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DOWNSTREAM EFFECTS OF CULVERT AND BRIDGE REPLACEMENT

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16 Abstract

The replacement of a culvert or bridge with a larger structure yields two benefits in all cases. The first benefit is less frequent flooding of the roadway and/or upstream structures due to lower headwater levels. The second benefit is a lesser potential for scour through the bridge opening or at the culvert outlet due to lower velocities through the larger opening. Downstream impacts, if any, are project-specific. This report presents a framework for evaluation of likely impacts on downstream flooding and channel erosion. Two methods for predicting changes in flood peaks are presented and demonstrated in examples. The first method, which requires flood hydrograph simulation and reservoir routing, is applicable to all cases. The second method, which does not require hydrograph simulation or routing, is applicable to culverts that operate under inlet control with no roadway overtopping.

The effect of detention storage on downstream sediment transport is investigated computationally. Our analysis shows that a reduction in detention storage results in an increase in the volume of sediment that the flood can transport. This increase in sediment transport capacity may lead to an increase in channel erosion downstream of the structure. However, reliable quantitative predictions of erosional impacts are not possible. If peak flows through an existing structure are unaffected by detention storage or split flow, enlargement of the structure will not increase erosion downstream.

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DOWNSTREAM EFFECTS OF CULVERT AND BRIDGE REPLACEMENT

Final Report

Prepared by

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THE KANSAS DEPARTMENT OF TRANSPORTATION
TOPEKA, KANSAS

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PREFACE

The Kansas Department of Transportation's (KDOT) Kansas Transportation Research and New-Developments (K-TRAN) Research Program funded this research project. It is an ongoing, cooperative and comprehensive research program addressing transportation needs of the state of Kansas utilizing academic and research resources from KDOT, Kansas State University and the University of Kansas. Transportation professionals in KDOT and the universities jointly develop the projects included in the research program.

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ABSTRACT

The replacement of a culvert or bridge with a larger structure yields two benefits in all cases. The first benefit is less frequent flooding of the roadway and/or upstream structures due to lower headwater levels. The second benefit is a lesser potential for scour through the bridge opening or at the culvert outlet due to lower velocities through the larger opening. Downstream impacts, if any, are project-specific. This report presents a framework for evaluation of likely impacts on downstream flooding and channel erosion. Two methods for predicting changes in flood peaks are presented and demonstrated in examples. The first method, which requires flood hydrograph simulation and reservoir routing, is applicable to all cases. The second method, which does not require hydrograph simulation or routing, is applicable to culverts that operate under inlet control with no roadway overtopping.

If the roadway over the existing structure is overtopped by floods, enlargement of the structure will increase the flow through the structure and decrease or eliminate the roadway overflow. However, the peak flow in the channel directly downstream of the structure will not necessarily change. If the stream crossing includes a relief structure located some distance from the main structure, or if roadway overtopping occurs at some distance from the main structure, split flow can occur for a short distance downstream of the crossing. Enlargement of the main structure will increase the flow through the main structure and reduce or eliminate the split flow.

If peak flows through the existing structure are affected by detention storage, enlargement of the structure will increase the peak flows. The peak flows through the enlarged structure will also occur sooner, which may be significant in an analysis of downstream flooding. The increase in peak flow, if any, diminishes with distance downstream from the enlarged

structure due channel and overbank storage and lateral inflows. The new streamflow characteristics will more closely resemble the natural conditions that existed before the highway was constructed. If the peak flows through the existing structure are unaffected by detention storage or split flow, enlargement of the structure will not increase the peak flows directly downstream. Few culverts and even fewer bridges are affected significantly by detention storage.

The effect of detention storage on downstream sediment transport is investigated computationally. Our analysis shows that a reduction in detention storage results in an increase in the volume of sediment that the flood can transport. This increase in sediment transport capacity may lead to an increase in channel erosion downstream of the structure. However, reliable quantitative predictions of erosional impacts are not possible. If peak flows through an existing structure are unaffected by detention storage or split flow, enlargement of the structure will not increase erosion downstream.

ACKNOWLEDGMENTS

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CHAPTER 1

INTRODUCTION

1.1 Motivation and Scope

This report addresses a concern that has been raised in connection with some culvert and bridge replacement projects of KDOT and local governments. This concern is a possible increase in downstream flooding and erosion resulting from enlargement of the waterway opening.

This report explains how to evaluate the likely downstream effects of culvert and bridge replacement projects where an evaluation is warranted. Chapter 2 presents a general procedure for predicting the effects on downstream peak flows. This procedure is applicable to all cases. Chapter 3 presents a simplified method for analyzing culverts that operate under inlet control with no overtopping. The simplified method does not require hydrograph routing. Chapter 4 examines the effects on downstream sediment transport and channel erosion.

1.2 General Principles

A larger waterway opening yields two benefits in all cases. The first benefit is less frequent flooding of the roadway and/or upstream structures due to lower headwater levels. The second benefit is a lesser potential for scour through the bridge opening or at the culvert outlet due to lower velocities through the larger opening. Downstream impacts, if any, are project-specific. Enlargement of the waterway opening might or might not result in higher peak flows and more channel erosion downstream of the structure. If the peak flows through the existing structure are affected by detention storage, enlargement of the structure will increase the peak flows and might also increase channel erosion. The peak flow through the enlarged structure will also occur sooner, which may be significant in an analysis of downstream impacts. The new streamflow characteristics will more closely resemble the natural conditions that existed before the highway was constructed. Few culverts and even fewer bridges are affected significantly by detention storage.

Crossings of streams with wide floodplains may include a relief structure located some distance from the main structure. In a flood, the relief structure conveys part of the total flow. The flow through the relief structure would rejoin the main stream somewhere downstream, usually a short distance. A similar situation can occur on a highway without a relief structure if the roadway is overtopped during floods and the low point on the roadway profile is some distance from the bridge or culvert. In both cases the flow is split between the main channel and the overflow path for some distance downstream of the highway. In these situations, enlargement of the main structure increases the flow through the main structure and reduces or eliminates the split flow. If the peak flows through the existing structure are unaffected by detention storage or split flow, enlargement of the structure will not cause any changes in peak flows or channel erosion.

In an analysis of downstream effects, the location of greatest concern might be the next culvert or bridge downstream of the enlarged structure. The increase in peak flow at the next structure downstream is generally smaller than the increase in the peak flow through the enlarged structure due to the channel and overbank storage and lateral inflows.

CHAPTER 2

ANALYSIS OF PEAK FLOWS

2.1 General Procedure

This chapter presents a procedure for analyzing the downstream flood impacts of culvert or bridge replacement where an analysis is warranted. The objective is to estimate the changes in the peak discharge and water-surface elevation at one or more locations downstream of the highway. The recurrence interval for the analysis is the one that applies to the design of the replacement structure.

The first task is to analyze the existing structure for detention storage and overtopping. A detention-storage analysis for a culvert or bridge requires flood-hydrograph simulation with reservoir routing. The recommended computer program for flood-hydrograph simulation is the HEC-HMS Hydrologic Modeling System of the U. S. Army Corps of Engineers (USACE, 2000). The steps are as follows.

1. Compute the design-flood hydrograph for the structure (the inflow hydrograph to the storage zone) as directed in the KDOT Design Manual, Volume I, Section 11.4.
2. Develop a stage-area relationship for the storage zone as directed in Section 2.2.
3. Develop a stage-discharge relationship for the stream crossing as directed in Section 2.3. The stage-discharge relationship must account for roadway overtopping and flow through any relief structures as well as the flow through the main structure.
4. Route the design-flood hydrograph through the storage zone to determine the peak flows through the main structure, through any relief structures and over the roadway.

If the peak outflow (the flow through the main structure and any relief structures plus any flow over the roadway) is not significantly less than the peak inflow and split flow does not occur, enlargement of the existing structure will not increase the peak flow. In this case, no further analysis is needed.

If the peak flow through the existing structure is affected by detention storage or split flow, then the proposed structure should also be analyzed for detention storage and split flow. The design-flood hydrograph is the same as for the existing structure. Develop a stage-discharge relationship for the stream crossing with the proposed structure. If the proposed project would alter the existing stage-area relationship, also develop a new stage-area relationship. Route the design-flood hydrograph through the storage zone to determine the peak flows through the main structure, through any relief structures and over the roadway.

If the peak outflow with the proposed structure is significantly greater than the peak outflow with the existing structure, increases in downstream flooding and erosion are possible and further analysis may be warranted.

If split flow does not occur for existing conditions and the peak outflow with the proposed structure is not significantly greater than the peak outflow with the existing structure, the proposed project would not increase downstream flooding or erosion and no further analysis is needed.

If split flow would occur with the existing structure, then enlargement of this structure will eliminate or reduce the split flow and increase the peak flow in channel within the reach currently affected by split flow. The reach affected by split flow is usually short.

2.2 Stage-Area Relationship for Storage Zone

The stage-area relationship for the storage zone must be estimated. Stage-area data can be extracted from a sufficiently detailed topographic map or from topographic survey data. A special topographic survey may be necessary for this purpose.

Small storage areas upstream of culverts can usually be measured with sufficient accuracy by one person with a rotating laser level and sensor, a level rod, some survey flags and a GPS unit. The GPS unit should be capable of calculating the area of a polygon. The following procedure yields the area at a given stage.

1. Set up the rotating laser level a few feet above the desired stage. Locate the laser level so that the laser beam has an unobstructed line to a sufficient number of points on the perimeter of the area to be measured.
2. Set the level rod at the flowline of the culvert entrance and position the sensor on the rod at the level of the laser beam. If the laser level is too high above the flowline, measure the height to the top of the pipe or headwall, then set the rod on top of the pipe or headwall and position the sensor at the level of the laser beam. Determine the height of the laser beam above the flowline by reading the height of the sensor on the rod and, if necessary, adding the height of the rod above the flowline. Reposition the sensor on rod so that the height of the sensor above the base of the rod equals the height of the laser level above the desired headwater level. For example, if the laser beam is 8.52 ft above the flowline and the desired headwater level is 4.00 ft, the sensor should be set at 4.52 feet (8.52 ft minus 4.00 ft) on the rod.
3. Use the level rod with the sensor to identify points on the perimeter of the area that would be inundated at the specified headwater level. Mark these points with survey flags. Mark enough points to adequately define the boundary.
4. Use the GPS unit to measure the area within the perimeter marked by the survey flags. The procedure varies with the make and model of the GPS unit.

2.3 Stage-Discharge Relationships

2.3.1 Culverts

Stage-discharge relationships for culverts should be developed with the Federal Highway Administration's computer program HY-8 or a similar program based on Hydraulic Design

Series No. 5, Hydraulic Design of Highway Culverts (FHWA, 1985). HY-8 can account for flow through multiple non-identical culverts (e.g., the main structure and one or more relief structures) and roadway overtopping.

KDOT's standard end sections for pipes are not identical to any of the end treatments in FHWA's HDS No. 5. Table 2-1 shows the end treatments in HDS No. 5 and HY-8 that are hydraulically similar to KDOT's standard end sections. Culverts should be analyzed using the values of Manning's roughness coefficient recommended in the KDOT Design Manual, Volume I, Section 14.3.

