Elevation (MPE) “at any point along the street shall not be higher than the natural ground elevation at the right-of-way line” or for new subdivisions/developments “no higher than 12 inches below the proposed finished slab elevations, or, if the proposed finished slab elevations are less than 12 inches above the ground elevations at the right-of-way, the ponding elevations shall be no higher than the ground elevations at the right-of-way”. In essence, the lowest water surface elevation is the controlling elevation for the MPE. However, it should also be noted that the Design Manual also says, “The limiting parameter…and the most restrictive condition (the lowest ponded water elevation) shall govern.” This means that the MPE cannot necessarily be taken as the limiting factor in determining the maximum allowable water surface elevation, but that the maximum depth of ponding at street high and low points, 6 inches and 18 inches above top of curb respectively (as defined in Chapter 9.05.D.4.b and c), must also be considered and compared to the MPE, with the lowest value governing. In this paper, it is assumed for the purpose of discussion, that the MPE is the most restrictive condition.

Extreme Storm Event – The extreme event is usually considered an event with a relatively large recurrence interval, and typically greater than the normal design event, which is usually between the 2-year and 10-year events (for Houston the storm sewer design event is the 2-year). Therefore, the extreme storm event could be a 25-, 50-, 100-, 500-year, or other event greater than the design event. Thus the methods in this paper can apply to any extreme event. In Houston, an extreme event is usually considered the 100-year storm event. In this paper the term extreme event is used when speaking in general terms about storm water effects that are not just specific to the 100-year event, but can be related to any extreme event.

100-year Storm Event – This is the recurrence interval of a storm event expressed as the inverse of the probability of occurrence, i.e. 1/P, where P is the probability that a given event will be equaled or exceeded in any given year. In the case of the 100-year event (the recurrence interval), for example, the probability is 0.01, or there is a 1% chance that an event will occur in any given year that will equal or exceed this event. A 100-year event does not mean that an event will only occur once every 100 years. A 2-year event, for example, has a 50% probability of occurrence in any given year.

Control Points – When considering overland flow and storage in a storm sewer analysis, a control point is the high point of a roadway on the downstream end of the segment of storm sewer that is being analyzed.

1.2 Basic Applications

The Maximum Ponding Elevation (MPE) criteria was developed to provide an increased flood protection level-of-service for the 100-year storm event. Analysis of open channels and roadside ditches are not specifically included in this paper; yet, the theoretical application is similar.

Three simplified methods presented herein yield acceptable results in the analysis of overland flow and WSELs within our ROWs. When applicable, more complex methods (i.e. unsteady and fully dynamic simulations) may be employed in storm water infrastructure design,

One of the fundamental assumptions correlated with the methods presented for the consideration of overland flow is that the determination and mitigation of hydraulic impacts resulting from the project at hand has already been addressed in some fashion. This is most commonly the case where detention basins are employed in a local, sub-regional, or regional basis. In the cases of applied detention basins, the basin outlet, restrictor, and storm sewer reach to the ultimate outfall, if utilized, remains unchanged in terms of design as currently dictated by Chapter 9 of the City Design Manual.

2.0 The Relationship of Overland and Conduit Flow

Overland flow is termed as flow resulting from a rainfall event that is routed along surface streets or surface channels in a defined manner. This differs from sheet flow which is a shallow depth of runoff on a sloping surface that does not have a precisely defined bounding condition. Conduit flow is that portion of the total system flow routed through the storm sewer system pipe, box, or other closed hydraulic conveyance element.

Figure 1 illustrates the general relationship of overland to conduit flow in a storm water system, which can be expressed as:

$$Q_{O\,in} + Q_{C\,in} + Runoff = Q_{O\,out} + Q_{C\,out} + \frac{\Delta \mathrm{S_T}}{\Delta \mathrm{t}},$$

where:

$Q_{O\,in}$ = overland flow in

$Q_{C\,in}$ = conduit flow in

$Q_{O\,out}$ = overland flow out

$Q_{C\,out}$ = conduit flow out

$\frac{\Delta \mathrm{S_T}}{\Delta \mathrm{t}}$ = the change in storage with respect to time

Runoff= that fraction of rainfall for a given intensity over a given area that flows into the system based on the physical parameters of that area

Rainfall runoff is accumulated within a street system and is conveyed to inlets and then ultimately a storm sewer. When the rainfall intensity and resulting runoff exceeds the capacity of the storm sewer system (inlets, leads, and trunk system), the excess is stored within the street and then, based upon the roadway profile geometry, is routed downhill, typically toward an outfall location. Water balance is maintained by the inclusion of the relationship of storage relative to time.

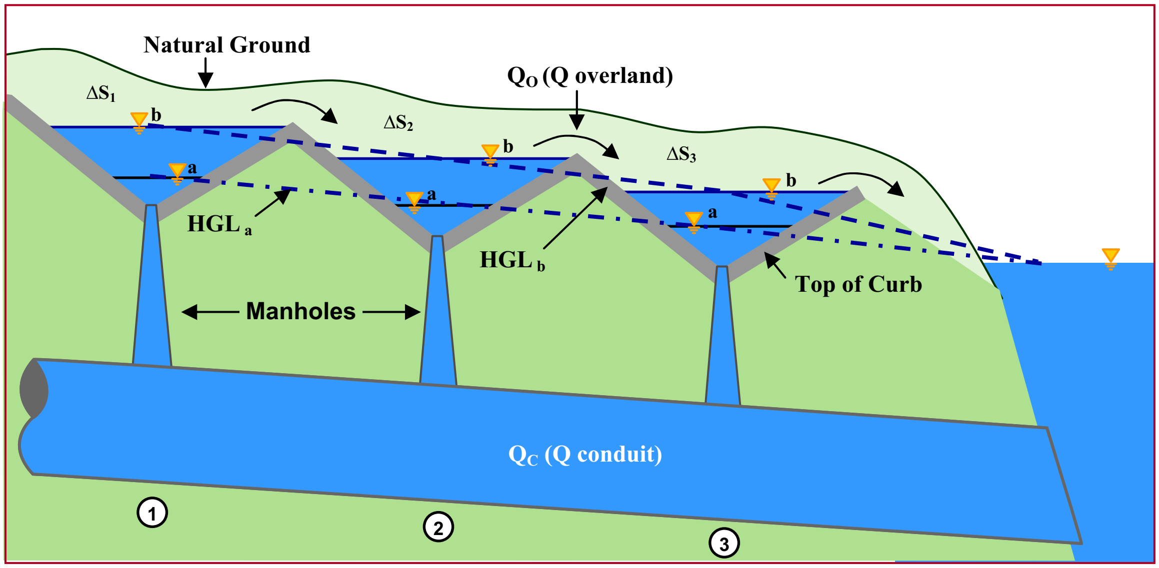

Figure 1 - Overland and Conduit Flow Relationship

The behavior of a street, inlet, inlet lead, and storm sewer system, when considering the effects of storage, can be complex. This is compounded when the effects of high tailwater at the outfall (or outlet) are considered. If the outfall WSEL remains unchanged, the increase in pressure head identified by the hydraulic grade line (HGL), say from *a* to *b* in Figure 1, will result in an increase in flow through the conduit. If all conduit conditions remain unchanged (i.e. size, length, and coefficient of friction), the flow in the conduit is directly proportional to the square root of the head loss as applied in Manning's equation. The relationship between the increase in driving head on a given conduit system and the resulting conduit flow is a fundamental aspect of controlling WSELs within streets and thoroughfares. If high tailwater conditions at the outfall of a system result in a reduced flow capacity of the conduit, then the overland flow component of the total system flow (overland plus conduit flow) in conjunction with the effects of storage in the street section relative to time becomes a more significant component of the system.