Table 2-1: Hydraulic Characteristics of KDOT End Sections for Pipes

KDOT end section	Hydraulically similar end treatment in HDS No. 5 and HY-8
Concrete Type I	RCP with groove end projecting
Metal Type I	CMP with headwall
Concrete Type III	RCP with side-tapered inlet
Metal Type III	CMP with side-tapered inlet
Type IV	CMP mitered to slope

HY-8 and similar programs require information on tailwater conditions. If the tailwater level would not be affected significantly by backwater from any downstream feature, a uniform flow condition can be assumed. HY-8 will compute a tailwater rating curve (stage-discharge relationship) for uniform flow. The required inputs are the slope of the channel bottom, a representative cross-section that includes the channel and the left and right overbanks, and Manning's roughness coefficients for the channel and overbanks. If backwater effects would be significant, the tailwater rating curve must be developed in a separate analysis outside of HY-8. The analysis of backwater effects requires calculation of water-surface profiles in the downstream channel for a range of discharges, using the HEC-RAS River Analysis System of the U.S. Army Corps of Engineers (USACE, 2002) or a similar program.

2.3.2 Bridges

The development of a stage-discharge relationship for a bridge requires an analysis of the flow in the downstream channel as well as the flow through the bridge opening. Water-surface profiles are computed for a range of discharges using HEC-RAS or a similar program. The required inputs include (1) cross-sections of the channel and overbanks at multiple locations downstream and directly upstream of the bridge, (2) Manning roughness coefficients for the channel and overbank regions of each cross section, (3) detailed information on the bridge and roadway. The *HEC-RAS User's Manual, Hydraulic Reference Manual* and *Applications Guide* (USACE, 2002) provide complete guidance.

2.4 Flooding Effects Farther Downstream

The location of greatest concern for increased flooding may be the next structure downstream. In this case, the analysis outlined in Section 2.1 can be extended to the downstream structure to estimate the change in the flood stage at this structure. The required peak flows are computed by adding the following operations to the hydrologic models for the existing and proposed conditions.

1. Route the flood hydrograph through the reach of channel from the structure to be enlarged to the downstream structure.
2. Compute the local runoff hydrograph from the sub-basin between the two structures.
3. Combine the hydrograph from the channel routing and the hydrograph from the local subbasin.
4. If detention storage at the downstream structure could be significant, route the combined hydrograph through the storage zone to obtain the peak flow at the downstream

structure, including any overflow. If the detention effect at the downstream structure is negligible, the peak flow through the downstream structure is the peak flow on the combined hydrograph from step 3.

Channel routing accounts for the attenuation of the flood peak caused by temporary storage of floodwater in the channel and the overbanks. The preferred method of channel routing is the Muskingum-Cunge method, an approximate hydraulic method available within HEC-HMS. The required inputs are the length and average slope of the channel reach, a typical cross-section that includes the channel and overbanks, and Manning's roughness coefficients for the channel and overbanks.

2.5 Example 1: Pipe-Culvert Replacement

Problem

A 60-in. RCP culvert in rural Shawnee County causes excessive backwater during high flows. The applicable recurrence interval is 25 years. The 25-year flood does not overtop the roadway with the existing culvert. A proposed project would replace the existing culvert with a double 60-inch RCP installation to reduce the headwater level.

The objective is to determine the impact of the proposed project on the 25-year headwater level and the 25-year discharge for this structure.

The watershed upstream of the culvert has a drainage area of 98.8 acres, a lag time of 15 minutes and a runoff curve number of 80.

The following information applies to both the existing and proposed culverts:

Allowable headwater elevation = 957.00 ft
Roadway overtopping elevation = 960.00 ft
Flowline elevation at entrance = 951.77 ft
Flowline elevation at exit = 950.95 ft
Length = 138 ft
End sections = concrete Type I

Tailwater condition = uniform flow in trapezoidal channel with 6-ft depth, 10-ft bottom width, 1:1 side slopes, bottom slope of 0.006 ft/ft and Manning's n of 0.035

Table 2-2 shows stage-area data for the storage zone on the upstream side of the culvert. These data are applicable to both the existing and proposed conditions.

Table 2-2: Stage-Area Data for Storage Zone

Headwater Elevation (ft)	Inundated Area (ac)
951.77	0
952	0.08
953	0.43
954	0.75
955	1.04
956	1.31
957	1.55
958	1.77
959	1.96
960	2.13

Solution

The first task is to analyze the existing structure for detention storage. The discharge-stage relationship for the existing culvert is computed with FHWA's HY-8 computer program. The concrete Type I end section is modeled as "RCP with the groove end projecting". Table 2-3 shows the results.

Table 2-3: Headwater Elevations at Selected Discharges for Existing Culvert

Culvert Discharge (cfs)	Headwater Elevation (ft)
0	951.77
25	953.57
50	954.52
75	955.25
100	955.87
125	956.46
150	957.06
175	957.73
200	958.49
225	959.35
250	960.33

Table 2-4 shows the discharges for the stages in Table 2-2, obtained by linear interpolation in Table 2-3.

Table 2-4: Stage-Discharge Relationship for Existing Culvert

Headwater Elevation (ft)	Discharge (cfs)
951.77	0
952	3
953	17
954	36
955	66
956	106
957	148
958	184
959	215
960	242

The flood hydrograph simulation and reservoir routing are performed with HEC-HMS, following the procedures in the KDOT Design Manual, Volume I, Section 11.4, using a computational time step of one minute. The key results are as follows:

Peak inflow to storage zone = 363 cfs
Peak flow through culvert = 208 cfs
Peak stage = 958.77 ft
Peak storage volume = 4.22 ac-ft
Peak inundated area = 1.92 ac (by interpolation in Table 2-2)

These results show that detention storage causes a 43% reduction in the 25-year flow through the existing culvert. Because the detention effect is substantial, enlargement of the culvert could cause a significant increase in discharge. A detention-storage analysis of the proposed culvert is needed to quantify this increase.

By adding a second 60-inch pipe identical to the existing pipe, the proposed project would double the discharge at any headwater level. Table 2-5 shows the stage-discharge relationship for the proposed culvert.

Table 2-5: Stage-Discharge Relationship for Proposed Culvert

Headwater Elevation (ft)	Discharge (cfs)
951.77	0
952	6
953	34
954	72
955	132
956	212
957	296
958	368
959	430
960	484

The HEC-HMS simulation for the proposed culvert is the same as for the existing culvert, except that the stage-discharge data in Table 2-5 are used for the reservoir routing. Figure 2-1 compares the hydrographs from the two simulations. Table 2-6 compares the results for the existing and proposed culverts.

Table 2-6: Comparison of Results for Existing and Proposed Culverts

Results for 25-year event	Existing culvert	Proposed culvert
Peak flow through culvert	208 cfs	291 cfs
Peak headwater elevation	958.77 ft	956.94 ft
Peak storage volume	7.41 ac-ft	4.22 ac-ft
Peak inundated area	1.92 ac	1.54 ac

The proposed project would achieve the goal of reducing the 25-year headwater level below the allowable level of 957.00 ft. The peak headwater depth would be reduced by 26% (from 7.00 ft to 5.17 ft) and the peak storage volume would be reduced by 43%. However, the 25-year discharge through the culvert would increase by 40%, which could increase downstream flooding. The changes in the downstream flood levels could be estimated by the procedure recommended in Section 2.4.

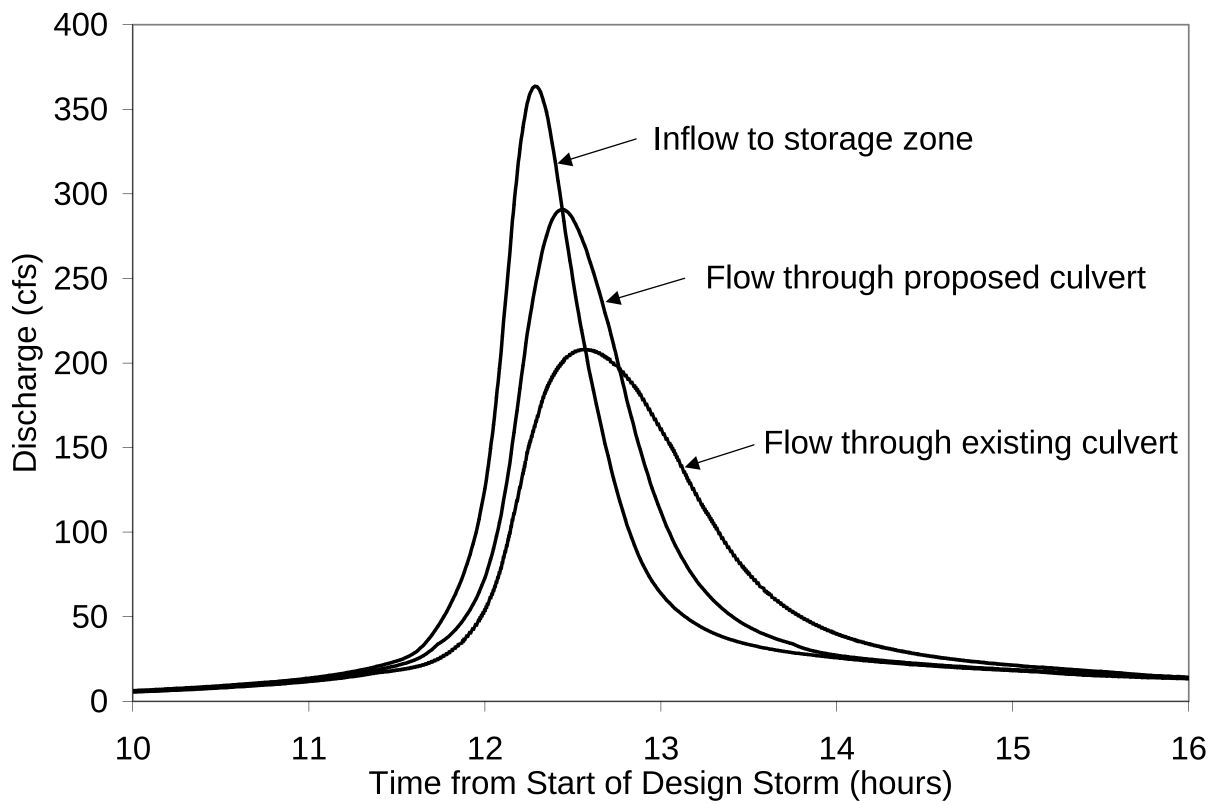

Figure 2-1: 25-Year Flood Hydrographs for Existing and Proposed Culverts

2.6 Example 2: Box-Culvert Replacement

Problem

A highway in rural Shawnee County is occasionally overtopped by flooding. The existing culvert is an 8 ft x 6 ft RCB. A proposed project would replace the existing culvert with an RCB with two 10 ft x 6 ft cells. The applicable recurrence interval is 25 years.

The objective is to determine the impact of the proposed project on the 25-year headwater level and discharge.

The watershed upstream of the culvert has a drainage area of 384 acres, a lag time of 30 minutes and a runoff curve number of 79.

The following information applies to both the existing and proposed culverts:

Allowable headwater elevation = 961.00 ft
Flowline elevation at entrance = 954.20 ft
Flowline elevation at exit = 953.30 ft
Length = 90 ft
End treatments = 45º wingwalls, top edge of inlet beveled
Tailwater condition = uniform flow in trapezoidal channel with 6-ft depth, 20-ft bottom width, 1:1 side slopes, a bottom slope of 0.004 ft/ft and a Manning's n of 0.035

The roadway profile over the culvert is a 400-ft vertical curve with an initial grade of –3.0% and a final grade of +3.0%. The elevation of the low point on the vertical curve is 963.40 ft. The low point on the vertical curve is over the culvert.

Table 2-7 shows stage-area data for the storage zone on the upstream side of the culvert. These data are applicable to both the existing and proposed conditions.

<u>Table 2-7: Stage-Area Data for Storage Zone</u>

Headwater Elevation (ft)	Inundated Area (ac)
954.20	0
955	0.01
956	0.05
957	0.12
958	0.22
959	0.35
960	0.50
961	0.69
962	0.91
963	1.16
964	1.44
964.75	1.67

<u>Solution</u>

The first task is to analyze the existing structure for detention storage. The discharge-stage relationship for the existing culvert is computed with FHWA's HY-8 computer program. Table 2-8 shows the headwater elevation, the flow through the culvert and the flow over the roadway for each total discharge.

Table 2-8: Performance of Existing Culvert at Selected Discharges

Total Discharge (cfs)	Headwater Elevation (ft)	Discharge through Culvert (cfs)	Discharge over Roadway (cfs)
0	954.20	0	0
100	955.56	100	0
200	956.36	200	0
300	957.03	300	0
400	957.66	400	0
500	958.26	500	0
600	958.83	578	22
700	959.38	594	106
800	959.91	605	195
900	960.45	613	287
1000	961.00	620	380

Table 2-9 shows the stage-area-discharge table used for the reservoir routing. This table includes extra detail at stages over 963.4 ft, the low point on the roadway, for more accurate modeling of the roadway overtopping. The inundated areas and discharges are interpolated from Tables 2-7 and 2-8.

Table 2-9: Stage-Area-Discharge Relationship for Existing Culvert

Headwater Elevation (ft)	Inundated Area (ac)	Total Discharge (cfs)
954.20	0	0
955	0.01	32
956	0.05	72
957	0.12	119
958	0.22	183
959	0.35	253
960	0.50	326
961	0.69	400
962	0.91	470
963	1.16	547
963.40	1.27	580
963.70	1.35	618
963.95	1.43	711
964.15	1.49	829
964.30	1.53	938
964.38	1.55	1000

The flood hydrograph simulation and reservoir routing are performed with HEC-HMS, following the procedures in the KDOT Design Manual, Volume I, Section 11.4, using a computational time step of one minute. The key results are as follows:

Peak inflow to storage zone = 966 cfs
Peak outflow from storage zone = 963 cfs
Peak discharge through culvert = 617 cfs (by interpolation in Table 2-8)
Peak discharge over highway = 356 cfs (by interpolation in Table 2-8)
Peak stage = 964.33 ft
Peak storage = 5.20 ac-ft
Peak inundated area = 1.54 ac (by interpolation in Table 2-7)

Figure 2-2 compares the inflow and outflow hydrographs.

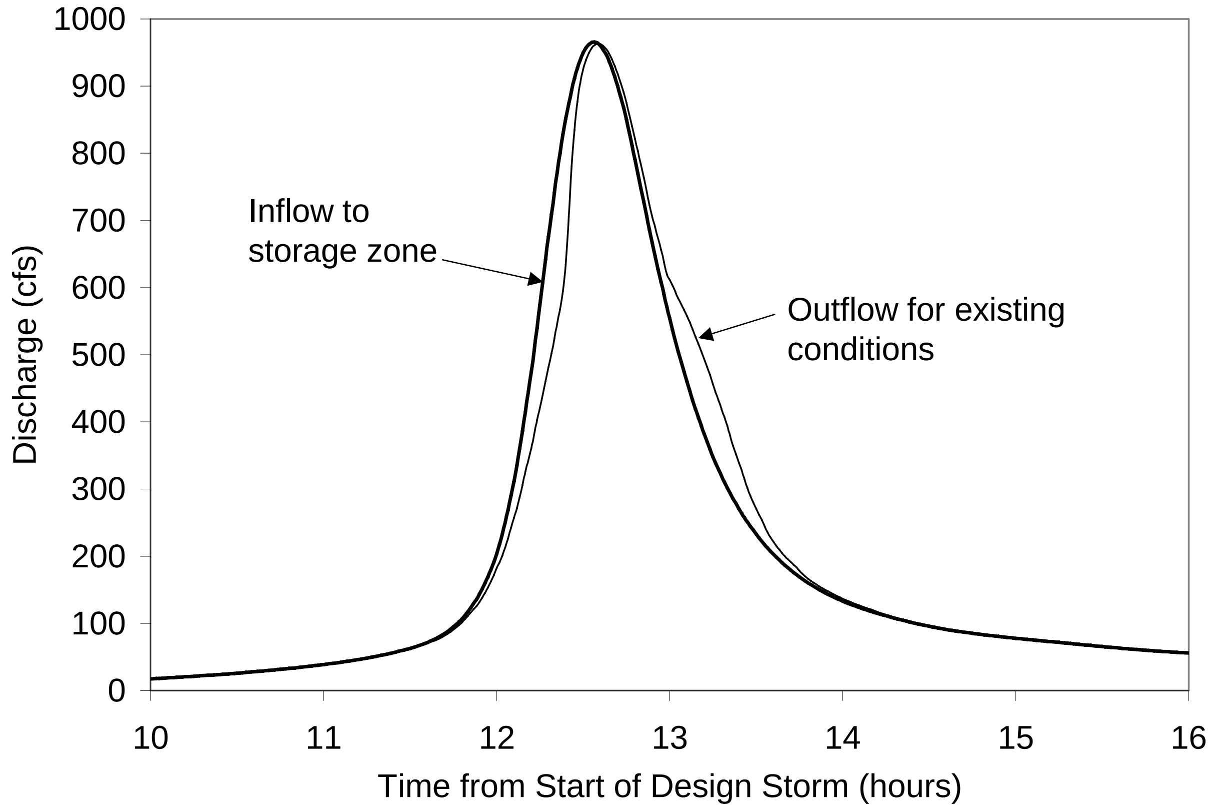

Figure 2-2: 25-Year Flood Hydrographs for Existing Culvert

These results show that detention storage reduces the 25-year discharge (culvert flow plus overtopping) by only 0.3%, an insignificant amount. Therefore, enlargement of the culvert would not increase the peak discharge, although it would eliminate the roadway flooding. Because the storage effect is negligible, the headwater elevation for the proposed culvert can be calculated for the peak inflow of 966 cfs, with no need for hydrograph simulation or reservoir routing. The 25-year headwater elevation for the proposed culvert is 960.81 feet, which is 0.91 feet below the allowable level and 3.45 feet below the current level.

CHAPTER 3

SIMPLIFIED ANALYSIS FOR CULVERTS

3.1 Overview of Simplified Method

Culverts that operate under inlet control with no overtopping can be analyzed by a simplified method developed in a previous K-TRAN research project (McEnroe and Gonzalez, 2004). This method, which does not require hydrograph routing, provides good estimates of storage effects on peak flows.

The simplified method can be used to assess the effects of culvert enlargement on downstream peak flows. The first step is to determine the extent to which detention storage affects the peak flow through the old culvert. If detention storage has a negligible effect on the peak flow through the old culvert, its effect on the peak flow through the new culvert will also be negligible, and no further analysis is needed. If the detention effect at the old culvert is significant, then the new culvert must also be analyzed. The peak discharges downstream of the new and old culverts can then be compared. Section 3.2 provides step-by-step procedure, and Section 3.3 demonstrates how the method is used to analyze a proposed culvert replacement. The development of this method is explained in a report by McEnroe and Gonzalez (2004).

3.2 Procedure

The simplified method yields an estimate of the peak discharge through the culvert downstream of the culvert, Q_p. The procedure is as follows.

1. Compute the peak discharge directly upstream of the culvert (the peak discharge for no storage effect), I_p, as directed in the KDOT Design Manual, Volume I, Section 11.

2. Compute the flood volume, V, with the appropriate regression equation from Table 3-1. Figure 3-1 defines the western and eastern hydrologic regions of Kansas.

3. Compute the time-to-peak, t_p, with equation 3-9.

$$t_p = 9.45 \frac{V}{I_p} \tag{3-9}$$

Table 3-1: Regression Equations for Volume of Design Flood

Hydrologic Region of Kansas	Recurrence Interval (years)	Equation	Standard Error (%)	Equation number
East	10	$V = 14.2\ W^{0.774}\ I_p^{0.332}$	11	3-1
East	25	$V = 48.2\ W^{0.851}\ I_p^{0.212}$	10	3-2
East	50	$V = 100.6\ W^{0.901}\ I_p^{0.135}$	9	3-3
East	100	$V = 192.8\ W^{0.774}\ I_p^{0.332}$	7	3-4
West	10	$V = 3.92\ W^{0.752}\ I_p^{0.409}$	15	3-5
West	25	$V = 6.18\ W^{0.758}\ I_p^{0.405}$	14	3-6
West	50	$V = 9.01\ W^{0.779}\ I_p^{0.368}$	13	3-7
West	100	$V = 17.26\ W^{0.809}\ I_p^{0.319}$	13	3-8

Note: V = volume in ac-ft, W = drainage area in mi^2, I_p = peak discharge in cfs