The WSEL, or flood level, in a roadway section during an extreme storm event is not the same elevation as the pressure head on the storm sewer system as identified by the HGL as the effects of inlets, leads, and manholes are not commonly included in the HGL computation. When the HGL of a given storm sewer for a specific event is below the gutter line of a roadway section, then the observed WSEL in the roadway is controlled by inlet, inlet lead capacity, and roadway geometry. The true effects of inlets, leads, and manholes can be noticeable depending on the storm sewer system and the storm event applied. There is a relationship, however, between the observed WSEL in a depressed roadway section and the HGL of the storm sewer serving the roadway during an extreme storm event when the HGL is above the gutter line. Given that sufficient inlet capacity exists and that the inlet leads are sized accordingly, the HGL of the storm sewer system will yield a simplified approximation of the anticipated WSEL in a roadway section, as long as bounding conditions of the HGL exist. As such the observed WSEL in a depressed roadway section during a 100-year storm event is used interchangeably with the resulting 100-year HGL of the storm sewer as long as the said bounding conditions are recognized as discussed later. This is a simplification that the designer should recognize.

Figure 2a represents a typical series of inflow and outflow hydrographs for a given depressed roadway section. The inflow hydrograph is representative of total system inflow at a given location from upstream overland flow, conduit flow, and rainfall. The system outflow is a function of conduit and downstream overland flow. By combining these two outflow hydrographs, in relation to time, the area under the inflow hydrograph and above the combined outflow hydrograph represents the storage within the roadway section that would be later routed through the conduit as depicted in Figure 2b. This is inherently a simplified representation as in many conditions in the Houston area, conduit flow will vary greatly based on outlet tailwater conditions relative to time. As described above, the conduit flow may be reduced to zero or even become negative flow – representing flow from the outlet *upstream* into the conduit – and overland flow in conjunction with storage will become the predominate drainage mechanism.

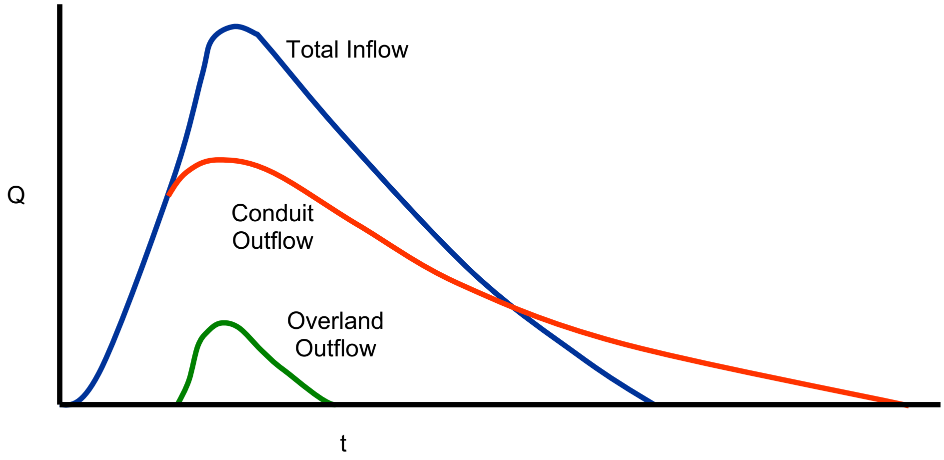

Figure 2a – Inflow and Outflow Hydrographs

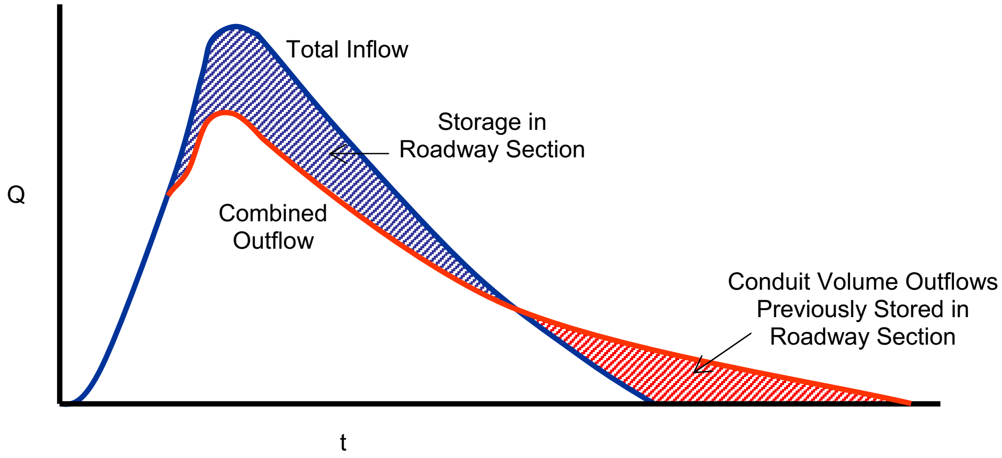

Figure 2b - Combined Outflow Hydrographs Showing Storage

3.0 Storage and Routing Considerations

The effects of storage and routing cannot be over emphasized in the process of urban storm water management. As the pressure head on a storm sewer system is increased as visualized by the HGL, the resulting WSEL may reach a level within a given depressed roadway section whereby flooding beyond the ROW will occur. While the HGL can be calculated based upon Manning's equation to represent an increasing pressure head, it is unrealistic to have the HGL exist above established boundary conditions such as adjoining natural ground elevations. Once the HGL reaches such known boundary conditions in terms of elevation, then the pressure head will cease to rise, or rise only minimally, as areal flooding will occur beyond the established bounding natural ground elevations.

As the WSEL rises within a roadway section, the head on the storm sewer increases, thereby increasing the flow within the storm sewer system (assuming the downstream head level is not increased by the same increment) up to a point at which little, if any, increase in WSEL is possible. This point may be when areal flooding beyond the ROW is experienced, when overland flow occurs within the ROW, when sheet flow occurs away from the system outside of the ROW, or a combination of these phenomena. In all of these cases, the storage within the roadway section plays an important part of the overall water balance equation.

The actual routing of the total system flows relative to WSEL and time, considering inflow, overland flow, conduit flow, and storage, and using hand calculation methods is complex and time consuming. Computer simulations using dynamic models may be employed[2], however, when difficult or complex systems may warrant their use.

4.0 Various Methods to Consider Overland Flow Criteria

The purpose of this section is to provide a range of computation methods for the consideration of overland flow during an extreme event, and for the City of Houston this is typically the 100-year event. The City of Houston Infrastructure Design Manual, Chapter 9, Stormwater Design Requirements, Section 9.05, Design Requirements states: "An overland flow analysis of the proposed drainage system shall be prepared by the design engineer. The design engineer shall submit supporting calculations, exhibits, and drawings." The computational methods described in this section refer to those listed in the Design Manual. This section will provide a step-by-step guide to performing the calculations needed in order to prove that the design meets or exceeds the City of Houston design requirements by demonstrating that the project maintains water surface elevations (WSELs) below the Maximum Ponding Elevation, as defined at the beginning of this paper, for the 100-year event.

The analysis of overland flow starts after the storm sewer has been properly designed for the 2-year storm event in accordance with the City of Houston design criteria (i.e., the 2-year HGL is maintained below gutter elevations for depressed curb-and-gutter sections). The overland flow computations are then performed with the results focusing on satisfying the City's criteria.

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[2] U.S. Environmental Protection Agency. *U.S. Environmental Protection Agency's Storm Water Management Model (SWMM)*, Retrieved May 1, 2003 from http://www.epa.gov/ednrmrl/swmm/

The four methods to be discussed include the following:

Method 1: Conduit Hydraulic Grade Line Analysis
Method 2: Conduit and Overland Flow Analysis
Method 3: Conduit and Overland Flow with Storage Analysis
Method 4: Dynamic Flow Routing Analysis

The first three methods are listed in the Design Manual in Section 9.05.D.2.a. The fourth method is contained in the Design Manual in Section 9.05.D.2.b, and is included in this Technical Paper. However, Method 4 involves an in depth computer model analysis of a system, which is too complex to describe in a step-by-step fashion similar to Methods 1 through 3.