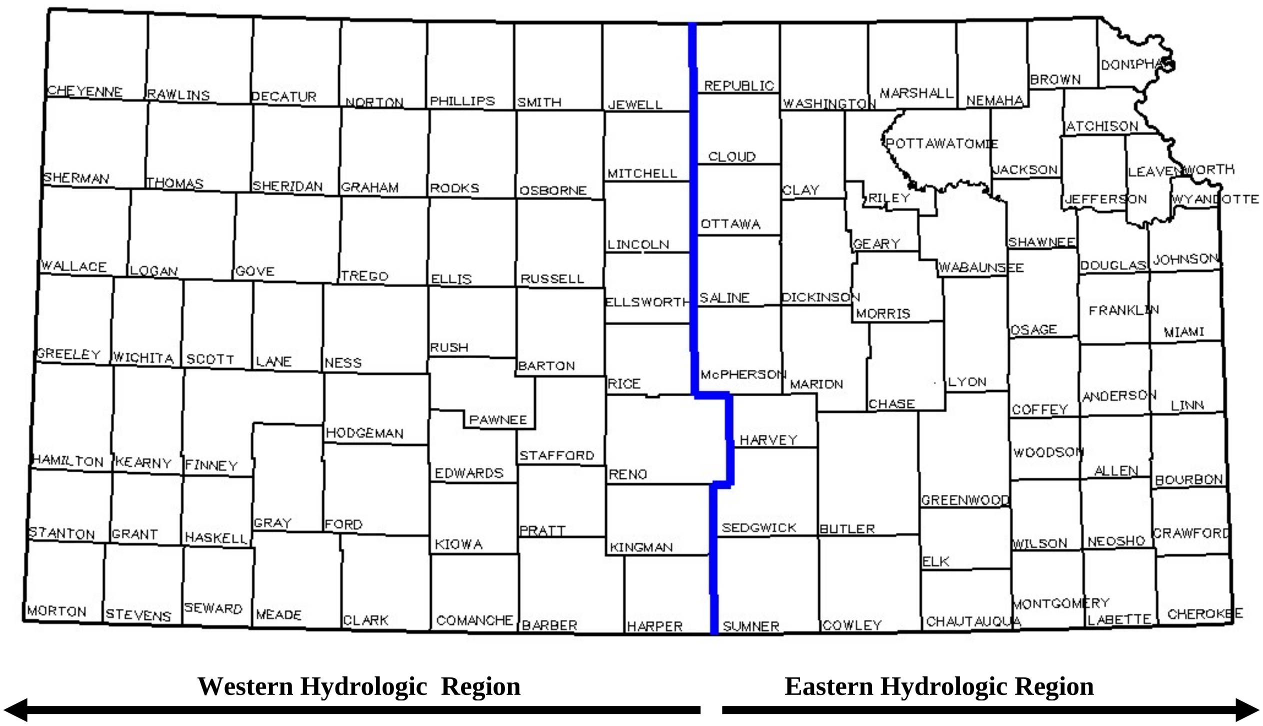

Figure 3-1: Eastern and Western Hydrologic Regions of Kansas

4. An approximate depth-area relationship must be developed for the ponding area upstream of the culvert. The required form of this relationship is

$$A = A_d \left(\frac{h}{d}\right)^m \qquad (3\text{-}10)$$

in which h is the headwater depth, d is the diameter or rise of the culvert, A is the pool area, A_d is the pool area at h = d, and m is a site-specific constant. The values m and A_d are determined from measured or estimated pool areas at two or more headwater depths. McEnroe and Gonzalez (2004) suggest a simple method for acquiring these data by survey where they cannot be estimated reliably from existing maps or survey data.

The values of m and A_d can be computed from the two data points with the equations 3-11 and 3-12.

$$m = \frac{\log\left(\frac{A_2}{A_1}\right)}{\log\left(\frac{h_2}{h_1}\right)} \tag{3-11}$$

$$A_d = A_1\left(\frac{d}{h_1}\right)^m \tag{3-12}$$

in which A_1 is the area at a depth h_1 and A_2 is the area at a depth h_2.

5. Compute the storage factor, C_S, for the culvert. For pipe culverts,

$$C_S = 2.13 \ \frac{A_d}{t_p \, N \, d^{1.5}} \tag{3-13}$$

for A_d in acres, t_p in hours and d in feet. For box culverts,

$$C_S = 2.13 \ \frac{A_d}{t_p \, B \, d^{0.5}} \tag{3-14}$$

for A_d in acres, t_p in hours, B in feet and d in feet.

6. Compute the discharge factor, C_I, for the culvert. For a pipe culvert,

$$C_I = 0.176 \ \frac{I_p}{N \, d^{2.5}} \tag{3-15}$$

in which I_p is in cfs, d is the pipe diameter in inches, and N is the number of barrels (parallel pipes). For a box culvert,

$$C_I = 0.176 \frac{I_p}{B\, d^{1.5}} \qquad (3\text{-}16)$$

in which I_p is in cfs, B is the span in feet and d is the rise in feet.

7. Find the ratio Q_p/I_p by interpolation in the appropriate table in the Appendix, as directed in Table 3-2.

Table 3-2. Where to Find Q_p/I_p

m	Pipe culverts	Box culverts
$m \le 1.75$	Table A-1	Table A-6
$1.75 < m \le 2.25$	Table A-2	Table A-7
$2.25 < m \le 2.75$	Table A-3	Table A-8
$2.75 < m \le 3.25$	Table A-4	Table A-9
$m > 3.25$	Table A-5	Table A-10

8. Multiply the peak discharge upstream of the culvert, I_p, by the ratio Q_p/I_p to obtain the peak discharge through the culvert, Q_p.

9. Compute the headwater levels for inlet control and outlet control at the peak culvert discharge, Q_p, as directed in the KDOT Design Manual, Volume I, Section 14. If the headwater level for outlet control is significantly higher than the level for inlet control, the results from the simplified method may not be realistic, and a complete analysis (as in Chapter 2) may be warranted. If higher of the two headwater levels overtops the roadway, then the simplified method is not applicable (the actual storage effect would be less than indicated by the simplified method) and a complete analysis may be warranted.

3.3 Example

Problem

An existing box culvert in rural Shawnee County will be replaced with a larger structure when the road is improved. The existing structure is a double 6 ft x 6 ft RCB; the replacement structure will be a double 10 ft x 6 ft RCB. The new culvert is designed for a 50-year recurrence interval. The 50-year discharge from the 605-acre watershed is 1220 cfs. Detention storage upstream of this culvert appears to be significant. At a headwater depth of 7.0 feet, 4.19 acres would be inundated; and at a headwater depth of 13.0 feet, 25.23 acres would be inundated. Estimate the peak flows through the existing and proposed culverts for the 50-year recurrence interval.

Solution

Shawnee County is located in the eastern hydrologic region of Kansas, as defined by Figure 3-1. The 50-year flood volume is obtained from equation 3-3.

$$V = 100.6\ W^{0.901}\ I_p^{0.135} = 100.6\ (605/640)^{0.901}\ (1220)^{0.135} = 250 \text{ acre-feet}$$

The time-to-peak is computed with equation 3-9.

$$t_p = 9.45\,\frac{V}{I_p} = 9.45\left(\frac{250}{1220}\right) = 1.94 \text{ hours}$$

The values of m and A_d in the area-depth relationship are computed with equations 3-11 and 3-12.

$$m = \frac{\log\left(\frac{A_2}{A_1}\right)}{\log\left(\frac{h_2}{h_1}\right)} = \frac{\log\left(\frac{25.23}{4.19}\right)}{\log\left(\frac{13.0}{7.0}\right)} = 2.90$$

$$A_d = A_1\left(\frac{d}{h_1}\right)^m = 4.19\left(\frac{6.0}{7.0}\right)^{2.90} = 2.68 \text{ acres}$$

The storage and discharge factors are for the existing and proposed culverts are computed with equations 3-14 and 3-16.

Existing culvert:

$$C_S = 2.13 \frac{A_d}{t_p B d^{0.5}} = 2.13 \frac{2.68}{1.94 (12)(6.0)^{0.5}} = 0.100$$

$$C_I = 0.176 \frac{I_p}{B d^{1.5}} = 0.176 \frac{1220}{12 (6.0)^{1.5}} = 1.217$$

Proposed culvert:

$$C_S = 2.13 \frac{A_d}{t_p B d^{0.5}} = 2.13 \frac{2.68}{1.94 (20)(6.0)^{0.5}} = 0.060$$

$$C_I = 0.176 \frac{I_p}{B d^{1.5}} = 0.176 \frac{1220}{20 (6.0)^{1.5}} = 0.730$$

The values of the ratio Q_p/I_p for the existing and proposed culverts are found by interpolation in Table 3-9 for m = 3.0.

Existing culvert: $Q_p/I_p = 0.800$

$Q_p = 0.800\ I_p = 0.800\ (1220) = 976$ cfs

Proposed culvert: $Q_p/I_p = 0.961$

$Q_p = 0.961\ I_p = 0.961\ (1220) = 1172$ cfs

Replacement of the existing double 6 ft x 6 ft RCB with a new double 10 ft x 6 ft RCB would increase the 50-year discharge from 976 cfs to 1172 cfs, a 20% increase.

The culvert replacement would have a beneficial impact on upstream flooding. The peak headwater level for the 50-year event would decrease from 10.45 ft to 7.72 ft, a 26% reduction. The peak area of inundation would decrease from 13.39 acres to 5.57 acres, a 58% reduction. (The peak headwater levels were determined from the inlet-control table for RCB culverts in

KDOT's *Design Manual*, Volume I, Section 14. The corresponding areas of inundation were computed with equation 3-10.)

CHAPTER 4

SEDIMENT TRANSPORT AND EROSION

4.1 Objective and General Approach

One concern associated with enlargement of a waterway opening is the potential for increased erosion in the downstream channel. If the replacement of the culvert or bridge causes an increase in peak flows, a change in channel erosion might also result. On the other hand, if peak flows do not change, channel erosion would not change. A potential change in channel erosion is indicated by a change in the sediment-transport capacity of the design flood. Streambed sediment is transported only by flows above a certain threshold. Any increase in peak discharge resulting from the culvert or bridge replacement is necessarily accompanied by a reduction in the duration of flows above this threshold, because the flood volume is unchanged. This chapter investigates the resulting impact on the total volume of sediment that the flood can transport. The analysis is limited to alluvial streams. The quantitative results are not applicable to streams with cohesive beds and banks.

Channel erosion is a separate issue from local scour in the immediate vicinity of the culvert or bridge. Enlargement of the waterway opening always reduces the velocities through the opening, even if the peak discharge is unchanged. Therefore, the potential for scour is always reduced.

4.2 Sediment Transport in Alluvial Streams

Many different relations have been proposed for prediction of sediment transport in alluvial streams. These relations yield widely varying estimates of sediment discharge. The relations of Karim and Kennedy (1990) have been shown to perform better than other well-known relations over a wide range of conditions (Thompson, 1988; Bechtler and Vetter, 1989). In this chapter, Karim and Kennedy's uncoupled velocity and sediment-discharge relations for alluvial streams are used to compute the total volume of sediment transported by floods.

Karim and Kennedy's uncoupled relation for velocity in an alluvial channel is

$$\frac{U}{\sqrt{(G-1)\,g\,D_{50}}} = 2.822\left(\frac{q}{\sqrt{(G-1)\,g\,D_{50}^{3}}}\right)^{0.376} S^{0.310} \tag{4-1}$$

in which U is the depth-averaged velocity, G is the specific gravity of the bed material, D_{50} is the median size of the bed material, g is the gravitational constant, q is the unit discharge (discharge per unit width), and S is the channel slope (dimensionless). This velocity relation is termed uncoupled because sediment discharge does not appear as an independent variable. The corresponding relation for unit sediment discharge (sediment discharge per unit width) in an alluvial channel is

$$\begin{aligned}\log\left(\frac{q_s}{\sqrt{(G-1)\,g\,D_{50}^{3}}}\right) &= -2.279 + 2.972\,\log\left(\frac{U}{\sqrt{(G-1)\,g\,D_{50}}}\right)\\ &+1.060\,\log\left(\frac{U}{\sqrt{(G-1)\,g\,D_{50}}}\right)\log\left(\frac{u_* - u_{*c}}{\sqrt{(G-1)\,g\,D_{50}^{3}}}\right)\\ &+0.299\,\log\left(\frac{y}{D_{50}}\right)\log\left(\frac{u_* - u_{*c}}{\sqrt{(G-1)\,g\,D_{50}}}\right)\end{aligned} \tag{4-2}$$

in which q_s is the unit sediment discharge (volumetric), y is the depth of flow ($y = q/U$), u_* is the bed shear velocity, and u_{*c} is Shields's critical value of the bed shear velocity for incipient motion of D_{50}-size particles. The relation for bed shear velocity is

$$u_* = \sqrt{g\,y\,S} \tag{4-3}$$

Shields's critical value of the bed shear velocity is computed with the equation

$$\frac{(u_{*c})^2}{(G-1)\,g\,D_{50}} = 0.22\,\mathrm{Re}_p^{-0.6} + 0.06 \cdot 10^{(-7.7\,\mathrm{Re}_p^{-0.6})} \tag{4-4}$$

in which Re_p is the particle Reynolds number for the median sediment size (Brownlie, 1981). The particle Reynolds number is defined as

$$\mathrm{Re}_p = \frac{\sqrt{(G-1)\,g\,D_{50}^3}}{\nu} \tag{4-5}$$

in which υ is the kinematic viscosity of the water. The units of the variables in equations 4-1 through 4-5 must be such that all quotients are dimensionless.

Equations 4-1 through 4-5 provide an estimate of the unit sediment discharge for specified values of five independent variables: q, υ, D_{50}, G and S.

4.3 Effect of Detention Storage on Downstream Sediment Transport

A spreadsheet program was developed to investigate the effect of detention storage on the total volume of sediment that could be transported by a flood. Figure 4-1 shows the family of dimensionless hydrographs used in this analysis. Discharge and time are normalized by q_{po} and t_{po}, which represent the peak unit discharge and time-to-peak with no detention storage. The six hydrographs, which all have the same dimensionless volume, represent hydrographs from the same flood volume with peak flows reduced by 0%, 10%, 20%, 30%, 40% and 50% due to detention storage. The peaks on the storage-affected hydrographs occur where these hydrographs intersect the falling limb of the no-storage hydrograph.

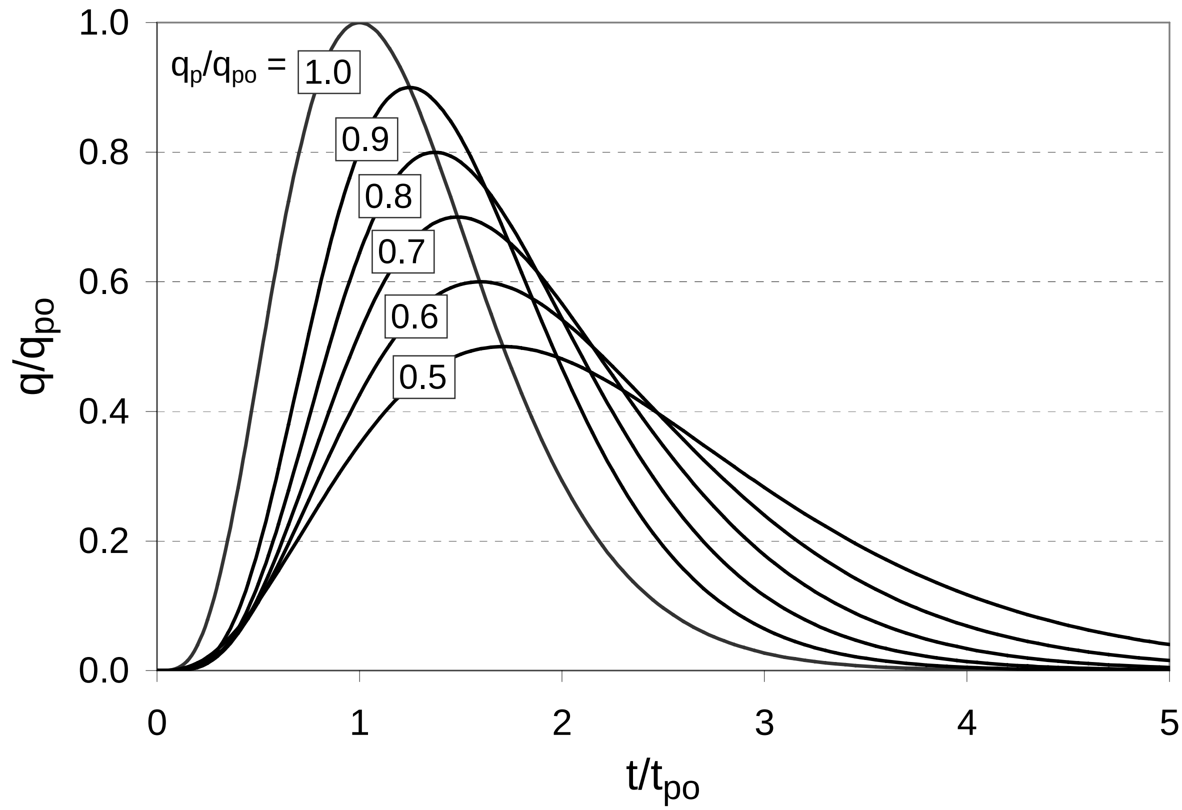

Figure 4-1: Dimensionless Hydrographs for Detention Storage Analysis

Each of the six hydrographs is described by an equation of the form

$$\frac{q}{q_p} = \left(\frac{t}{t_p}\right)^k \exp\left[-k\left(\frac{t}{t_p}\right) - 1\right] \qquad (4\text{-}6)$$

in which q_p is the peak unit discharge, t is time from the start of the flood, t_p is the time-to-peak, and k is a hydrograph shape factor. The no-storage hydrograph was assigned a shape factor of 4. The shape factors for the other hydrographs were adjusted so these hydrographs all have the same volume as the no-storage hydrograph. Table 4-1 shows the values of q_p/q_{po}, t_p/t_{po} and k for the six hydrographs.

Table 4-1: Specifications for Dimensionless Hydrographs in Figure 4-1

q_p/q_{po}	t_p/t_{po}	k
1.0	1.000	4.000
0.9	1.247	4.994
0.8	1.372	4.786
0.7	1.484	4.303
0.6	1.594	3.672
0.5	1.709	2.961

In the sediment-transport analysis, the peak unit discharge and time-to-peak on the no-storage hydrograph were specified and the six dimensionless hydrographs were converted to dimensional form, and the volumes of sediment that could be transported by these floods were computed and compared. The other inputs were the channel slope, the median sediment size, the specific gravity of the sediment, and the kinematic viscosity of the water. These calculations and comparisons were performed for many feasible combinations of these inputs.

Figures 4-2, Table 4-2 and Figure 4-3 and show representative results for the following baseline inputs: q_{po} = 100 ft^2/s, t_{po} = 3 hours, S = 0.002 ft/ft, D_{50} = 0.05 in. (coarse sand), G = 2.65 and υ = 0.0000122 ft^2/s (water at 60º F). Figure 4-2 shows the sediment discharge hydrographs for q_p/q_{po} = 1.0, 0.9, 0.8, 0.7, 0.6 and 0.5. Table 4-2 compares the volumes of sediment transported by the six flood hydrographs; V_s is the volume of sediment transported per unit width, and V_{so} is the volume of sediment transported by the no-storage hydrograph. For these baseline inputs, detention storage reduces the volume of sediment that the flood can transport, with the percentage reduction in the sediment volume exceeding the percentage reduction in the peak flow. The relationship between V_s/V_{so} and q_p/q_{po} for the baseline inputs is shown graphically in Figure 4-3.

These calculations were repeated for many other feasible combinations of inputs, with each input varied over a typical range. Although the volumes of sediment transported are sensitive to the values of all six inputs, the effect detention storage on downstream sediment transport, as indicated by the relationship between V_s/V_{so} and q_p/q_{po}, exhibits little sensitivity to any of the inputs. In all cases, an increase in detention storage resulted in a decrease in the volume of sediment transported, with the percentage reduction in the volume of sediment transported exceeding the percentage reduction in the peak discharge.

These results indicate that where enlargement of a waterway opening causes the peak flow to increase because detention storage is reduced, the volume sediment that the flood can transport also increases. This increase in sediment transport capacity may lead to an increase in channel erosion downstream of the structure. However, a reliable quantitative prediction of erosional impacts is not possible.

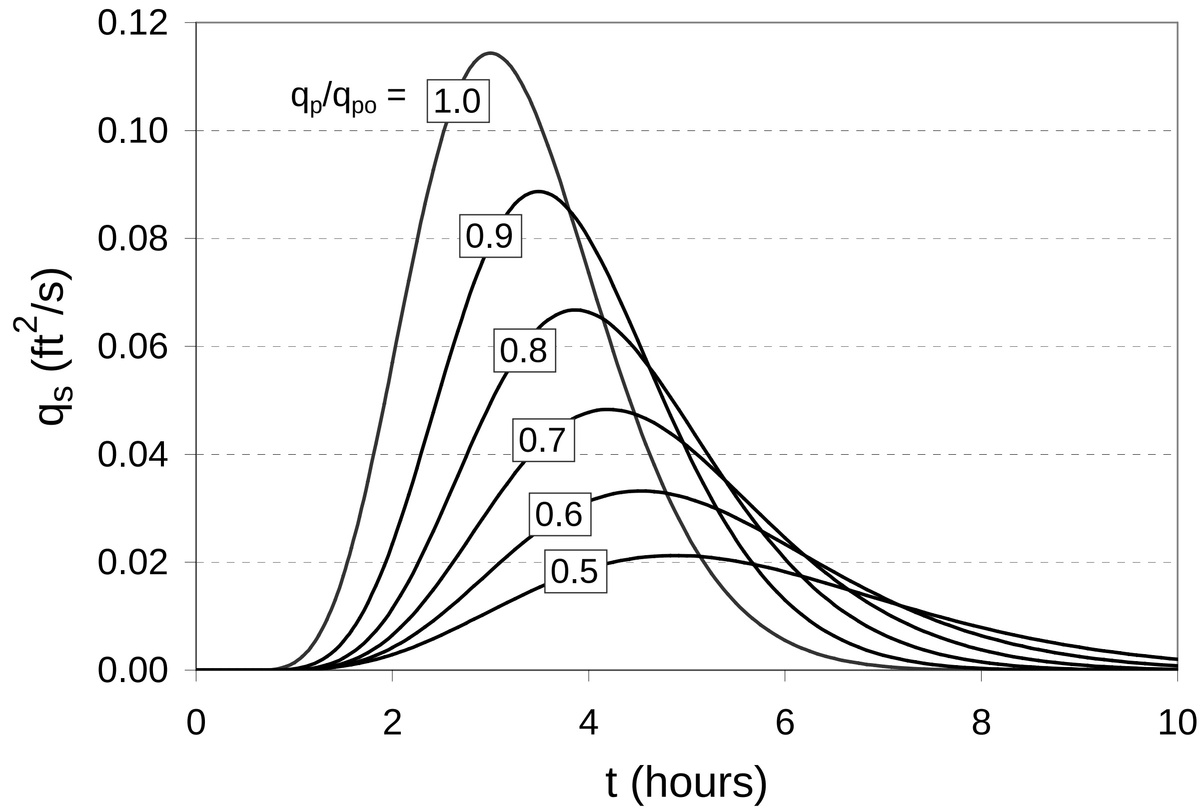

Figure 4-2: Sediment Discharge Hydrographs for Baseline Inputs

Table 4-2: Effect of Detention Storage on Downstream Sediment Transport for Baseline Inputs

q_p/q_{po}	V_s (ft^3/ft)	V_s/V_{so}
1.0	1000	1.00
0.9	863	0.86
0.8	728	0.73
0.7	599	0.60
0.6	477	0.48
0.5	362	0.36