The methods become more complex in terms of the parameters that need to be determined and the equations used to perform the analyses. The analysis methods are intended to build upon each other, such that the parameters determined from one method can be used in another method. The analysis methods do not need to be performed in successive order. The analysis could consider the overland storage effects (Method 3) to show compliance with the design criteria without determining the 100-year HGL within the storm sewer (Method 1), or computing the available overland flow capacity (Method 2).

4.1 Method 1: Conduit Hydraulic Grade Line Analysis

Method 1 considers only flow within the storm sewer conduit. Overland flow conveyance and/or storage volume capacities are not considered. This method computes the 100-year HGL by applying the 100-year peak flow rates to the storm sewer system without regard to the natural ground elevations that exist above the storm sewer. The acceptance trigger for this Method is if the computed 100-year HGL remains below the MPE, as defined in Section 9.05.D.4.g of the Design Manual.

The approach is to control the 100-year WSEL by modifying the designed storm sewer (resulting in a change in the frictional losses), thereby adjusting the position of the HGL in order to meet the criteria. Again, this method does not consider the effects of storage.

The friction loss of a given conduit can be represented by Manning's equation as follows:

$$h_\ell = L\left(\frac{Qn}{1.49\,AR^{2/3}}\right)^2$$

where: h_ℓ = head loss in conduit reach (ft)
L = length of conduit reach (ft)
Q = flow (cfs)
n = Manning's roughness coefficient
A = cross - sectional area (ft^2)
R = A/P, the hydraulic radius (ft)
P = wetted perimeter (ft)

This equation is commonly used to compute the friction loss of a given conduit reach and the resulting upstream HGL ordinate is plotted in a profile view. A series of successive computations, running from downstream to upstream, result in a plot of the HGL for a given design event.

As typically applied in Houston, the 2-year frequency design storm event is used to initially size the storm sewer system for a given project and then the 2-year HGL is computed, using the outfall soffit as the starting WSEL[3], to insure its position is at or below the gutter line of the roadways within a given project area. A 100-year storm event, as identified by the intensity-duration-frequency (IDF) curve in the City Design Manual, and the resulting runoff is then applied to the storm sewer system and a check is made to insure that the 100-year HGL, using the 10-year WSEL at the outlet as the starting WSEL, is at or below the MPE. If these criteria are not met, then the system should be reviewed, and steps taken to modify the system to ensure that these criteria are in fact met. For example, steps can be taken to identify reaches having a relatively high degree of frictional head loss, and therefore can be upsized in order to reduce the HGL and meet the criteria[4].

The 10-year starting water surface elevation should be applied to the 100-year HGL computations. The logic behind this application is simply that when a 100-year event occurs over a project area, the tailwater elevation at the outlet is usually some lesser level other than the 100-year WSEL. The issue of tailwater and the determination of the 10-year WSEL at the outlet are discussed in the next section.

It is important to note that the system is sized for the 2-year storm event, and then stressed with the 100-year storm event to assess the performance of the system. The system is then modified as needed, if necessary, to ensure that the 100-year HGL remains below the MPE in all locations of the system.

The steps to perform the Method 1 analysis are as follows:

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[3] In cases where drops exist, the HGL computations begin again at the soffit of the conduit upstream of the drop. Refer to Chapter 9 of the City Design Manual for further explanation.

[4] The issue of inlet capacity is not specifically addressed herein, but test cases of typical new development projects using procedures outlined in the FHWA's HEC 22 have shown that the standard inlet design density as called for in Chapter 9 of the City Design Manual provides for a very suitable inlet spacing to accommodate the hydraulic connection required for the 100-year storm event analyses as described within this Technical Paper.

1) Referring back to the water balance equation in Figure 1:

$$Q_{O\,in} + Q_{C\,in} + Runoff = Q_{O\,out} + Q_{C\,out} + \frac{\Delta S_T}{\Delta t},$$

Now, defining all inflows at a given location as total flow, Q_T, then:

$$Q_T = Q_O + Q_C + \frac{\Delta S_T}{\Delta t},$$

where: Q_T = total peak 100-year flow from the upstream drainage area
Other terms as previously defined

Since this method does not consider any overland flow or storage:

$$Q_T = Q_C$$

The total 100-year flow rates (Q_T) are computed based on parameters used to compute the 2-year design storm with the Rational Method as follows:

$$Q_T = I\Sigma\,(CA)$$

where:
C = watershed coefficient
A = drainage area (acres)
I = rainfall intensity (inches per hour)

The drainage area (A) and watershed coefficient (C) will not change. The rainfall intensity (I) will be revised from the 2-year storm to the 100-year storm by adjusting the e, b, and d coefficients in the equation:

$$I = b\,/\,(TC + d)^e$$

where:
I = rainfall intensity (inches per hour)
TC = time of concentration (minutes)
b,d,e = coefficients based on rainfall frequency from Chapter 9, Design Manual

The time of concentration (TC) will not change from that which was computed for the 2-year storm sewer design (another acknowledged simplification).

2) Compute the 100-year HGL along the storm sewer, moving upstream, using the 100-year flows at every location. Start the computations with a tailwater elevation equal to the 10-year water surface elevation of the receiving system.

3) Compare the computed 100-year HGL to the MPE.
4) Select one of the following conclusions:

   - If the 100-year HGL exceeds the MPE, then the design is not acceptable[5]. The engineer must up-size selected conduits (i.e. those with excessive headloss) until the 100-year HGL remains below the MPE; or,

   - another analysis method must be considered; or,

   - if the 100-year HGL is less than or equal to MPE, then the design meets City of Houston requirements.

4.2 Method 2: Conduit and Overland Flow Analysis

This method considers the conveyance capacity of the overland flow path in addition to the flow in the storm sewer conduit. The idea is that if the storm sewer capacity plus the overland flow capacity is greater than the actual peak 100-year flow, then the design is acceptable. This method does not take into consideration the effects of overland storage in attenuating the peak flow. The basic form of the equation used in this analysis method is:

$$Q_T = Q_O + Q_C$$

where:
Q_T = total peak 100-year flow from the upstream drainage area
Q_O = overland flow
Q_C = storm sewer conduit flow (assumed to be the 2-year design flow)

The steps involved in performing the Method 2 analysis are as follows:

1) The peak 100-year flow (Q_T) is the same value defined in Method 1. The peak flow rate is determined at points along the overland flow path where the overland flow would be most restricted. These control points will typically be located at high points in the streets.

2) Compute the allowable overland flow capacity (Q_{Oallow}) at the control points. The allowable overland flow is the flow that would occur at the MPE. The overland flow is

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[5] The HGL does not correlate exactly with the observed WSEL in a roadway section during an extreme storm event as the effects of inlet, lead, and manhole losses must be accounted for. Studies have shown that these losses vary depending upon the ability of the storm sewer to receive additional inflow. The HGL position, as applied herein, is only an approximation of the actual WSEL that would be observed in a roadway section during such an extreme event.

computed using standard backwater computations, Manning's Equation for open channel computations or roadway design template rating curves as developed by the engineer.

- Backwater computations are similar to open channel calculations and could be determined with a hydraulic modeling program such as HEC-RAS. The cross sections would be established at the control points and the discharges would be adjusted until the computed water surface profiles match the MPE's.

- Manning's Equation for open channel flow can be applied to the roadway by treating the curb-and-gutter section as the channel:

$$Q = (1.49/n)\ AR^{2/3}\ S^{1/2}$$

where: Q = flow (cfs)
n = Manning's roughness coefficient (0.013 for concrete)
A = area (ft^2)
R = Hydraulic Radius (ft)=area/wetted perimeter
S = channel slope (ft/ft)

- Roadway design template rating curves provide a discharge value for typical roadway cross sections and depths and slopes. The slope is usually measured between high points along the roadway.

- As an additional simplified and conservative (i.e. results in a relatively smaller flow for a given depth, and therefore more of the total storm water flow must be accounted for by conduit flow) alternative, the use of a weir equation to determine overland flow can be used to estimate this flow. However, weir flow will only be considered within the actual width of the road (curb face to face). The weir equationurves will also take into account the geometry of the road cross section (i.e. cross slope of the road section), and depth will be taken from the gutter line. Rating curves for standard City roadway cross sections showing weir flow verses depth are included at the end of this section.