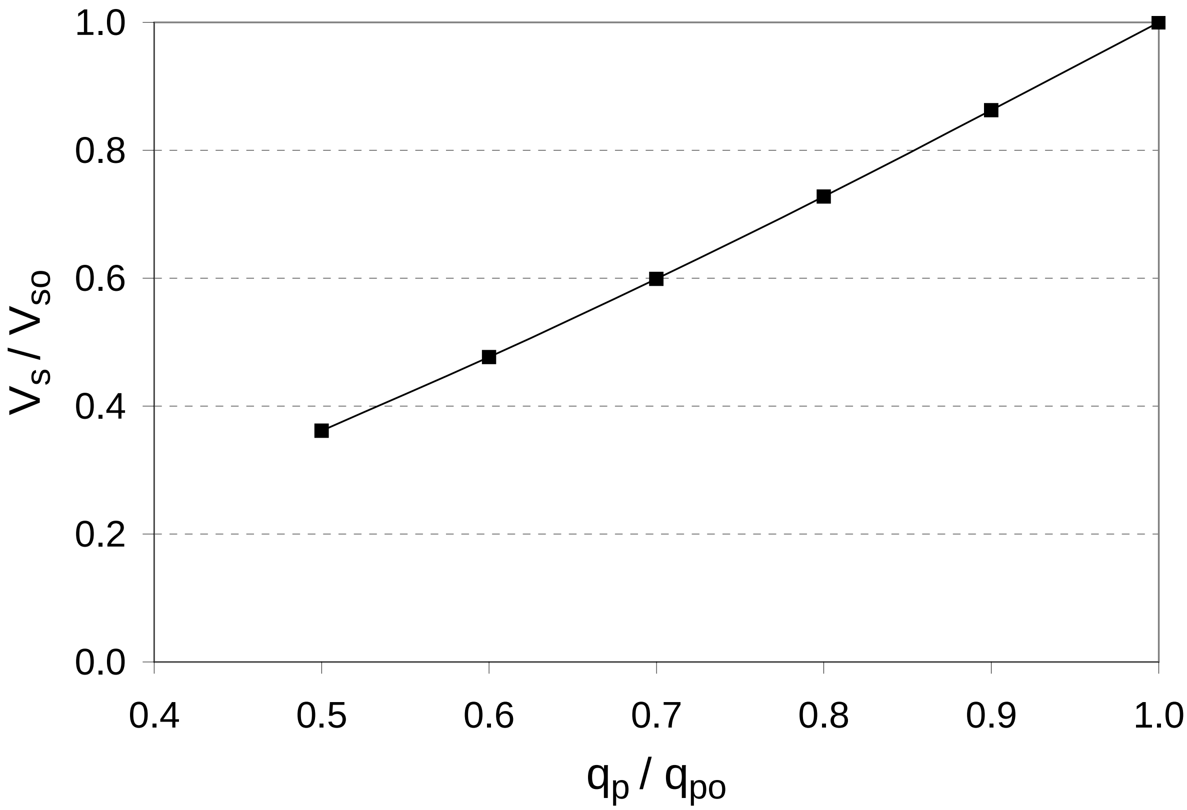

Figure 4-3: Relative Effect of Detention Storage on Downstream Sediment Transport for Baseline Inputs

CHAPTER 5

SUMMARY AND CONCLUSIONS

The replacement of a culvert or bridge with a larger structure yields two benefits in all cases. The first benefit is less frequent flooding of the roadway and/or upstream structures due to lower headwater levels. The second benefit is a lesser potential for scour through the bridge opening or at the culvert outlet due to lower velocities through the larger opening. Downstream impacts, if any, are project-specific.

If the roadway over the existing structure is overtopped by floods, enlargement of the structure will increase the flow through the structure and decrease or eliminate the roadway overflow. However, the peak flow in the channel directly downstream of the structure will not necessarily change. If the stream crossing includes a relief structure located some distance from the main structure, or if roadway overtopping occurs at some distance from the main structure, split flow can occur for a short distance downstream of the crossing. Enlargement of the main structure will increase the flow through the main structure and reduce or eliminate the split flow.

If peak flows through the existing structure are affected by detention storage, enlargement of the structure will increase the peak flows. The peak flows through the enlarged structure will also occur sooner, which may be significant in an analysis of downstream flooding. The increase in peak flow, if any, diminishes with distance downstream from the enlarged structure due channel and overbank storage and lateral inflows. The new streamflow characteristics will more closely resemble the natural conditions that existed before the highway was constructed. Few culverts and even fewer bridges are affected significantly by detention storage. If the peak flows through the existing structure are unaffected by detention storage or split flow, enlargement of the structure will not increase the peak flows directly downstream. Few culverts and even fewer bridges are affected significantly by detention storage.

This report presents and demonstrates two methods for predicting the changes in downstream flood peaks. The first method, which requires flood hydrograph simulation and reservoir routing, is applicable to all cases. The second method, which does not require hydrograph simulation or routing, is applicable to culverts that operate under inlet control with no roadway overtopping.

A decrease in detention storage results in an increase in the volume of sediment that the flood can transport. This increase in sediment transport capacity may lead to an increase in channel erosion downstream of the structure. However, reliable quantitative predictions of erosional impacts are not possible. If peak flows through an existing structure are unaffected by detention storage or split flow, enlargement of the structure will not increase erosion downstream.

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APPENDIX

TABLES FOR SIMPLIFIED ANALYSIS FOR CULVERTS

Table A-1: Effect of Storage on Peak Discharge for Pipe Culverts, m = 1.5

C_S	Q_p/I_p = (Peak discharge with storage)/(Peak discharge without storage)										
	C_I = 0.5	C_I = 0.6	C_I = 0.7	C_I = 0.8	C_I = 0.9	C_I = 1.0	C_I = 1.1	C_I = 1.2	C_I = 1.3	C_I = 1.4	C_I = 1.5
0.0000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
0.0010	1.000	1.000	1.000	1.000	1.000	1.000	0.999	0.999	0.998	0.996	0.994
0.0015	1.000	1.000	1.000	1.000	1.000	0.999	0.999	0.997	0.995	0.993	0.989
0.0020	1.000	1.000	1.000	1.000	0.999	0.999	0.998	0.996	0.993	0.988	0.983
0.0030	1.000	1.000	1.000	0.999	0.999	0.997	0.995	0.991	0.986	0.980	0.971
0.0040	1.000	1.000	1.000	0.999	0.998	0.996	0.992	0.987	0.980	0.971	0.961
0.0060	1.000	0.999	0.999	0.998	0.995	0.991	0.985	0.977	0.967	0.955	0.942
0.0080	0.999	0.999	0.998	0.996	0.992	0.986	0.978	0.968	0.955	0.941	0.925
0.0100	0.999	0.998	0.997	0.994	0.989	0.981	0.971	0.959	0.944	0.928	0.911
0.0150	0.997	0.995	0.994	0.989	0.980	0.969	0.955	0.939	0.921	0.902	0.882
0.0200	0.995	0.992	0.989	0.982	0.971	0.957	0.940	0.921	0.901	0.881	0.860
0.0300	0.990	0.984	0.978	0.969	0.954	0.935	0.914	0.893	0.870	0.847	0.825
0.0400	0.984	0.976	0.966	0.956	0.937	0.916	0.893	0.869	0.845	0.822	0.799
0.0600	0.972	0.958	0.943	0.930	0.908	0.884	0.859	0.833	0.808	0.783	0.759
0.0800	0.960	0.940	0.922	0.906	0.883	0.858	0.831	0.805	0.779	0.754	0.730
0.1000	0.948	0.924	0.902	0.883	0.861	0.835	0.808	0.781	0.755	0.731	0.707
0.1500	0.921	0.888	0.860	0.835	0.814	0.788	0.762	0.736	0.710	0.686	0.664
0.2000	0.897	0.857	0.825	0.798	0.774	0.751	0.726	0.701	0.677	0.653	0.632
0.3000	0.857	0.809	0.772	0.741	0.714	0.691	0.670	0.648	0.626	0.605	0.585
0.4000	0.819	0.771	0.731	0.699	0.671	0.646	0.625	0.606	0.587	0.568	0.550
0.6000	0.739	0.715	0.672	0.638	0.610	0.585	0.563	0.544	0.527	0.512	0.497
0.8000	0.677	0.661	0.630	0.595	0.567	0.542	0.521	0.502	0.485	0.470	0.456
1.0000	0.626	0.611	0.598	0.563	0.534	0.510	0.489	0.471	0.454	0.440	0.426

Table A-2: Effect of Storage on Peak Discharge for Pipe Culverts, m = 2.0

C_S	Q_p/I_p = (Peak discharge with storage)/(Peak discharge without storage)										
	C_I = 0.5	C_I = 0.6	C_I = 0.7	C_I = 0.8	C_I = 0.9	C_I = 1.0	C_I = 1.1	C_I = 1.2	C_I = 1.3	C_I = 1.4	C_I = 1.5
0.0000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
0.0010	1.000	1.000	1.000	1.000	1.000	0.999	0.998	0.997	0.993	0.989	0.982
0.0015	1.000	1.000	1.000	1.000	0.999	0.998	0.996	0.993	0.988	0.981	0.971
0.0020	1.000	1.000	1.000	1.000	0.999	0.997	0.994	0.989	0.982	0.973	0.961
0.0030	1.000	1.000	1.000	0.999	0.997	0.994	0.989	0.982	0.971	0.958	0.944
0.0040	1.000	1.000	0.999	0.998	0.996	0.991	0.984	0.974	0.961	0.946	0.929
0.0060	0.999	0.999	0.998	0.996	0.992	0.984	0.973	0.960	0.943	0.925	0.905
0.0080	0.999	0.998	0.997	0.994	0.987	0.977	0.964	0.947	0.928	0.907	0.886
0.0100	0.999	0.997	0.996	0.991	0.983	0.970	0.954	0.936	0.915	0.893	0.870
0.0150	0.997	0.994	0.991	0.983	0.971	0.954	0.934	0.912	0.888	0.864	0.840
0.0200	0.995	0.990	0.985	0.975	0.960	0.940	0.917	0.893	0.867	0.842	0.817
0.0300	0.990	0.982	0.972	0.959	0.939	0.915	0.889	0.862	0.835	0.809	0.783
0.0400	0.984	0.972	0.959	0.944	0.921	0.895	0.867	0.839	0.811	0.784	0.758
0.0600	0.971	0.953	0.934	0.916	0.890	0.862	0.832	0.803	0.774	0.747	0.721
0.0800	0.960	0.936	0.912	0.891	0.865	0.835	0.805	0.775	0.747	0.720	0.694
0.1000	0.948	0.920	0.893	0.868	0.842	0.813	0.783	0.753	0.725	0.698	0.673
0.1500	0.923	0.886	0.853	0.822	0.796	0.768	0.739	0.711	0.683	0.658	0.634
0.2000	0.902	0.858	0.821	0.788	0.759	0.733	0.705	0.678	0.652	0.628	0.605
0.3000	0.867	0.814	0.773	0.737	0.705	0.677	0.654	0.630	0.606	0.584	0.563
0.4000	0.839	0.781	0.736	0.699	0.667	0.639	0.613	0.592	0.571	0.551	0.531
0.6000	0.771	0.731	0.684	0.646	0.613	0.585	0.560	0.538	0.518	0.501	0.485
0.8000	0.717	0.694	0.647	0.608	0.575	0.548	0.523	0.502	0.482	0.465	0.449
1.0000	0.673	0.650	0.618	0.579	0.547	0.519	0.495	0.474	0.456	0.439	0.423

Table A-3: Effect of Storage on Peak Discharge for Pipe Culverts, m = 2.5

C_S	Q_p/I_p = (Peak discharge with storage)/(Peak discharge without storage)										
	C_I = 0.5	C_I = 0.6	C_I = 0.7	C_I = 0.8	C_I = 0.9	C_I = 1.0	C_I = 1.1	C_I = 1.2	C_I = 1.3	C_I = 1.4	C_I = 1.5
0.0000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
0.0010	1.000	1.000	1.000	1.000	0.999	0.998	0.996	0.991	0.984	0.974	0.961
0.0015	1.000	1.000	1.000	1.000	0.999	0.997	0.992	0.985	0.975	0.961	0.945
0.0020	1.000	1.000	1.000	0.999	0.998	0.994	0.988	0.979	0.966	0.950	0.931
0.0030	1.000	1.000	0.999	0.998	0.995	0.990	0.980	0.967	0.950	0.931	0.910
0.0040	1.000	0.999	0.999	0.997	0.993	0.985	0.972	0.956	0.937	0.915	0.893
0.0060	0.999	0.999	0.998	0.994	0.987	0.975	0.958	0.938	0.915	0.891	0.867
0.0080	0.999	0.998	0.996	0.990	0.980	0.965	0.946	0.923	0.898	0.873	0.847
0.0100	0.998	0.996	0.994	0.987	0.974	0.956	0.935	0.910	0.884	0.857	0.831
0.0150	0.997	0.993	0.987	0.977	0.960	0.937	0.912	0.884	0.856	0.828	0.801
0.0200	0.994	0.988	0.980	0.967	0.946	0.921	0.893	0.864	0.835	0.806	0.778
0.0300	0.989	0.978	0.965	0.949	0.924	0.895	0.864	0.833	0.803	0.774	0.746
0.0400	0.983	0.968	0.950	0.932	0.904	0.873	0.842	0.810	0.779	0.750	0.722
0.0600	0.971	0.949	0.925	0.902	0.872	0.840	0.807	0.775	0.745	0.716	0.688
0.0800	0.959	0.932	0.903	0.876	0.847	0.814	0.781	0.749	0.719	0.690	0.664
0.1000	0.949	0.916	0.884	0.853	0.825	0.792	0.760	0.728	0.698	0.670	0.644
0.1500	0.925	0.884	0.846	0.811	0.780	0.749	0.718	0.688	0.660	0.633	0.608
0.2000	0.905	0.857	0.816	0.779	0.745	0.716	0.687	0.658	0.631	0.605	0.582
0.3000	0.873	0.817	0.771	0.732	0.696	0.665	0.638	0.613	0.588	0.565	0.543
0.4000	0.848	0.787	0.739	0.698	0.662	0.630	0.602	0.578	0.556	0.535	0.515
0.6000	0.793	0.742	0.691	0.649	0.614	0.582	0.555	0.531	0.509	0.489	0.472
0.8000	0.746	0.710	0.658	0.615	0.580	0.549	0.522	0.499	0.478	0.458	0.441
1.0000	0.707	0.677	0.632	0.589	0.554	0.524	0.498	0.475	0.454	0.436	0.419

Table A-4: Effect of Storage on Peak Discharge for Pipe Culverts, m = 3.0

C_S	Q_p/I_p = (Peak discharge with storage)/(Peak discharge without storage)										
	C_I = 0.5	C_I = 0.6	C_I = 0.7	C_I = 0.8	C_I = 0.9	C_I = 1.0	C_I = 1.1	C_I = 1.2	C_I = 1.3	C_I = 1.4	C_I = 1.5
0.0000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
0.0010	1.000	1.000	1.000	1.000	0.999	0.996	0.991	0.982	0.969	0.953	0.933
0.0015	1.000	1.000	1.000	0.999	0.997	0.993	0.985	0.972	0.956	0.935	0.913
0.0020	1.000	1.000	1.000	0.999	0.996	0.990	0.979	0.963	0.944	0.922	0.898
0.0030	1.000	1.000	0.999	0.997	0.992	0.982	0.967	0.948	0.925	0.900	0.874
0.0040	1.000	0.999	0.998	0.995	0.988	0.975	0.957	0.935	0.910	0.883	0.856
0.0060	0.999	0.998	0.996	0.991	0.980	0.962	0.940	0.914	0.886	0.858	0.830
0.0080	0.999	0.997	0.994	0.986	0.971	0.951	0.926	0.898	0.868	0.839	0.810
0.0100	0.998	0.995	0.991	0.981	0.964	0.941	0.913	0.884	0.854	0.824	0.794
0.0150	0.996	0.991	0.983	0.969	0.947	0.919	0.889	0.857	0.826	0.795	0.765
0.0200	0.994	0.986	0.974	0.958	0.932	0.902	0.870	0.837	0.805	0.774	0.744
0.0300	0.988	0.975	0.957	0.937	0.908	0.875	0.840	0.807	0.774	0.743	0.714
0.0400	0.982	0.964	0.942	0.919	0.887	0.853	0.818	0.784	0.751	0.721	0.692
0.0600	0.970	0.945	0.916	0.888	0.855	0.820	0.784	0.750	0.718	0.688	0.660
0.0800	0.959	0.927	0.894	0.862	0.829	0.794	0.759	0.726	0.694	0.665	0.637
0.1000	0.949	0.912	0.876	0.840	0.808	0.773	0.739	0.706	0.675	0.646	0.619
0.1500	0.926	0.881	0.839	0.800	0.764	0.732	0.699	0.668	0.639	0.611	0.586
0.2000	0.908	0.857	0.811	0.770	0.732	0.700	0.669	0.640	0.612	0.586	0.562
0.3000	0.878	0.819	0.770	0.726	0.688	0.654	0.624	0.598	0.572	0.549	0.526
0.4000	0.855	0.791	0.739	0.695	0.656	0.622	0.592	0.565	0.543	0.521	0.500
0.6000	0.810	0.750	0.696	0.651	0.613	0.579	0.550	0.524	0.500	0.479	0.461
0.8000	0.767	0.721	0.665	0.620	0.582	0.549	0.520	0.495	0.472	0.452	0.434
1.0000	0.733	0.697	0.642	0.596	0.559	0.526	0.498	0.474	0.452	0.432	0.414

Table A-5: Effect of Storage on Peak Discharge for Pipe Culverts, m = 3.5

C_S	Q_p/I_p = (Peak discharge with storage)/(Peak discharge without storage)										
	C_I = 0.5	C_I = 0.6	C_I = 0.7	C_I = 0.8	C_I = 0.9	C_I = 1.0	C_I = 1.1	C_I = 1.2	C_I = 1.3	C_I = 1.4	C_I = 1.5
0.0000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
0.0010	1.000	1.000	1.000	0.999	0.998	0.993	0.984	0.969	0.950	0.927	0.902
0.0015	1.000	1.000	1.000	0.999	0.995	0.988	0.975	0.956	0.933	0.907	0.880
0.0020	1.000	1.000	0.999	0.998	0.993	0.983	0.966	0.944	0.919	0.892	0.863
0.0030	1.000	0.999	0.999	0.995	0.987	0.973	0.952	0.926	0.898	0.869	0.839
0.0040	1.000	0.999	0.998	0.993	0.982	0.964	0.940	0.912	0.882	0.851	0.821
0.0060	0.999	0.998	0.995	0.987	0.971	0.948	0.920	0.889	0.858	0.826	0.795
0.0080	0.999	0.996	0.992	0.980	0.961	0.935	0.905	0.872	0.840	0.807	0.777
0.0100	0.998	0.994	0.988	0.974	0.952	0.924	0.892	0.858	0.825	0.793	0.762
0.0150	0.996	0.989	0.978	0.960	0.933	0.901	0.866	0.832	0.797	0.765	0.734
0.0200	0.993	0.983	0.967	0.947	0.917	0.883	0.847	0.812	0.777	0.745	0.714
0.0300	0.988	0.972	0.949	0.925	0.891	0.855	0.818	0.782	0.748	0.716	0.686
0.0400	0.982	0.960	0.934	0.906	0.871	0.833	0.796	0.760	0.726	0.695	0.665
0.0600	0.970	0.940	0.907	0.874	0.838	0.801	0.764	0.728	0.695	0.664	0.636
0.0800	0.959	0.923	0.886	0.847	0.813	0.776	0.739	0.704	0.672	0.642	0.614
0.1000	0.949	0.909	0.868	0.827	0.792	0.755	0.720	0.686	0.654	0.625	0.598
0.1500	0.927	0.879	0.832	0.789	0.750	0.716	0.682	0.650	0.620	0.592	0.567
0.2000	0.910	0.855	0.806	0.761	0.721	0.685	0.654	0.623	0.595	0.568	0.544
0.3000	0.882	0.820	0.767	0.721	0.680	0.643	0.611	0.584	0.558	0.534	0.511
0.4000	0.861	0.794	0.739	0.692	0.651	0.615	0.583	0.554	0.530	0.508	0.487
0.6000	0.823	0.756	0.699	0.651	0.611	0.575	0.544	0.517	0.492	0.470	0.451
0.8000	0.784	0.729	0.671	0.623	0.583	0.548	0.518	0.491	0.467	0.446	0.427
1.0000	0.753	0.708	0.649	0.601	0.561	0.527	0.498	0.472	0.448	0.428	0.409

<u>**Table A-6: Effect of Storage on Peak Discharge for Box Culverts, m = 1.5**</u>