3) Sum the allowable overland flow (Q_{Oallow}) with the conduit flow (Q_C) and compare the results against the actual peak 100-year flow (Q_T). Qc is assumed to be the 2-year design flow.

4) Select one of the following conclusions:

- If Q_T is greater than the sum of allowable overland flow and conduit flow, then the design is not acceptable.

$$Q_T > Q_{Oallow} + Q_C \qquad \text{Not acceptable}$$

The engineer must up-size selected conduits as described in Method 1 and/or increase the conveyance capacity of the overland flow path, or another analysis method must be considered.

- If Q_T is less than or equal to the sum of the allowable overland flow and conduit flow, then the design meets City of Houston requirements.

$$Q_T \leq Q_{Oallow} + Q_C \qquad \text{Acceptable}$$

4.3 Method 3: Conduit and Overland Flow with Storage Analysis

This method, building upon Methods 2 and 3 previously discussed, considers, in addition to conduit flow and overland flow, the effects overland storage has on attenuating runoff hydrographs and reducing peak flow rates in a simplified manner. The runoff hydrograph is not routed through the available overland storage volume, but the peak flow is reduced in accordance with a simplified relationship between the total runoff volume and the available overland storage volume. This method equates the 100-year peak flow to the storm sewer conduit flow plus the available overland flow plus a form of the routing term, change-in-storage per change-in-time:

$$Q_T = Q_O + Q_C + \frac{\Delta S_T}{\Delta t}$$

where:

Q_T = total peak 100-year flow from the upstream drainage area

Q_O = overland flow capacity

Q_C = storm sewer conduit flow (assumed to be the 2-year design flow)

$\frac{\Delta S_T}{\Delta t}$ = the change in storage with respect to time

A triangular hydrograph is the basis for the computations. The peak flow is equal to the 100-year runoff (Q_T) and the volume is equal to the total runoff for the 100-year storm (V_T). The hydrograph is segmented into that portion of the runoff volume that is conveyed through the storm sewer conduit, the volume that is stored in the overland area and the volume that is conveyed through the overland flow path. Figure 3 illustrates this segmentation.

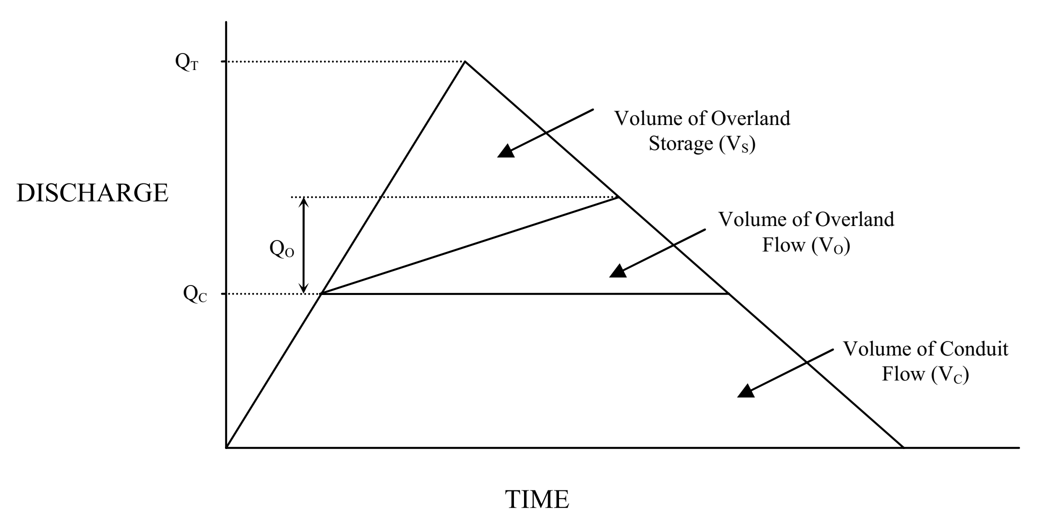

Figure 3 - Triangular Hydrograph Considering Volume Components

The total runoff volume (V_T) is computed based on the percent impervious cover of the drainage area and the storm duration. The storm duration is in accordance with the Design Manual, Chapter 9.05.B.1.a, which states that for drainage area less than 200-acres the duration will be no less than 3-hours and for areas larger than 200-acres the storm duration will be no less than 6-hours. The 100-year total rainfall depths for Houston, Texas as reported by the Tropical Storm Allison Recovery Project for 3- and 6-hour durations are 6.7 and 8.9 inches respectively (Region 2, based on USGS). Table 1 yields various runoff depths for various percentages of impervious cover for both the 100-year and 10-year frequencies using the SCS TR-20 methodology. Interpolation may be used for various percentages of cover not stipulated in Table 1.

		100-year		10-year	
	SCS	Runoff Depth (inches)		Runoff Depth (inches)	
% Impervious Cover	Curve No.	3-hr duration	6-hr. duration	3-hr duration	6-hr. duration
0%	75	3.9	5.8	1.7	2.5
25%	85	5.0	7.1	2.5	3.5
40%	87	5.2	7.3	2.7	3.7
70%	93	5.9	8.0	3.3	4.3
85%	95	6.1	8.3	3.5	4.5
100%	98	6.5	8.7	3.9	4.9
Total Depth*		6.7	8.9	4.1	5.1

**USGS Rainfall Amounts as utilized in TSARP Region 2*

Table 1-Runoff Depths

The volume of overland storage (V_S) is measured upstream of the analysis control points. By using the Maximum Ponding Elevations (MPE's) for this measurement, the volume calculated equates to the available storage volume. In development projects that have successive analysis control points, the available overland storage volume is the sum of the measured volumes from each upstream area. Acceptable methods of estimating this volume include average end area methods or terrain modeling (DTM, *.tin files, etc.).

The steps involved in performing the Method 3 analysis are as follows:

1) Compute the total 100-year runoff volume (V_T) for the entire drainage area upstream of the analysis control point based on Table 1 above.

2) Measure the available overland storage volume (V_{Savail}) based on the MPE that is applicable to the area upstream of the analysis control point. The available overland storage volume will include the storage volume that is available for the entire upstream drainage system.

3) Compute the required overland flow (Q_{Oreqd}) at the analysis control point with the following equation, which is derived from the triangular hydrograph above.

$$Q_{Oreqd} = Q_T - Q_C - \left(\frac{(V_{Savail} \times Q_T^2)}{V_T \times (Q_T - Q_C)} \right)$$

where: Q_{Oreqd} = required overland flow

V_{Savail} = volume of available overland storage

V_T = total runoff volume

Other terms as previously defined

4) Select one of the following conclusions:

    - If the calculated required overland flow is equal to or less than zero, then the design meets the City of Houston requirements.

      $Q_{Oreqd} <= 0$ Acceptable

    - If the calculated required overland flow is greater than zero, the engineer must then calculate the allowable overland flow by the Methods previously discussed and continue with Step 5 below.

      $Q_{Oreqd} > 0$ Calculate allowable overland flow

5) Select one of the following conclusions:

    - If the required overland flow is greater than allowable overland flow then the design is not acceptable. The engineer must up-size portions of the conduit and/or increase the overland flow path conveyance capacity and/or increase the available overland storage volume, or another analysis method must be considered.

$$Q_{Oreqd} > Q_{Oallow} \qquad \text{Not acceptable}$$

- If the required overland flow is less than or equal to the allowable overland flow, then the design meets City of Houston requirements.

$$Q_{\mathrm{Oreqd}} \leq Q_{\mathrm{Oallow}} \qquad \text{Acceptable}$$

4.4 Method 4: Dynamic Flow Routing Analysis

This method routes the inflow hydrographs through overland and conduit drainage segments by utilizing computer models. The typical computer programs used for this analysis are the EPA's SWMM (as previously referenced-see footnote 2), or a variant such as XPSWMM, PCSWMM, MIKE SWMM, or others.

The computer program analyzes the drainage system by defining the conduits, manholes, overland flow paths, storage area, etc. as a series of links and nodes. The computer program routes the inflow hydrographs through the use of the fully dynamic equations for gradually varied flow (St. Venant equations) through time series computations. The User's Manual for the program must be consulted for explicit instructions and application limitations.

The programs are complex in their input data requirements and evaluation. The typical parameters needed as input to dynamic models shall follow current HCFCD guidelines.
Prior to performing this type of analysis, the engineer should consult with the City of Houston City Engineer, Engineering Services regarding the project to which this method is being considered.

5.0 Determination of the 10-year or Other Starting Water Surface Elevation

In project situations where the 100-year WSEL, or less (i.e., 50-year or 10-year), already inundates the project area(s), for whatever reason, it is not the intent of these guidelines to require the design engineer to resolve problem areas beyond their reasonable control. Chapter 9 of the City Design Manual stipulates this criterion and allows the submittal of documentation and an analysis demonstrating this project situation. The intent is to have the design engineer examine the overall system performance (overland and conduit components alike) to insure a desired level-of-service is achieved. In cases where the 10-year WSEL of a nearby bayou, for example, inundates a project area, then obviously lesser tailwater elevations, for analysis purposes, would be warranted. Again, the proper submittal of documentation with supporting analyses is stipulated for these conditions within Chapter 9.

In most commonplace applications, the 100-year WSEL at the outfall is known or established. This outfall condition is usually at a detention basin or channel. In many cases, lesser event WSELs are also known and the 10-year WSEL for an outfall may be determined by stage-frequency interpolation (or other applicable methods). In cases where actual routing is performed through a detention basin, the determination of the 10-year WSEL is easily determined via interpolation given the bounding design event WSELs that are established for the basin.

Lacking suitable and available data from which the 10-year WSEL can be reasonably estimated, a consistent means of establishing the 10-year WSEL at an unspecified outfall location in Harris County is needed. This would allow a designer the ability to rapidly establish the 10-year WSEL without laborious routing or other calculations. To simplify the discussion within this section, Figure 4 illustrates a decision flow chart applicable to typical projects at an unspecified location involving a detention basin used to collect storm water from one or multiple storm sewer systems within a new development or improvement project.

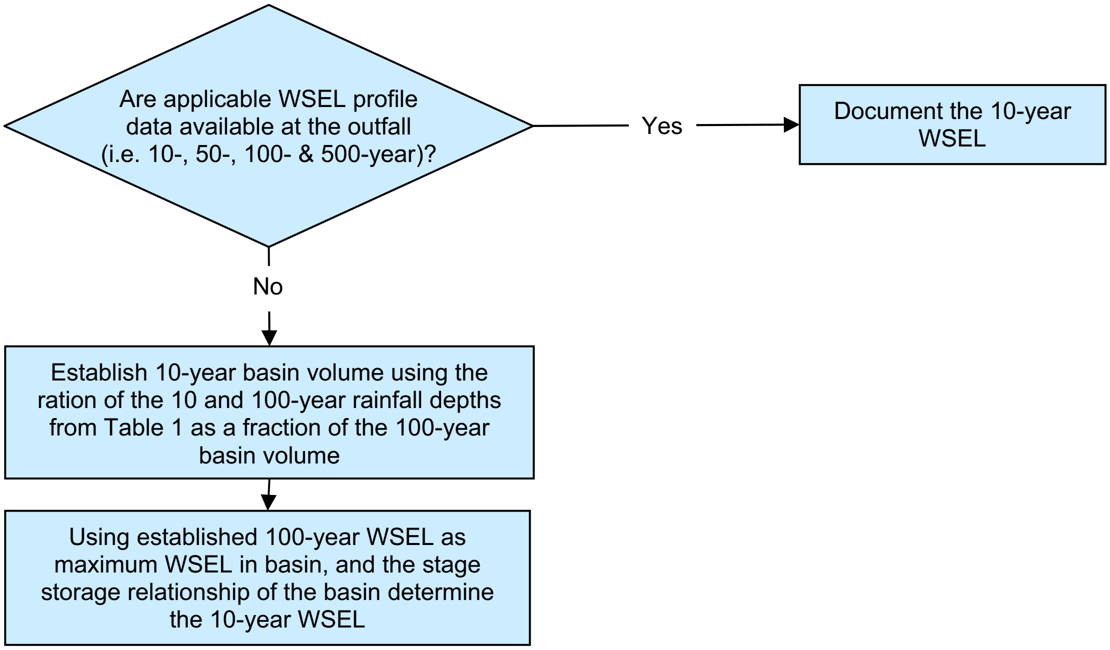

Figure 4 – Decision Flow Chart for Determining the 10-Year WSEL in a Detention Basin

Where detention basins are utilized, the 100-year WSEL in the basin is known or established as a design parameter based upon basin sizing as well as a stage storage relationship for the basin. While other frequency storm event WSELs are unknown without performing storm water routing through the basin, the 10-year WSEL can then be estimated for the basin utilizing ratio of the 10-year runoff to the 100-year runoff as calculated using Table 1 and multiplied to the 100-year storage volume of the basin to get an estimate of the 10-year basin storage volume. This volume is than applied to the stage storage relationship of the basin to get the 10-year WSEL.

For establishing the 10-year WSEL in outfall channels where suitable data does not exit, another method can be used to estimate the WSEL based on a relationship between the 100-year and 2-year WSELs. In order to better understand the *typical* relationship between the 2- and 100-year WSELs throughout Harris County, several bayous and creeks were examined based upon level-of-service analyses previously performed on these channels[6]. In these previous studies, WSELs for various storm events and associated flow rates were determined in the channels. A 10-year flow rate was used to compute the 10-year WSELs. From these test runs of numerous channels throughout Harris County, it was determined that the 10-year WSEL exists at approximately a 48% level between the 2 and 100-year WSELs. In other words, taking 48% of the difference between the 2 and 100-year WSELs will yield a reasonable approximation of the 10-year WSEL.

As with the 2-year HGL computation, drops in storm sewers can prove problematic as partial flow in the conduits may exist towards the outfall. This same situation applies if the 10-year WSEL determined for a project location is below the soffit of the outfall conduit. In these cases, the soffit of the outfall conduit should be used as the starting WSEL for the 100-year HGL computation similarly as with the 2-year HGL computation. The upstream conduit soffit at the drop should be used as the starting WSEL for upstream HGL computations if the HGL at the drop location is below the said conduit soffit. This typically eliminates the computation of partial flow within any conduits.

In any case, the position of the 10-year WSEL will have a direct correlation to the adjustment, if needed, of the 2-year storm sewer design as described in previous sections. There are many applicable methods of determining the 10-year WSEL at a given outfall location. Where routing procedures are not commonly performed, the method described in this section will prove suitable in most applications. Should situations dictate the utilization of a lesser event starting WSEL for the examination of the position of the 100-year HGL within a given project as described at the beginning of this section, due documentation as outlined in the City Design Manual is suitable in support of the design engineer's judgment in these regards.

6.0 Consideration of Overland Flow Paths

Despite the best design employed for a given storm sewer system, in typical situations, a given extreme storm event will render the storm sewer ineffective due to high tail water conditions. As such, the proper consideration of the overland flow path to the project outfall or outlet is critical

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[6] These level-of-service analyses were previously performed in support of the Harris County Watershed Master Plan, Harris County Flood Control District (HCFCD), 2004.

in terms of flood protection to the project area. In essence, this is primarily insured by the design of the roadway profile of a given project in a cascading manner to the project storm sewer outfall or outlet. In some cases, the storm sewer may not track readily with the overland flow path(s) which is not necessarily a negative as long as the proper consideration of overland flow in relation to conduit flow is understood and the design accommodates this condition. While the roadway serves as the primary overland flow mechanism, it is also critical to consider the means by which overland flow will be routed from the roadway to an outlet such as a detention basin within a storm sewer easement or other such allocated pathway. Conveyance links (i.e. large swales, ditches, etc.) are needed to provide a suitable pathway for anticipated floodwaters to the outlet. All projects shall comply with overland flow requirements defined in Section 9.05.D - Consideration of Overland Flow for the Extreme Event. The following are minimum requirements and work in conjunction with Section 13 of the Harris County Flood Control District's Policy Criteria & Procedure Manual[7] (exhibits in this section of the HCFCD Manual are applicable):

1) Streets shall be designed such as to accommodate overland flow in a cascading manner to the outfall (i.e. detention pond, channel, etc.).