C_S	Q_p/I_p = (Peak discharge with storage)/(Peak discharge without storage)							
	$C_I = 0.6$	$C_I = 0.8$	$C_I = 1.0$	$C_I = 1.2$	$C_I = 1.4$	$C_I = 1.6$	$C_I = 1.8$	$C_I = 2.0$
0.0000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
0.0010	1.000	1.000	1.000	1.000	1.000	0.999	0.998	0.997
0.0015	1.000	1.000	1.000	1.000	0.999	0.998	0.997	0.994
0.0020	1.000	1.000	1.000	0.999	0.999	0.997	0.994	0.990
0.0030	1.000	1.000	1.000	0.999	0.997	0.994	0.989	0.982
0.0040	1.000	1.000	0.999	0.998	0.995	0.991	0.984	0.974
0.0060	1.000	0.999	0.998	0.996	0.991	0.983	0.972	0.958
0.0080	1.000	0.999	0.997	0.993	0.986	0.975	0.961	0.943
0.0100	0.999	0.999	0.996	0.990	0.981	0.967	0.950	0.930
0.0150	0.999	0.997	0.992	0.982	0.967	0.948	0.926	0.902
0.0200	0.998	0.995	0.987	0.973	0.955	0.932	0.906	0.880
0.0300	0.995	0.989	0.976	0.956	0.931	0.903	0.874	0.844
0.0400	0.992	0.983	0.965	0.940	0.911	0.880	0.848	0.817
0.0600	0.983	0.969	0.944	0.912	0.878	0.843	0.809	0.776
0.0800	0.973	0.955	0.924	0.888	0.851	0.814	0.778	0.745
0.1000	0.962	0.941	0.906	0.867	0.828	0.790	0.754	0.721
0.1500	0.931	0.907	0.867	0.824	0.783	0.744	0.707	0.674
0.2000	0.902	0.875	0.833	0.789	0.748	0.709	0.673	0.641
0.3000	0.848	0.818	0.779	0.735	0.695	0.657	0.623	0.593
0.4000	0.802	0.769	0.734	0.693	0.655	0.619	0.587	0.558
0.6000	0.729	0.692	0.663	0.629	0.595	0.563	0.534	0.507
0.8000	0.673	0.635	0.605	0.579	0.550	0.521	0.495	0.471
1.0000	0.629	0.591	0.561	0.537	0.513	0.488	0.464	0.443

Table A-7: Effect of Storage on Peak Discharge for Box Culverts, m = 2.0

C_S	Q_p/I_p = (Peak discharge with storage)/(Peak discharge without storage)							
	$C_I = 0.6$	$C_I = 0.8$	$C_I = 1.0$	$C_I = 1.2$	$C_I = 1.4$	$C_I = 1.6$	$C_I = 1.8$	$C_I = 2.0$
0.0000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
0.0010	1.000	1.000	1.000	1.000	0.999	0.998	0.995	0.989
0.0015	1.000	1.000	1.000	0.999	0.998	0.995	0.990	0.981
0.0020	1.000	1.000	1.000	0.999	0.997	0.992	0.984	0.973
0.0030	1.000	1.000	0.999	0.998	0.994	0.986	0.974	0.958
0.0040	1.000	1.000	0.999	0.996	0.990	0.979	0.964	0.944
0.0060	1.000	0.999	0.997	0.992	0.982	0.966	0.946	0.922
0.0080	1.000	0.999	0.995	0.988	0.974	0.954	0.930	0.903
0.0100	0.999	0.998	0.993	0.983	0.966	0.943	0.916	0.887
0.0150	0.999	0.996	0.987	0.971	0.948	0.920	0.888	0.856
0.0200	0.998	0.993	0.981	0.960	0.932	0.900	0.866	0.832
0.0300	0.995	0.986	0.967	0.939	0.905	0.869	0.832	0.796
0.0400	0.991	0.979	0.954	0.921	0.883	0.844	0.806	0.769
0.0600	0.982	0.964	0.931	0.891	0.849	0.807	0.767	0.730
0.0800	0.972	0.949	0.910	0.866	0.822	0.779	0.739	0.702
0.1000	0.961	0.934	0.892	0.845	0.799	0.756	0.716	0.679
0.1500	0.932	0.900	0.853	0.803	0.756	0.713	0.673	0.638
0.2000	0.905	0.870	0.821	0.771	0.724	0.681	0.642	0.608
0.3000	0.857	0.817	0.770	0.721	0.675	0.634	0.598	0.565
0.4000	0.817	0.773	0.730	0.683	0.639	0.600	0.565	0.534
0.6000	0.754	0.705	0.666	0.625	0.586	0.550	0.519	0.490
0.8000	0.705	0.655	0.615	0.581	0.546	0.514	0.485	0.459
1.0000	0.666	0.615	0.576	0.544	0.514	0.485	0.458	0.434

Table A-8: Effect of Storage on Peak Discharge for Box Culverts, m = 2.5

C_S	Q_p/I_p = (Peak discharge with storage)/(Peak discharge without storage)							
	$C_I = 0.6$	$C_I = 0.8$	$C_I = 1.0$	$C_I = 1.2$	$C_I = 1.4$	$C_I = 1.6$	$C_I = 1.8$	$C_I = 2.0$
0.0000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
0.0010	1.000	1.000	1.000	0.999	0.998	0.994	0.985	0.972
0.0015	1.000	1.000	1.000	0.999	0.995	0.988	0.976	0.958
0.0020	1.000	1.000	0.999	0.998	0.993	0.983	0.966	0.945
0.0030	1.000	1.000	0.999	0.995	0.987	0.972	0.950	0.924
0.0040	1.000	1.000	0.998	0.992	0.981	0.961	0.936	0.907
0.0060	1.000	0.999	0.996	0.986	0.969	0.944	0.913	0.881
0.0080	1.000	0.998	0.993	0.980	0.958	0.928	0.895	0.860
0.0100	0.999	0.997	0.990	0.973	0.947	0.915	0.880	0.843
0.0150	0.998	0.994	0.981	0.958	0.926	0.888	0.850	0.811
0.0200	0.997	0.991	0.973	0.944	0.907	0.867	0.827	0.788
0.0300	0.994	0.983	0.957	0.921	0.879	0.835	0.793	0.753
0.0400	0.990	0.974	0.943	0.901	0.856	0.811	0.768	0.728
0.0600	0.981	0.958	0.917	0.870	0.821	0.774	0.731	0.691
0.0800	0.970	0.942	0.896	0.845	0.795	0.747	0.704	0.665
0.1000	0.959	0.927	0.877	0.824	0.773	0.726	0.683	0.644
0.1500	0.932	0.893	0.839	0.784	0.732	0.686	0.644	0.607
0.2000	0.907	0.865	0.808	0.753	0.702	0.656	0.616	0.580
0.3000	0.864	0.815	0.761	0.707	0.657	0.614	0.575	0.541
0.4000	0.828	0.774	0.724	0.672	0.625	0.583	0.546	0.514
0.6000	0.771	0.712	0.666	0.619	0.576	0.538	0.505	0.475
0.8000	0.727	0.667	0.620	0.580	0.541	0.506	0.474	0.447
1.0000	0.693	0.632	0.585	0.547	0.512	0.480	0.451	0.425

Table A-9: Effect of Storage on Peak Discharge for Box Culverts, m = 3.0

C_S	Q_p/I_p = (Peak discharge with storage)/(Peak discharge without storage)							
	C_I = 0.6	C_I = 0.8	C_I = 1.0	C_I = 1.2	C_I = 1.4	C_I = 1.6	C_I = 1.8	C_I = 2.0
0.0000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
0.0010	1.000	1.000	1.000	0.999	0.995	0.985	0.969	0.946
0.0015	1.000	1.000	0.999	0.997	0.990	0.976	0.954	0.926
0.0020	1.000	1.000	0.999	0.995	0.986	0.967	0.941	0.910
0.0030	1.000	1.000	0.998	0.991	0.976	0.952	0.921	0.886
0.0040	1.000	0.999	0.996	0.987	0.967	0.939	0.904	0.867
0.0060	1.000	0.998	0.993	0.978	0.951	0.917	0.879	0.839
0.0080	0.999	0.997	0.989	0.969	0.938	0.900	0.859	0.818
0.0100	0.999	0.996	0.985	0.961	0.926	0.885	0.843	0.802
0.0150	0.998	0.992	0.974	0.943	0.902	0.857	0.813	0.770
0.0200	0.997	0.988	0.964	0.927	0.882	0.836	0.790	0.748
0.0300	0.994	0.979	0.946	0.902	0.852	0.804	0.757	0.715
0.0400	0.989	0.969	0.930	0.881	0.830	0.780	0.733	0.691
0.0600	0.980	0.951	0.904	0.850	0.796	0.745	0.699	0.658
0.0800	0.969	0.935	0.882	0.825	0.770	0.720	0.674	0.634
0.1000	0.958	0.920	0.863	0.805	0.750	0.699	0.655	0.615
0.1500	0.932	0.887	0.826	0.766	0.711	0.662	0.619	0.580
0.2000	0.909	0.859	0.797	0.737	0.683	0.635	0.593	0.556
0.3000	0.868	0.811	0.752	0.693	0.641	0.596	0.556	0.521
0.4000	0.835	0.773	0.717	0.661	0.611	0.568	0.530	0.497
0.6000	0.783	0.717	0.664	0.613	0.567	0.527	0.492	0.461
0.8000	0.744	0.676	0.622	0.577	0.535	0.497	0.465	0.436
1.0000	0.713	0.643	0.590	0.547	0.509	0.474	0.443	0.416

Table A-10: Effect of Storage on Peak Discharge for Box Culverts, m = 3.5

C_S	Q_p/I_p = (Peak discharge with storage)/(Peak discharge without storage)							
	$C_I = 0.6$	$C_I = 0.8$	$C_I = 1.0$	$C_I = 1.2$	$C_I = 1.4$	$C_I = 1.6$	$C_I = 1.8$	$C_I = 2.0$
0.0000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
0.0010	1.000	1.000	1.000	0.997	0.989	0.972	0.945	0.913
0.0015	1.000	1.000	0.999	0.995	0.982	0.958	0.927	0.890
0.0020	1.000	1.000	0.998	0.992	0.975	0.947	0.912	0.873
0.0030	1.000	0.999	0.996	0.985	0.962	0.928	0.888	0.847
0.0040	1.000	0.999	0.994	0.979	0.950	0.913	0.871	0.828
0.0060	1.000	0.998	0.989	0.967	0.932	0.889	0.844	0.800
0.0080	0.999	0.996	0.984	0.956	0.916	0.871	0.824	0.780
0.0100	0.999	0.995	0.979	0.947	0.903	0.856	0.808	0.764
0.0150	0.998	0.990	0.966	0.926	0.877	0.827	0.779	0.734
0.0200	0.997	0.985	0.955	0.909	0.857	0.806	0.757	0.712
0.0300	0.993	0.974	0.935	0.883	0.828	0.775	0.726	0.682
0.0400	0.988	0.964	0.918	0.862	0.805	0.752	0.703	0.660
0.0600	0.978	0.945	0.891	0.830	0.772	0.719	0.671	0.629
0.0800	0.968	0.928	0.869	0.806	0.748	0.695	0.648	0.607
0.1000	0.957	0.913	0.850	0.787	0.728	0.676	0.630	0.589
0.1500	0.932	0.880	0.814	0.750	0.692	0.641	0.597	0.558
0.2000	0.909	0.853	0.786	0.722	0.665	0.616	0.573	0.536
0.3000	0.871	0.807	0.743	0.681	0.627	0.580	0.539	0.504
0.4000	0.841	0.772	0.711	0.651	0.599	0.554	0.515	0.481
0.6000	0.793	0.720	0.660	0.607	0.558	0.517	0.481	0.449
0.8000	0.757	0.682	0.622	0.573	0.529	0.490	0.456	0.426
1.0000	0.728	0.652	0.593	0.546	0.505	0.468	0.436	0.408

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