2) Overland flow from the street to the outfall shall be provided via an overland flow swale within a minimum 20-foot dedicated drainage easement.

3) The actual sizing of the overland flow swale shall be based upon that component of overland flow, Q_O, as determined from the methods described in this Technical Paper and shall be minimally sized as based upon referenced Harris County Flood Control District Criteria as follows:

    - 2 foot minimum bottom width
    - 4 foot maximum depth
    - Minimum depth of 1 foot
    - side slopes 4(H):1(V) or flatter
    - lined with concrete, grass covered riprap, or articulated concrete block

4) The construction of fences, walls, and other improvements across drainage easements are prohibited.

Caution should be used in new developments where houses, buildings, fences, and other structures would block or impede these floodwaters from proceeding along their intended pathway.

Another overland flow consideration that must be addressed is the flow from a localized outfall or outlet such as a detention basin to an ultimate outfall at the watershed level. In other words, the designer must consider the scenario of overland flow in terms of leaving the project area via a spillway, within a roadway section, or another travel pathway towards the ultimate receiving

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[7] Harris County Flood Control District, Policy Criteria and Procedure Manual for Approval and Acceptance of Infrastructure, October 2004

channel(s) within the watershed. This should be documented in the design via flow arrows on the drainage area map or by other means. The purpose is to demonstrate the overland flow pathway(s) given the condition that the storm sewer system in conjunction with the detention basin, if applicable, for a project area have been completely inundated and rendered incapacitated in terms of facilitating additional inflow.

Given that the 100-year WSEL in a given roadway section is maintained at or below the MPE as previously described, the roadway profile itself must be designed in such a fashion to provide a reasonable surface pathway to the project outfall or outlet. Once completed, an easy check can be made of the resulting hydraulic profile, as required in Chapter 9 of the City Design Manual. Figure 6 illustrates a hydraulic profile of an example project where the storm sewer as designed for the 2-year event maintains the 100-year HGL below natural ground elevations at the ROW using the 10-year starting WSEL determined as described above. In this case, no adjustment to any conduit sizing was required. What is problematic, however, is the roadway profile relative to the detention basin (labeled *Pond*). This basin is intended to be the receiving outlet for overland flow; yet, the roadway profile does not adequately accommodate this. Also, not visible in the profile plot, the dedicated storm sewer easement containing the last short reach of storm sewer into the basin is not designed to accommodate the incoming overland flow at this location.

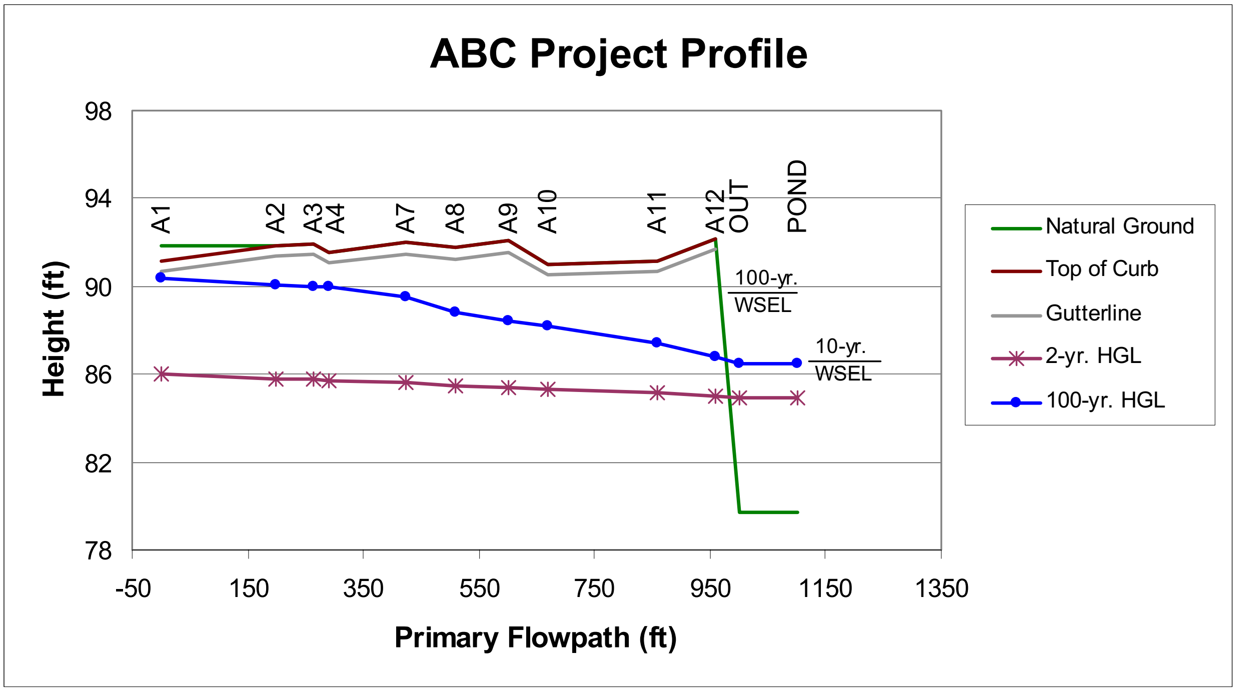


Figure 6 – Example ABC Project Hydraulic Profile

Figure 7 illustrates another example project hydraulic profile. In this case, the 2-year design HGL is suitable in terms of its position relative to the gutter line as required in the City Design Manual, but the 100-year HGL was above natural ground through much of the project area (labeled as *100-year HGL*) indicating overland flow in conjunction with storage will certainly be an integral part of the overall drainage system given a 100-year storm event. In this case, in lieu of computing the effects of storage and the routing of these overland flows, a few reaches of storm sewer where high levels of head loss were evident were increased one size only. The result is viewed in the HGL plot labeled *100-year Altered*. This proved to be an easily applied economical solution which provided the increased level-of-service desired. As with the previous example, notice the problematic design with respect to the overland flow path to the basin or *Pond* which is indicated as the receiving outlet for overland flow.

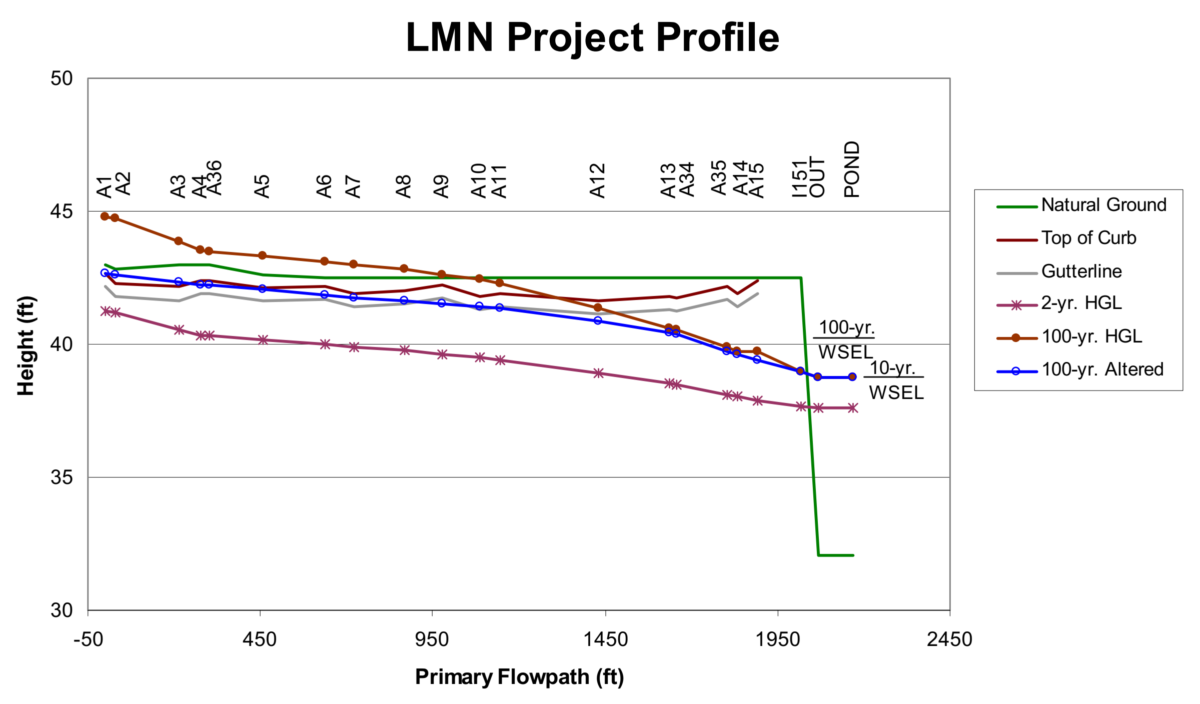


Figure 7 – Example LMN Project Hydraulic Profile

Lastly, Figure 8 illustrates an example project hydraulic profile where the 2-year design facilitated the position of the 100-year HGL relative to the natural ground elevations within the project area. Notice the evident drop in the storm sewer near the outlet as viewed by the sudden drop in the 2-year HGL. Also notice that the 10-year starting WSEL for the 100-year HGL plot was slightly above the soffit of the upstream conduit at the drop as indicated by an ever slightly higher position of the 100-year HGL at the drop. Had the 10-year starting WSEL been below this elevation, it would have been necessary to raise the said 10-year WSEL to match the upstream conduit soffit at the drop. While not clearly visible, this roadway profile suitably accommodates the overland flow pathway to the designated overland flow outlet, again in this case the detention basin or *Pond.*

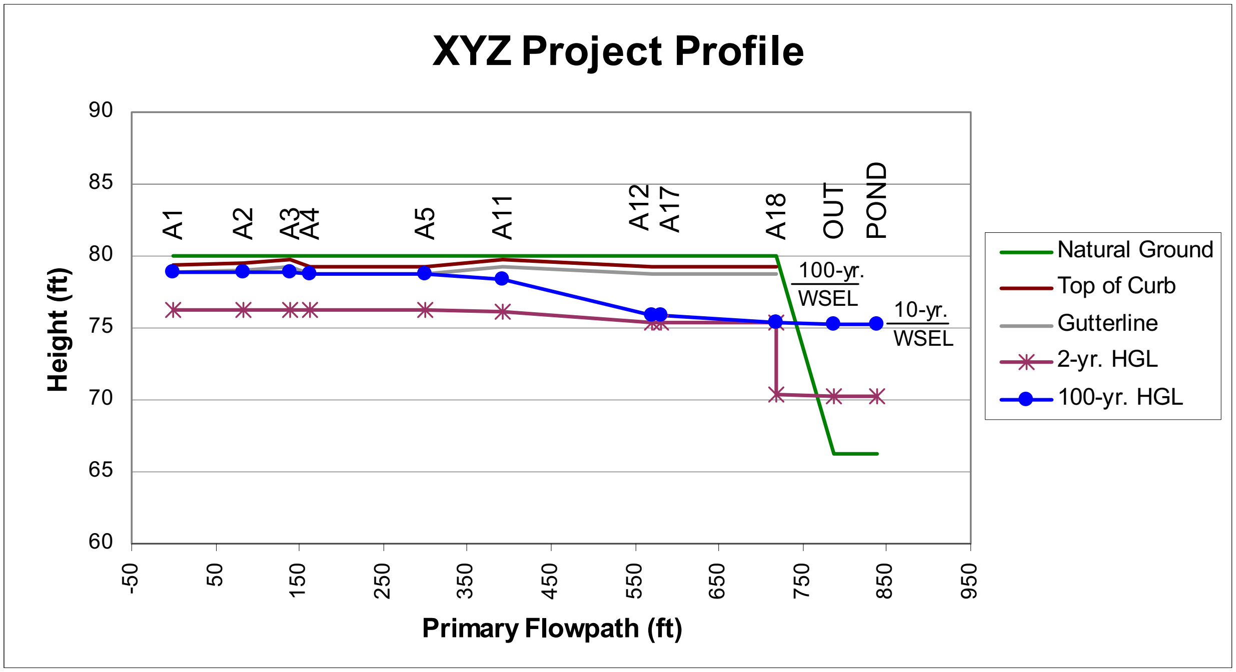
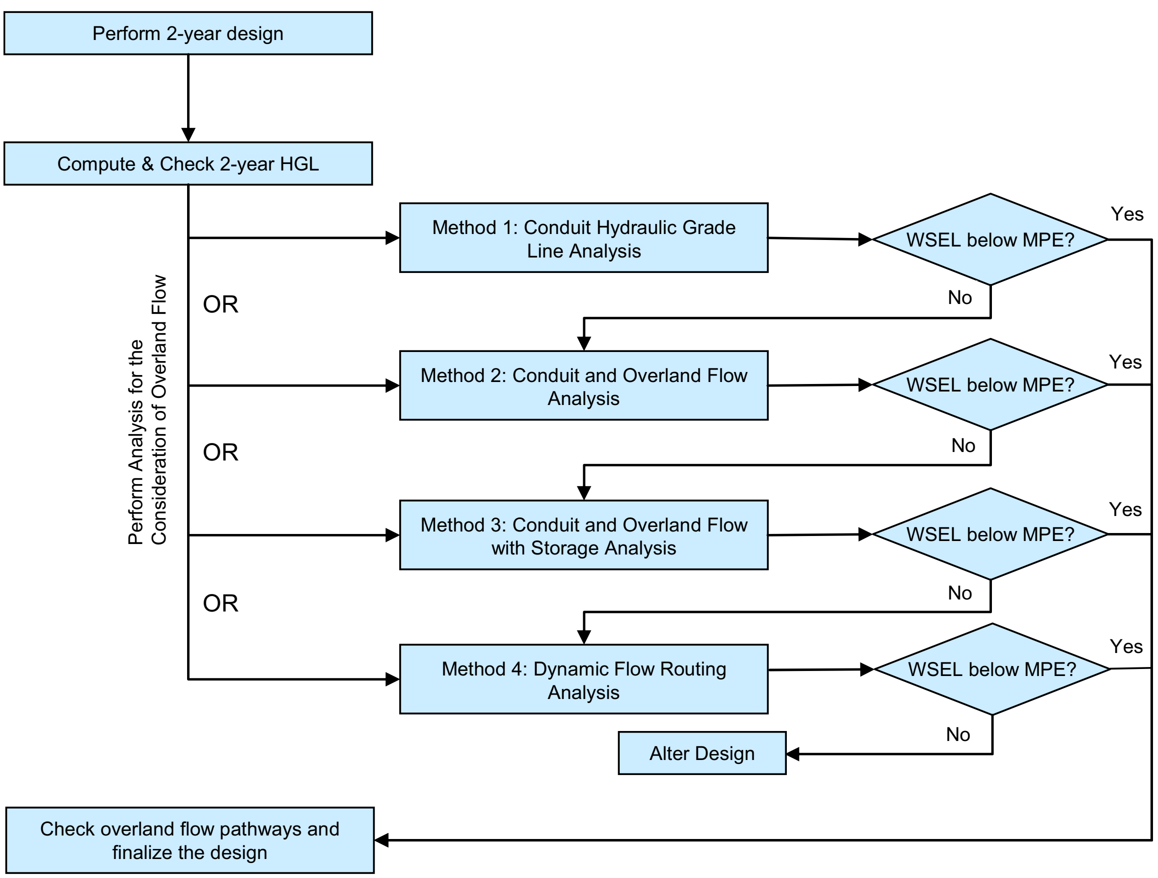


Figure 8 – Example XYZ Project Hydraulic Profile

7.0 Design Process Flowchart

Figure 9 is a design process flowchart depicting an overview of the process discussed herein. Again, there are many variations of this application and simplifications of the actual performance of the system are assumed. This is primarily true in cases of inlet, lead, and manhole behavior as applied to the overall system performance.



Notes:

WSEL = Water Surface Elevation

MPE = Maximum Ponding Elevation

Figure 9 – Design Process Flow Chart

8.0 Summary

Given the repetitive nature of severe storm events and the associated flooding which commonly occurs in the Houston region, it is desired to have an increased level-of-service in terms of flood level reduction. To achieve this, several changes to Chapter 9 of the City of Houston Design Manual have been initiated. The specific criterion requires 100-year water surface elevations (WSELs) to be maintained below the Maximum Ponding Elevation (MPE).

This technical paper outlines four methods that can be used to design storm water infrastructure considering the extreme event overland flow criteria of Chapter 9. The first method involves the simple adjustment of the 2-year storm sewer design (if needed at all) to reduce the head loss identified in certain reaches of the storm sewer system such that the resulting pressure head represented by the 100-year HGL is lowered to a desired level. Based upon several test case studies, this methodology provides an easily applied solution to the design requirement without significantly affecting, if at all, the initial storm sewer sizing. The second method involves the computation of overland flows to demonstrate maintaining the 100-year WSELs within a project area below the MPE. The third method adds to the complexity of the second method by considering storage in the overland flow analysis. Finally, the fourth method utilizes more complex computer models that simulate the actual dynamic behavior of the system's flow routing and may also be employed if deemed warranted. An example spreadsheet that may be utilized for Methods 1-3 is shown on the following page.

In all projects, overland flow pathways are critical to the performance of the entire storm sewer system and the behavior of the overland flow pathways to the receiving outfall or outlet must be carefully scrutinized in terms of anticipated performance.

Overland Flow Computations

Drainage Area ID	Analysis Node		Storm Sewer					Drainage Area		Runoff Coefficient 'C'		
	From	To	Size	Length	Cross Sectional Area	Hydraulic Radius	'n' Value	Incremental	Cumulative	Incremental	Cumulative CA	Composite
				(ft)	(sq ft)	(ft)		(ac)	(ac)			
A	1	2	24" RCP	100	3.14	0.50	0.013	6.00	6.00	0.40	2.40	0.40
B	2	3	30" RCP	120	4.91	0.63	0.013	3.00	9.00	0.30	3.30	0.37
C	3	4	36" RCP	140	7.07	0.75	0.013	4.00	13.00	0.20	4.10	0.32
D	4	5	42" RCP	150	9.62	0.88	0.013	5.00	18.00	0.60	7.10	0.39
E	5	6	48" RCP	120	12.57	1.00	0.013	6.00	24.00	0.70	11.30	0.47
F	6	7	48" RCP	140	12.57	1.00	0.013	2.50	26.50	0.55	12.68	0.48
Outlet	7	Out	48" RCP	100	12.57	1.00	0.013		26.50			

→ Continued Below for Clarity

Drainage Area ID	Time of Concentration			Intensity		Peak Flow		Maximum Ponding Elevation (MPE)	Maximum Allowable Overland Flow (Q_{Oallow})	Available Overland Storage Volume (V_{Savail})		Method 1			Method 2		Method 3				
	per Drainage Area	Travel Time	Maximum Time	2-year	100-year	2-year (Q_C)	100-year (Q_T)			Incremental	Cumulative	Head loss per segment	Hydraulic Grade Line (HGL)	**CHECK Is MPE >=HGL?**	Required Overland Flow (Q_{Oreqd})	**CHECK Is Q_{Oallow} >= Q_{Oreqd}?**	Percent Impervious	Runoff Depth	Runoff Volume (V_T)	Required Overland Flow (Q_{Oreqd})	**CHECK Is Q_{Oallow} >= Q_{Oreqd}?**
	(min)	(min)	(min)	(in/hr)	(in/hr)	(cfs)	(cfs)	(ft)	(cfs)	(ac-ft)	(ac-ft)	(ft)	(ft)	Yes = Acceptable No = Unacceptable	(cfs)	Yes = Acceptable No = Unacceptable	%	(in)	(ac-ft)	(cfs)	Yes = Acceptable No = Unacceptable
A	30.00	-	30.00	3.10	6.49	7.43	15.59	102.20	12.0	0.5	0.5	0.47	101.80	Yes	8.15	Yes (Method 1)	33%	5.0	2.52	2.23	Yes (Method 1)
B	20.00	0.70	30.70	3.06	6.43	10.09	21.22	101.30	12.0	0.5	1.0	0.32	101.33	No	11.12	Yes	28%	4.9	3.66	0.06	Yes (Method 2)
C	25.00	0.97	31.68	3.01	6.34	12.33	26.00	101.00	14.0	0.5	1.5	0.21	101.02	No	13.67	Yes	19%	4.6	5.03	0.00	Yes (Method 2)
D	20.00	1.34	33.02	2.94	6.22	20.86	44.20	100.50	20.0	0.4	1.9	0.29	100.80	No	23.33	No	32%	5.0	7.51	2.15	Yes
E	45.00	1.15	45.00	2.45	5.37	27.70	60.64	100.50	24.0	0.4	2.3	0.21	100.52	No	32.94	No	45%	5.4	10.73	9.01	Yes
F	20.00	0.91	45.91	2.42	5.31	30.70	67.34	100.50	24.0	0.6	2.9	0.31	100.31	Yes	36.64	Yes (Method 1)	46%	5.4	11.92	6.54	Yes (Method 1)
Outlet													100.00								
	Inlet Tc as Determined by engineer.	Travel Time (from previous storm sewer reach) = Length/Velocity or determined by engineer.	Maximum Time is either TC of the drainage area or the previous time plus the travel time, which ever is greater.	Intensity = b/(Tc + d)^e Rainfall Intensity 2-Year b 75.01 d 16.2 e 0.8315 100-Year b 125.4 d 21.8 e 0.75		Peak Flow = CAI		Determined by engineer in accordance with City of Houston Design Manual, Chapter 9, Section 9.05.D.4.g. Maximum Ponding Elevation.	Determined by engineer, based on Manning's Equation applied to overland flow path geometry.	Determined by engineer based on average end area, digital topographic, or similar method.		Head loss = $L(Qn/(1.49AR^{2/3}))^2$ Computed for 100-year flows	Hydraulic grade line for 100-year starts at 10-year tailwater elevation at the Outlet. Tailwater elevation determined by engineer.	If **"Yes"**, design for storm sewer segment meets City of Houston requirements. If **"No"**, design for storm sewer segment does not meet City of Houston requirements and adjustment in storm sewer size or additional analysis are needed.	$Q_{Oreqd} = Q_T - Q_C$	If **"Yes"**, design for storm sewer segment meets City of Houston requirements. If **"No"**, design for storm sewer segment does not meet City of Houston requirements and adjustment in storm sewer size and/or overland flow capacity or additional analysis are needed.	Percent Impervious = ($C_{Composite}$ - 0.2)/0.6	Runoff Depth based on values listed below. Rainfall Depth (in) 3-Hr Duration 0.00% 4.10 40.00% 5.22 100.00% 6.90 6-Hr Duration 0% 5.60 40% 6.80 100% 8.60	V_T = Runoff Depth x A_{CUM}	$Q_{Oreqd} = Q_T - Q_C - ((V_{Savail} \times Q_T^2)/(V_T (Q_T - Q_C)))$ V_{Savail} and V_T are cumulative values.	If **"Yes"**, design for storm sewer segment meets City of Houston requirements. If **"No"**, design for storm sewer segment does not meet City of Houston requirements and adjustment in storm sewer size and/or overland flow capacity or additional analysis are needed.